MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

ROCKY STREAM BRIDGE
STATE ROUTE 9 OVER ROCKY STREAM
CRAWFORD, MAINE

Prepared by:
Brandon Slaven
Assistant Geotechnical Engineer

Reviewed by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Washington County
WIN 18949.00

Soils Report No. 2018-26
Bridge No. 3620

August 8, 2018

Table of Contents

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Rocky Stream Bridge which carries State Route 9 over Rocky Stream in Crawford, 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 consists of a structural plate pipe culvert constructed in 1985. The culvert is approximately 96 feet long and 12.8 feet wide. The existing culvert replaced a 25-foot span concrete frame structure with mass concrete abutments founded on soil. The preexisting concrete abutments are buried and abandoned in place. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culverts are in overall poor condition with heavy rusting, pitting, and holes.

The proposed replacement structure is a 24-foot span and 8-foot rise, 88-foot long, precast concrete box culvert. 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 to match the 2H:1V (horizontal:vertical) sideslopes. 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.

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 closely match the existing. The culvert will be replaced using staged construction techniques and provide one 15-foot wide lane of alternating two-way traffic on State Route 9.

2.0 GEOLOGIC SETTING

The existing structure carries State Route 9 over Rocky Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Wesley Quadrangle, Maine, Open-file No. 86-26 (1986), indicates the surficial soils in the vicinity of the bridge project consist of swamp and tidal-marsh deposits with nearby glacial till deposits. Swamp and tidal-marsh deposits commonly consist of organic peat, silt, clay, and sand which have accumulated in depressions and poorly drained areas. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones. Glacial till includes two varieties; basil till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

According to the MGS Bedrock Geology Map of the Wesley 15' Quadrangle, Maine, Open-file No. 81-90 (1981), the project site is at a contact of the Pocomoonshine Gabbro-Diabase

Complex and the Indian Lake Granite formation. The Pocomoonshine Complex is a gabbro and diabase. The Indian Lake Granite is a muscovite-biotite granite.

3.0 SUBSURFACE INVESTIGATION

Three test borings explored subsurface conditions at the project location. Boring BB-CRS-101 was drilled west of the existing structure. Borings BB-CRS-102 and BB-CRS-102A were 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 April 21, 2016. 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 dill 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 October 2014. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.908 to the raw field N-values. This hammer efficiency factor (0.908) 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-CRS-101 and BB-CRS-102A 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 three 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 and Glacial Till underlain by igneous 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 13.7 to 15.7 feet. The unit generally consisted of:

- Brown, moist, gravelly sand, little to trace silt;
- Brown, moist to damp, sand, some gravel, trace silt;
- Brown, moist, sand, little gravel, little silt, wood;
- Cobbles; and
- Concrete structures.

Boring BB-CRS-102 encountered approximately 5.3 feet of the fill unit before advancing the roller cone bit into the remains of a concrete abutment.

Corrected SPT N-values in the fill unit ranged from 14 to 59 blows per foot (bpf), indicating the fill is medium to very dense in consistency. Two grain size analyses of the fill material resulted in the soil being classified as A-1-b under the AASHTO Soil Classification System and SW and SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from 11 to 33 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill material in borings BB-CRS-101 and BB-CRS-102A. The thickness ranged from approximately 2.8 to 9.7 feet. The deposit generally consisted of:

- Grey-brown, moist, gravelly sand, little silt;
- Grey, moist, gravelly sand, trace silt;
- Grey, wet, gravel, some sand, little silt; and
- Cobbles.

Corrected SPT N-values in the glacial till deposit ranged from 51 to 68 bpf, indicating the glacial till is very dense in consistency. One grain size analysis conducted on a sample from the glacial till deposit resulted in the sample being classified as A-1-a under the AASHTO Soil Classification System and GW-GM under the USCS. The moisture content of the tested sample was approximately 2 percent.

5.3 Bedrock

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

<table>
<tr><th>Boring</th><th>Station</th><th>Offset (feet)</th><th>Approximate Depth to Core (feet bgs)</th><th>Approximate Elevation of Top of Core (feet)</th><th>RQD</th></tr>
<tr><td rowspan="2">BB-CRS-101</td><td rowspan="2">5+81.5</td><td rowspan="2">13.0 Lt</td><td rowspan="2">18.8</td><td rowspan="2">111.7</td><td>R1, 72%</td></tr>
<tr><td>R2, 35%</td></tr>
<tr><td>BB-CRS-102A</td><td>6+26.1</td><td>13.4 Lt</td><td>23.7</td><td>105.8</td><td>R1, 42%</td></tr>
</table>

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

The bedrock recovered from BB-CRS-101 is identified as light grey, fine to medium grained, muscovite-biotite granite, hard, fresh, low to steep angle joints, close to moderately close, open. The upper 1 to 2 feet of BB-CRS-101 indicates a shattered zone. The bedrock recovered from BB-CRS-102A is identified as grey, fine to medium grained, gabbro, fresh to slightly weathered, low to steep angle joints, very close to moderately close, open. Detailed bedrock descriptions and the RQD are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.4 Groundwater

Groundwater was not observed in the test borings. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Rehabilitation of the existing culvert and replacement bridge structures were considered to satisfy the purpose and need of this project. The following alternatives were considered;

- invert lining of the existing structure,
- a precast concrete box culvert with a 1.2 bankfull width, and
- an aluminum box culvert with a 1.2 bankfull width.

It was determined that an invert lining rehabilitation would raise the invert creating a possible barrier to fish passage through the structure. The aluminum structure cannot accommodate the necessary fills on top of the structure at this location. An 88-foot long precast concrete box culvert with a 24-foot span and 8-foot rise was found to meet the site hydraulic conditions, environmental considerations, and allow for both an increased service life and simplicity of construction. The box will be embedded approximately 2 feet into the streambed and 2 feet of special fill will be placed inside to create a natural streambed.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of an 88-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: ϕ=32°, γ = 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 be slope-tapered to match the 2H:1V sideslopes. 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 that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient, K_o, of 0.47 assuming a level backslope. The at-rest earth pressure coefficient will change if the backslope conditions are different. 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 also change if the backslope conditions are different. See Appendix C – Calculations for supporting documentation.

7.1.3 Precast 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.

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 a bottom elevation ranging from approximately 111 to 116 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 26 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 43 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 14 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. 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 0.3*f'c*. See Appendix C – Calculations for supporting calculations.

7.2 Settlement

The 14 to 16-foot-thick fill unit encountered at the site is medium to very dense in consistency. The reworked glacial till deposit at the bearing elevation is also medium to very dense in consistency. These coarse-grained materials are cohesionless and undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment will remain the same as the existing vertical alignment. As a result, little to no increase in vertical overburden pressure is expected. Any settlement is anticipated to be small and will occur relatively quickly.

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. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

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, Crawford has a design freezing index (DFI) of approximately 1500 F-degree days. A water content of 20% was used for coarse-grained soils. These components correlate to a frost depth of 5.7 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Gardiner, Maine lies near the 1500 F-degree isoline of Crawford. Gardiner has a DFI from the Modberg database of approximately 1489 F-degree days. A water content of 20% was assumed. These components correlate to a frost depth of approximately 5.3 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 5.7 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

Construction of the new structure will occur in stages. Staged construction will require temporary lateral earth support systems. The relatively shallow bedrock surface indicates cantilever earth support systems are unlikely to develop the necessary resistance to support earth loads. The Contractor should consider externally braced sheet pile walls and backfilled double sheet pile walls, or other internally supported retaining systems, for lateral earth support. Lateral earth support systems also need to accommodate the removal of the portions of the bridge structure constructed in 1938 remaining in the existing approaches and the existing pipe culvert while complying with the requirements and guidance in the AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Edition, 2017.

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 or boulders 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

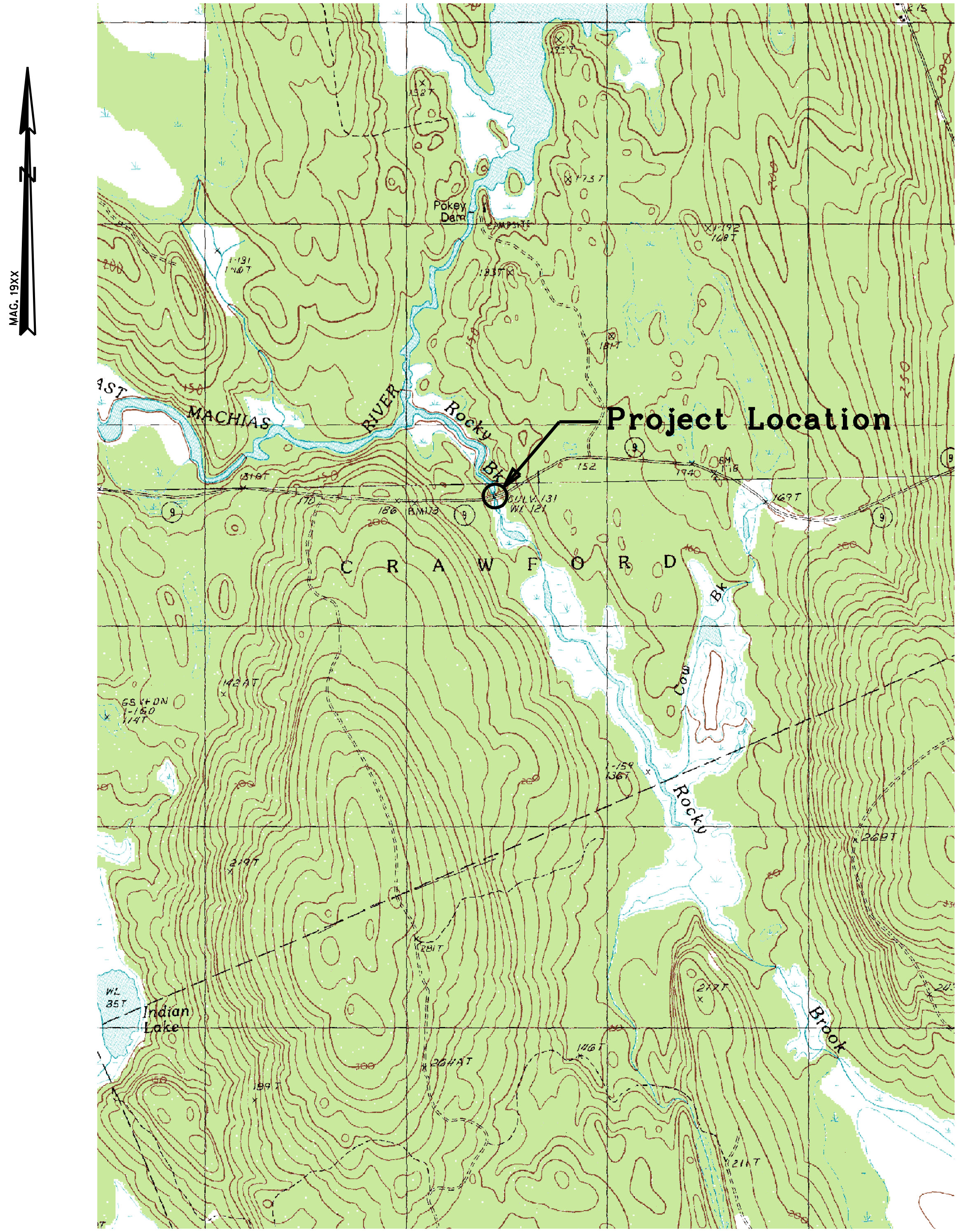
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed construction of Pembroke Stream Bridge in Crawford, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed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

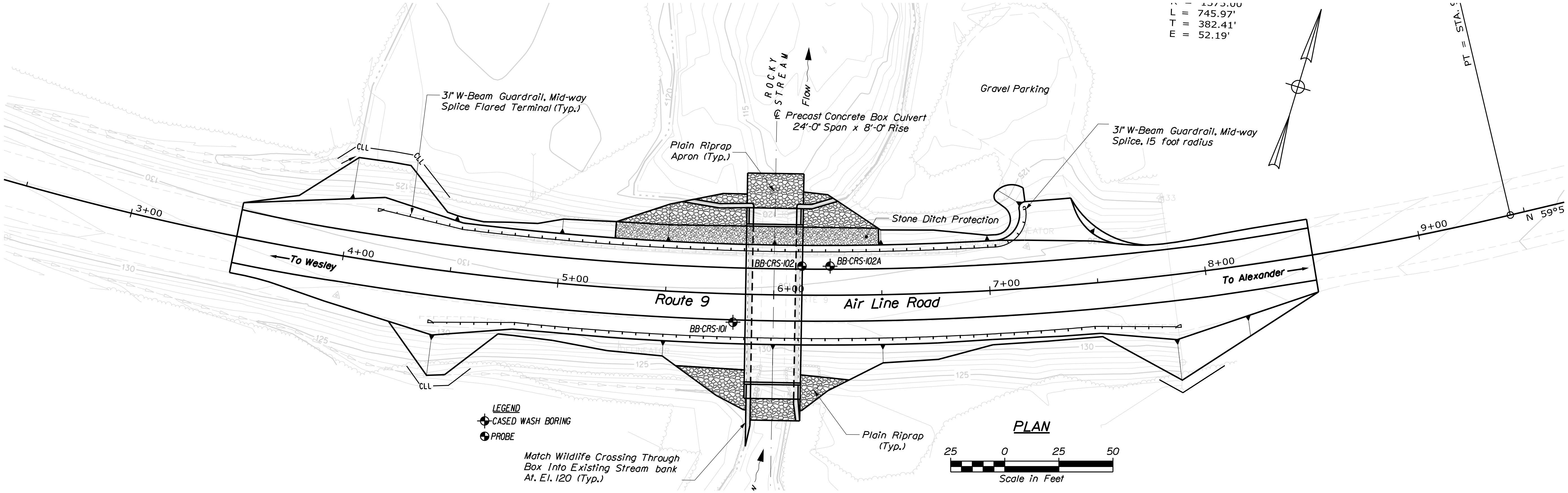
Sheets

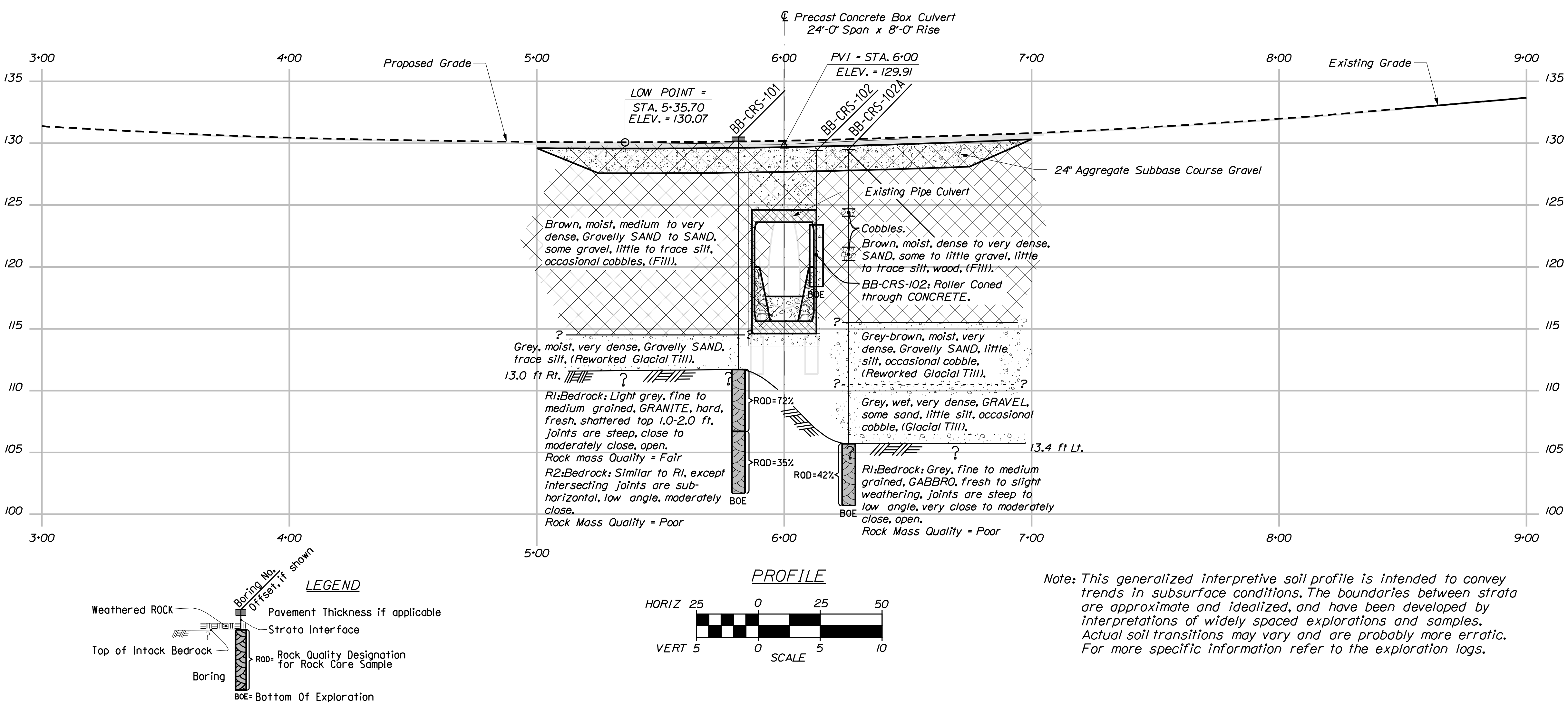
Username: terry.white Division: GEOTECH Date:1/25/2017



SHEET NUMBER	ROCKY STREAM BRIDGE ROCKY STREAM	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	CRAWFORD WASHINGTON COUNTY	018949.00
OF 3	LOCATION MAP	WIN BRIDGE NO. 3620 18949.00 BRIDGE PLANS

Filename: ...\GEOTECH\MSTA\005_BLP&ISP1.dgn   Division: GEOTECH   Username: Brandon.Slaven   Date:8/9/2018





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

018949.00

WIN 18949.00

BRIDGE NO. 3620   BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	
DESIGN3-DETAILED3	T.WHITE	MAY 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

ROCKY STREAM BRIDGE
ROCKY STREAM
CRAWFORD   WASHINGTON COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\msta\007_BORING LOGS1.dgn   Division: GEOTECH   Username: Terry.White   Date:5/25/2018

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

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-101
WIN: 18949.00

Driller:	MaineDOT	Elevation (ft.)	130.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+81.5, 13.0 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		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 = 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	130.2	4" HMA. (0.3)	
	1D	24/16	1.50 - 3.50	6/15/22/22	37	56			Brown, moist, very dense, Gravelly, fine to coarse SAND, little silt, (Fill).	
5	2D	24/15	5.00 - 7.00	8/8/8/8	16	24			Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#303575 A-1-b, SW WC=33.2%
10	3D	24/10	10.00 - 12.00	4/5/4/4	9	14	15		Brown, moist, medium dense, Gravelly, fine to coarse SAND, trace silt, occasional cobbles, (Fill).	
							12			
							11			
							17			
							42			
15	4D	24/12	15.00 - 17.00	14/9/30/30	39	59	10	114.5	16.0 Grey, moist, very dense, Gravelly, fine to coarse SAND, trace silt, (Reworked Glacial Till). Roller Coned ahead to 18.8 ft bgs.	
							43			
							100 RC			
	R1	60/60	18.80 - 23.80	RQD = 72%			NQ-2	111.7	18.8 Top of Bedrock at Elev. 111.7 ft. R1:Bedrock: Light Grey, fine to medium grained, muscovite-biotite GRANITE, hard, fresh, shattered zone in top 1 to 2-Ft., joints are steep, close to moderately close, open. [Indian Lake Granite] Rock Mass Quality = Fair. R1:Core Times (min:sec) 18.8-19.8 ft (5:30) 19.8-20.8 ft (5:00) 20.8-21.8 ft (5:00) 21.8-22.8 ft (5:05) 22.8-23.8 ft (5:10) 100% Recovery	
20										
	R2	60/60	23.80 - 28.80	RQD = 35%					R2:Bedrock: Similar to R1 except intersecting joints are low angle, subhorizontal, moderately close. Rock Mass Quality = Poor. R2:Core Times (min:sec) 23.8-24.8 ft (5:00) 24.8-25.8 ft (5:00) 25.8-26.8 ft (5:05) 26.8-27.8 ft (4:10) 27.8-28.8 ft (4:15) 100% Recovery	
25										
								101.7	28.8 Bottom of Exploration at 28.8 feet below ground surface.	
30										
35										
40										
45										
50										

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-CRS-101

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

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-102
WIN: 18949.00

Driller:	MaineDOT	Elevation (ft.)	129.4	Auger ID/OD:	5" Dia.
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	Drilling Method:	Solid Stem/Roller Cone	Core Barrel:	N/A
Boring Location:	6+13, 13.3 ft Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		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 = 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	129.1	4" HMA. (0.3)	
5	1D	7.2/7.2	5.00 - 5.60	20/30(1.2")	---		RC	123.8	Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobble, (Fill). (5.6) CONCRETE. Roller Coned ahead from 5.6-11.0 ft bgs.	
10								118.4	11.0 Bottom of Exploration at 11.0 feet below ground surface. Moved to BB-CRS-102A.	
15										
20										
25										

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-CRS-102

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

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-102A
WIN: 18949.00

Driller:	MaineDOT	Elevation (ft.)	129.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+26.1, 13.4 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							ssa	129.2	4" HMA. (0.3)	
5									Cobble from 4.8-5.4 ft bgs.	
									Cobble from 7.9-9.0 ft bgs.	
10	1D	24/14	10.00 - 12.00	8/9/13/7	22	33	6		Brown, moist, dense, fine to coarse SAND, little gravel, little silt, wood, (Fill).	G#303576 A-1-b, SM WC=11.1%
							10			
							11			
							34			
							42	115.5	14.0	
15	2D	24/16	15.00 - 17.00	8/14/20/20	34	51	11		Grey-brown, moist, very dense, Gravelly, fine to coarse SAND, little silt, occasional cobble, (Reworked Glacial Till).	
							26			
							30			
							49			
							80	110.5	19.0	
20	3D	24/15	20.00 - 22.00	18/22/23/23	45	68	60		Grey, wet, very dense, GRAVEL, some sand, little silt, occasional cobble, (Glacial Till).	G#303577 A-1-a, GW-GM WC=1.6%
							88			
							a90		a90 blows for 0.7 ft. Roller Coned ahead to 23.7 ft bgs.	
	R1	60/60	23.70 - 28.70	RQD = 42%			NQ-2	105.8	23.7 Top of Bedrock at Elev. 105.8 ft. R1:Bedrock: Grey, fine to medium grained, GABBRO, fresh to slight weathering, joints are steep to low angle, very close to moderately close, open. [Pocomoonshine Gabbro-Diorite Complex] Rock Mass Quality = Poor. R1:Core Times (min:sec) 23.8-24.8 ft (4:15) 24.8-25.8 ft (4:20) 25.8-26.8 ft (4:00) 26.8-27.8 ft (4:10) 27.8-28.8 ft (4:00) 100% Recovery	
25										
								100.7	28.8 Bottom of Exploration at 28.8 feet below ground surface.	
30										
35										
40										
45										
50										

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-CRS-102A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
018949.00
BRIDGE NO. 3620
WIN 18949.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	MAY 2018
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

ROCKY STREAM BRIDGE
ROCKY STREAM
CRAWFORD WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, 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, 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.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

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

MODIFIED BURMISTER SYSTEM

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

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. Density is rated according to standard penetration resistance (N-value).

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 undrained 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):

$$RQD\ (\%) = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*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, 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 Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream Location: Crawford, Maine	Boring No.: BB-CRS-101 WIN: 18949.00

Driller:	MaineDOT	**Elevation (ft.)**	130.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+81.5, 13.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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 = 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	130.2		4" HMA. ——0.3	
	1D	24/16	1.50 - 3.50	6/15/22/22	37	56				Brown, moist, very dense, Gravelly, fine to coarse SAND, little silt, (Fill).	
5	2D	24/15	5.00 - 7.00	8/8/8/8	16	24				Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#303575 A-1-b, SW WC=33.2%
10	3D	24/10	10.00 - 12.00	4/5/4/4	9	14	15			Brown, moist, medium dense, Gravelly, fine to coarse SAND, trace silt, occasional cobbles, (Fill).	
							12				
							11				
							17				
							42				
15	4D	24/12	15.00 - 17.00	14/9/30/30	39	59	10				
							43	114.5		——16.0 Grey, moist, very dense, Gravelly, fine to coarse SAND, trace silt, (Reworked Glacial Till). Roller Coned ahead to 18.8 ft bgs.	
							100 RC				
	R1	60/60	18.80 - 23.80	RQD = 72%			NQ-2	111.7		——18.8 Top of Bedrock at Elev. 111.7 ft. R1:Bedrock: Light Grey, fine to medium grained, muscovite-biotite GRANITE, hard, fresh, shattered zone in top 1 to 2-Ft., joints are steep, close to moderately close, open. [Indian Lake Granite] Rock Mass Quality = Fair. R1:Core Times (min:sec) 18.8-19.8 ft (5:30) 19.8-20.8 ft (5:00) 20.8-21.8 ft (5:00) 21.8-22.8 ft (5:05) 22.8-23.8 ft (5:10)	
20											
	R2	60/60	23.80 - 28.80	RQD = 35%							
25											

Remarks:

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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.

Boring No.: BB-CRS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-101
WIN: 18949.00

Driller:	MaineDOT	**Elevation (ft.)**	130.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+81.5, 13.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										100% Recovery R2:Bedrock: Similar to R1 except intersecting joints are low angle, subhorizontal, moderately close. Rock Mass Quality = Poor. R2:Core Times (min:sec) 23.8-24.8 ft (5:00) 24.8-25.8 ft (5:00)	
								101.7		25.8-26.8 ft (5:05) 26.8-27.8 ft (4:10) 27.8-28.8 ft (4:15) 100% Recovery	
30										28.8	
										Bottom of Exploration at 28.8 feet below ground surface.	
35											
40											
45											
50											

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.

Boring No.: BB-CRS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-102
WIN: 18949.00

Driller:	MaineDOT	**Elevation (ft.)**	129.4	**Auger ID/OD:**	5" Dia.
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	**Drilling Method:**	Solid Stem/Roller Cone	**Core Barrel:**	N/A
Boring Location:	6+13, 13.3 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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 = 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	129.1		4" HMA. 0.3	
5	1D	7.2/7.2	5.00 - 5.60	20/30(1.2")	---		RC	123.8		Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobble, (Fill). 5.6	
										CONCRETE. Roller Coned ahead from 5.6-11.0 ft bgs.	
10								118.4		11.0 **Bottom of Exploration at 11.0 feet below ground surface.** Moved to BB-CRS-102A.	
15											
20											
25											

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.

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream
Location: Crawford, Maine

Boring No.: BB-CRS-102A

WIN: 18949.00

Driller:	MaineDOT	**Elevation (ft.)**	129.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+26.1, 13.4 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:	R = Rock Core Sample	S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T_v = Pocket Torvane Shear Strength (psf)
D = Split Spoon Sample	SSA = Solid Stem Auger	$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
MD = Unsuccessful Split Spoon Sample Attempt	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
U = Thin Wall Tube Sample	RC = Roller Cone	N-uncorrected = Raw Field SPT N-value	PL = Plastic Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	WOH = Weight of 140lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOR/C = Weight of Rods or Casing	N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
MV = Unsuccessful Field Vane Shear Test Attempt	WO1P = Weight of One Person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							ssa	129.2		4" HMA. 0.3	
5										Cobble from 4.8-5.4 ft bgs.	
										Cobble from 7.9-9.0 ft bgs.	
10	1D	24/14	10.00 - 12.00	8/9/13/7	22	33	6			Brown, moist, dense, fine to coarse SAND, little gravel, little silt, wood, (Fill).	G#303576 A-1-b, SM WC=11.1%
							10				
							11				
							34				
							42	115.5		14.0	
15	2D	24/16	15.00 - 17.00	8/14/20/20	34	51	11			Grey-brown, moist, very dense, Gravelly, fine to coarse SAND, little silt, occasional cobble, (Reworked Glacial Till).	
							26				
							30				
							49				
							80	110.5		19.0	
20	3D	24/15	20.00 - 22.00	18/22/23/23	45	68	60			Grey, wet, very dense, GRAVEL, some sand, little silt, occasional cobble, (Glacial Till).	G#303577 A-1-a, GW-GM WC=1.6%
							88				
							a90			a90 blows for 0.7 ft. Roller Coned ahead to 23.7 ft bgs.	
	R1	60/60	23.70 - 28.70	RQD = 42%			NQ-2	105.8		23.7 Top of Bedrock at Elev. 105.8 ft. R1:Bedrock: Grey, fine to medium grained, GABBRO, fresh to slight	
25											

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.

Boring No.: BB-CRS-102A

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Rocky Stream Bridge #3620 carries Route 9 over Rocky Stream

Location: Crawford, Maine

Boring No.: BB-CRS-102A

WIN: 18949.00

Driller:	MaineDOT	**Elevation (ft.)**	129.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/21/2016; 07:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+26.1, 13.4 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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 = 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 140 lb. 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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										weathering, joints are steep to low angle, very close to moderately close, open. [Pocomoonshine Gabbro-Diorite Complex] Rock Mass Quality = Poor. R1:Core Times (min:sec) 23.8-24.8 ft (4:15) 24.8-25.8 ft (4:20) 25.8-26.8 ft (4:00) 26.8-27.8 ft (4:10) 27.8-28.8 ft (4:00) 100% Recovery	
								100.7		28.8	
										Bottom of Exploration at 28.8 feet below ground surface.	
30											
35											
40											
45											
50											

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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Crawford   Work Number: 18949.00

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	Classification AASHTO	Classification Frost
BB-CRS-101, 2D	5+81.5	13.1 Rt.	5.0-7.0	303575	1	33.2			SW	A-1-b	0
BB-CRS-102A, 1D	6+13	13.3 Lt.	10.0-12.0	303576	1	11.1			SM	A-1-b	II
BB-CRS-102A, 3D	6+26.1	13.4 Lt.	20.0-22.0	303577	1	1.6			GW-GM	A-1-a	I

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.

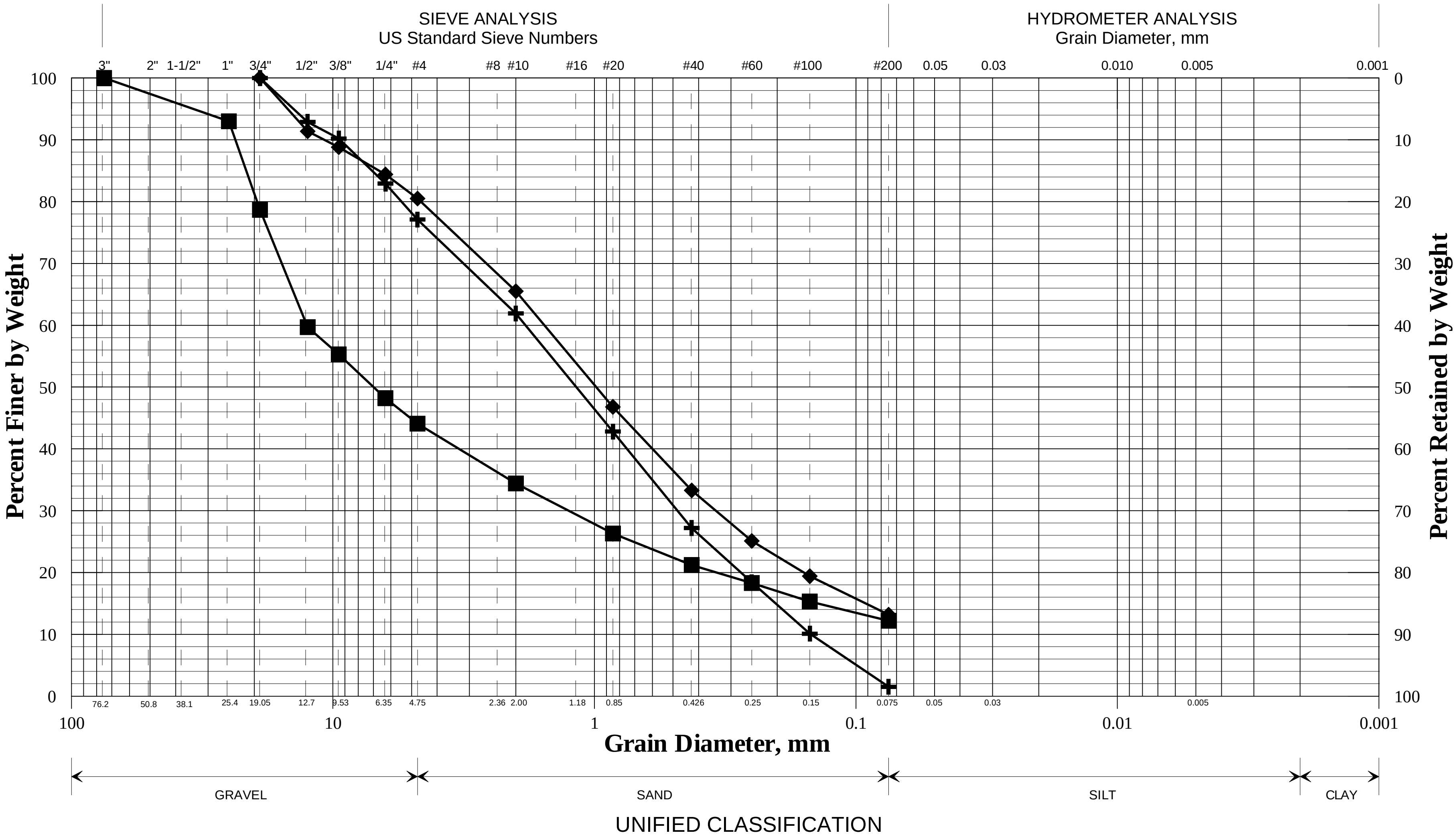
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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   NP = Non Plastic

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-CRS-101/2D	5+81.5	13.1 RT	5.0-7.0	SAND, some gravel, trace silt.	33.2			
◆	BB-CRS-102A/1D	6+26.1	13.4 LT	10.0-12.0	SAND, little gravel, little silt.	11.1			
■	BB-CRS-102A/3D	6+26.1	13.4 LT	20.0-22.0	GRAVEL, some sand, little silt.	1.6			
●									
▲									
✖									

WIN
018949.00
Town
Crawford
Reported by/Date
WHITE, TERRY A 5/9/2016

SHEET 1

Appendix C

Calculations

Earth Pressure

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 $\gamma := 125 \cdot pcf$

Internal friction angle $\phi := 32 \cdot deg$

Cohesion $c := 0 \cdot psf$

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

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

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

$$K_o = 0.47$$

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

Outlet 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 cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

β = Angle of fill slope to the horizontal $\beta := 26.56 \cdot deg$

$$K_{ar\_slope} := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \qquad K_{ar\_slope} = 0.46$$

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

Fang, Foundation Engineering Handbook 2nd ed. p. 224

6.1 AT-REST LATERAL PRESSURES

At-rest pressures exist in level ground, and develop under long-term conditions as the soil is deposited and acted upon by changes in the loading environment as caused by erosion, glaciers, and physicochemical processes. At-rest pressures rigorously only apply for walls that are placed into the ground with a minimum of disturbance and that remain unmoved during loading, or for unmoving, frictionless walls with a backfill placed with a minimum of compactive effort. In practice such conditions are rarely achieved. However, at-rest pressures are still useful in design as either a baseline against which other pressure states can be judged or as an assumed conservative choice for the design loading.

At-rest effective lateral pressures are often assumed to follow a linear distribution (Fig. 6.2), with the effective lateral pressure σ'_x taken as a simple multiple of the vertical effective pressure σ'_z:

$$\sigma'_x = K_0(\sigma'_z) \qquad (6.1)$$

In homogeneous, dry soil with a constant K_0 and unit weight, both the vertical and lateral pressures are linearly distributed. With the presence of a water table, the at-rest pressure distribution exhibits a break in slope at the water table, reflecting the use of submerged unit weights to determine vertical effective stresses (Fig. 6.2).

Our early concepts of the parameter K_0 were formed on the basis of normally consolidated soils. Jaky (1944) proposed a relationship between K_0 and the drained friction angle ϕ' for normally consolidated soils:

$$K_0 = 1 - \sin\phi' \qquad (6.2)$$

Numerous studies have confirmed the general validity of this empirical equation (Brooker and Ireland, 1965; Mayne and Kulhawy, 1982). However, results from laboratory experiments and in-situ tests have shown that the K_0 value also varies as a function of overconsolidation ratio (OCR) and stress history. For the case of a soil that has been subjected to one or more cycles of unloading, Schmidt (1966) proposed that K_0 can be determined as a function of its value in the normally consolidated state using the relationship

$$K_{0u} = K_{0nc}(\text{OCR})^{\alpha} \qquad (6.3)$$

in which K_{0u} is the coefficient for unloading, K_{0nc} is the coefficient for the normally consolidated soil, and α is a dimensionless coefficient. Experimental data have confirmed this relationship, and Mayne and Kulhawy (1982) showed that, for most soils, α can be taken as $\sin\phi'$.

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may be enough to know that the K_0 during reloading falls about halfway between that for unloading and normally consolidated conditions. Also, K_0 might be directly determined through in-situ testing methods.

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

	Coefficient of Lateral Earth Pressure			
Soil type	*OCR = 1*	*OCR = 2*[a]	*OCR = 5*[a]	*OCR = 10*[a]
Loose sand	0.45	0.65	1.10	1.50
Medium sand	0.40	0.60	1.05	1.55
Dense sand	0.35	0.55	1.00	1.50
Silt	0.50	0.70	1.10	1.60
Lean clay, CL	0.60	0.80	1.20	1.65
Highly plastic clay, CH	0.65	0.80	1.10	1.40

[a] Unloading cycle.

Table 6.1 presents typical values for K_0 for a subset of soils. For other conditions, K_0 values can be determined directly from Equations 6.2 and 6.3, and/or using in-situ testing techniques.

Because the K_0 value in a given soil often varies with depth, and the soil types themselves may change with depth, the at-rest lateral pressure distribution is typically not linear as shown in Figure 6.2. Self-boring pressuremeter tests in clays with overconsolidated profiles induced by desiccation have demonstrated that the K_0 under such conditions decreases with depth in the soil deposit and reaches a steady state where the desiccation effects are no longer present (Clough and Denby, 1980).

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

Most walls move, either by global shifting or by local deformations. These movements cause adjustments to occur in the earth loads and the pressure distributions. Conventional means for assessing the effects of system movements are to set them into the context of extreme conditions. These are referred to as the active and passive earth pressure loadings.

6.2.1 Active Pressure

Assuming that a gravity wall with no friction on its face is translated away from a soil mass that is initially at the at-rest condition, then the soil mass adjacent to the wall will pass into a failure state as shown in Figure 6.3. At this stage, the

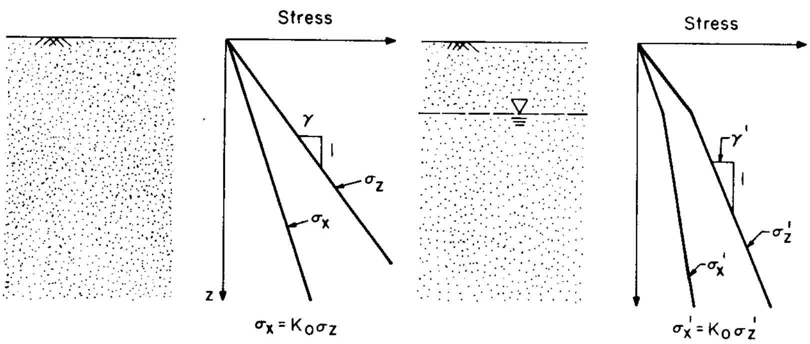


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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 β = 0°, the value of the coefficient of active earth pressure (Rankine), K_a, may be taken as:

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

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 β > 0°, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

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

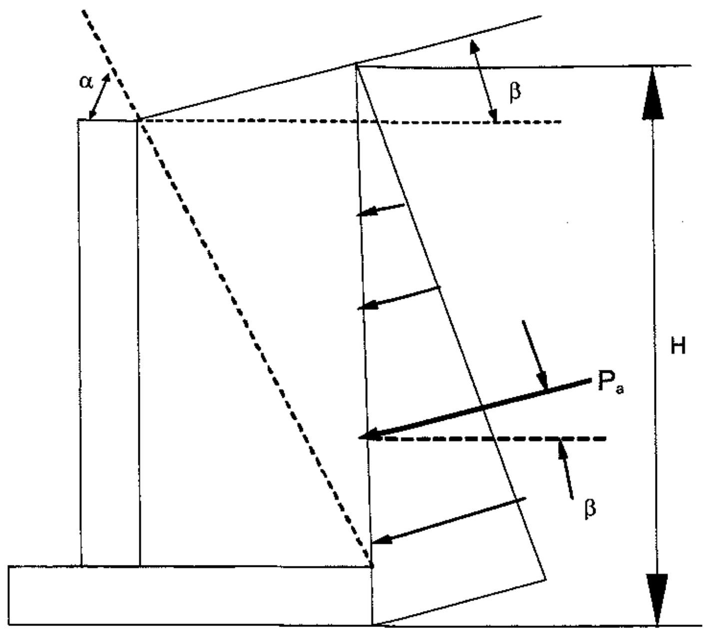


Figure 3-3 Rankine Theory

The resultant earth pressure force, P_a, is oriented at an angle, β, as shown if 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, Ka, 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 conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. The one foot thick layer of proposed Granular Borrow bedding material is ignored.
3. The proposed bearing elevation is approximately 114.5 feet.
4. Proposed finish roadway grade elevation is approximately 130 feet.
5. Proposed precast concrete box base is 26 feet wide.
6. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site. Use design N-value of 51 bpf for the consistency of the soils encountered at the box bearing elevation, based on BB-CRS-102A 2D.
7. 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
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12-20	14
	Medium dense to dense	8-14	10

The glacial till deposit is very dense. Recommend 14 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

$$B := 26ft$$

$$D_f := 2.0 \cdot ft$$

$$D_w := 0 \cdot ft$$

$$\gamma_w := 62.4 \cdot pcf$$

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

$\gamma_{above} := 125 \cdot pcf$ MainDOT Bridge Design Guide p. 3-3
Soil Type 4

Foundation soils:

$\gamma 1_d := 134 \cdot pcf$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 Dry, dense angular sand - dry unit weight

$w_{sat} := .10$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 Dry, dense angular sand - natural moisture content in a saturated state

$\gamma 1_{sat} := \gamma 1_d \cdot (1 + w_{sat})$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit weight relationships

$\gamma 1_{sat} := 139 \cdot pcf$

$N_{design} := 51$

$\phi := 40 \cdot deg$ — Lambe and Whitman, Soil Mechanics Fig. 11.14

Cohesion $c := 0$

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

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

$s_\gamma := 1.0$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for Glacial Till, ϕ = 40 degrees.

$N_c := 75.25$ $N_q := 64.1$ $N_\gamma := 93.6$

Nominal Bearing Resistance per Terzaghi equation

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $q = 0.153 \cdot ksf$ — Das Principles of Foundation Engineering 7th Ed. p. 142: Eq. 3.16 Water table modification

$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$ — Bowles Foundation Analysis and Design 5th Ed. p. 220: Table 4-1 Bearing-capacity Equations

$q_n = 103 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$B = 26 \cdot ft$

$\boxed{q_r = 46.4 \cdot ksf}$ for

Nominal Bearing Resistance for Strength Limit States

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

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

$N_c := 75.3$

$N_q := 64.2$

$N_\gamma := 109.4$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$L := 88 \cdot ft \qquad B := 26 \cdot ft$

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

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

$s_\gamma = 0.88 \qquad s_q = 1.25$

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 0.5.

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

Load Inclination factors

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

$i_c := 1.0 \qquad i_\gamma := 1.0 \qquad i_q := 1.0$

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

$$\frac{D_f}{B} = 0.077$$

Therefore : $d_q := 1.0$

Terms

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

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

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

$N_{cm} = 75.3$ $N_{\gamma m} = 96.471$ $N_{qm} = 80.116$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c \cdot N_{cm} + \gamma_{above} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma} \right]$$

$q_n = 97.2 \cdot ksf$

Factored Bearing Resistance

$\phi_b := 0.45$

$q_r := q_n \cdot \phi_b$ $q_r = 43.7 \cdot ksf$

Recommend a limiting value for the factored bearing resistance of 43 ksf or 21.5 tsf, for footings 26 ft or greater on Glacial Till.

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 δ, 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.

Das, Principles of Geotechnical Engineering 7th ed.

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	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

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/ft³	Dry unit weight, γ_d kN/m³
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

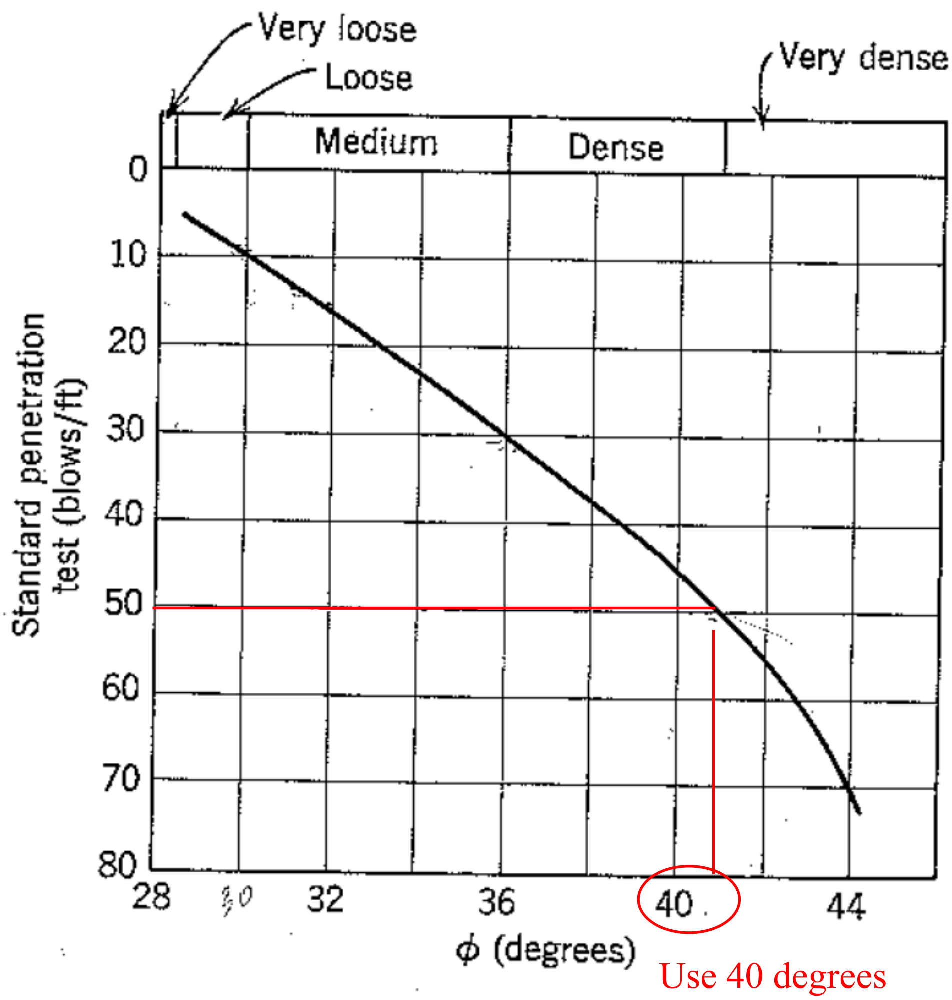


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

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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).

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

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.

10.5.5.2.2—Spread Footings

C10.5.5.2.2

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.

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.

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: **Crawford, Maine**
DFI = 1500 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1500

$d := 67.9 \cdot in$

Depth of Frost Penetration $d := 67.9\,in$ $d = 5.7 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

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

--- ModBerg Results ---

Project Location: Gardiner, Maine
Air Design Freezing Index = 1489 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1191 F-days
Mean Annual Temperature = 44.1 deg F
Design Length of Freezing Season = 128 days
--

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	64.0	20.0	110.0	30	41	2.3	1.4	3,168

--
t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.
****
Total Depth of Frost Penetration = 5.33 ft = 64.0 in.
****

Recommendation: 5.7 feet for design of foundations constructed on coarse grained soils

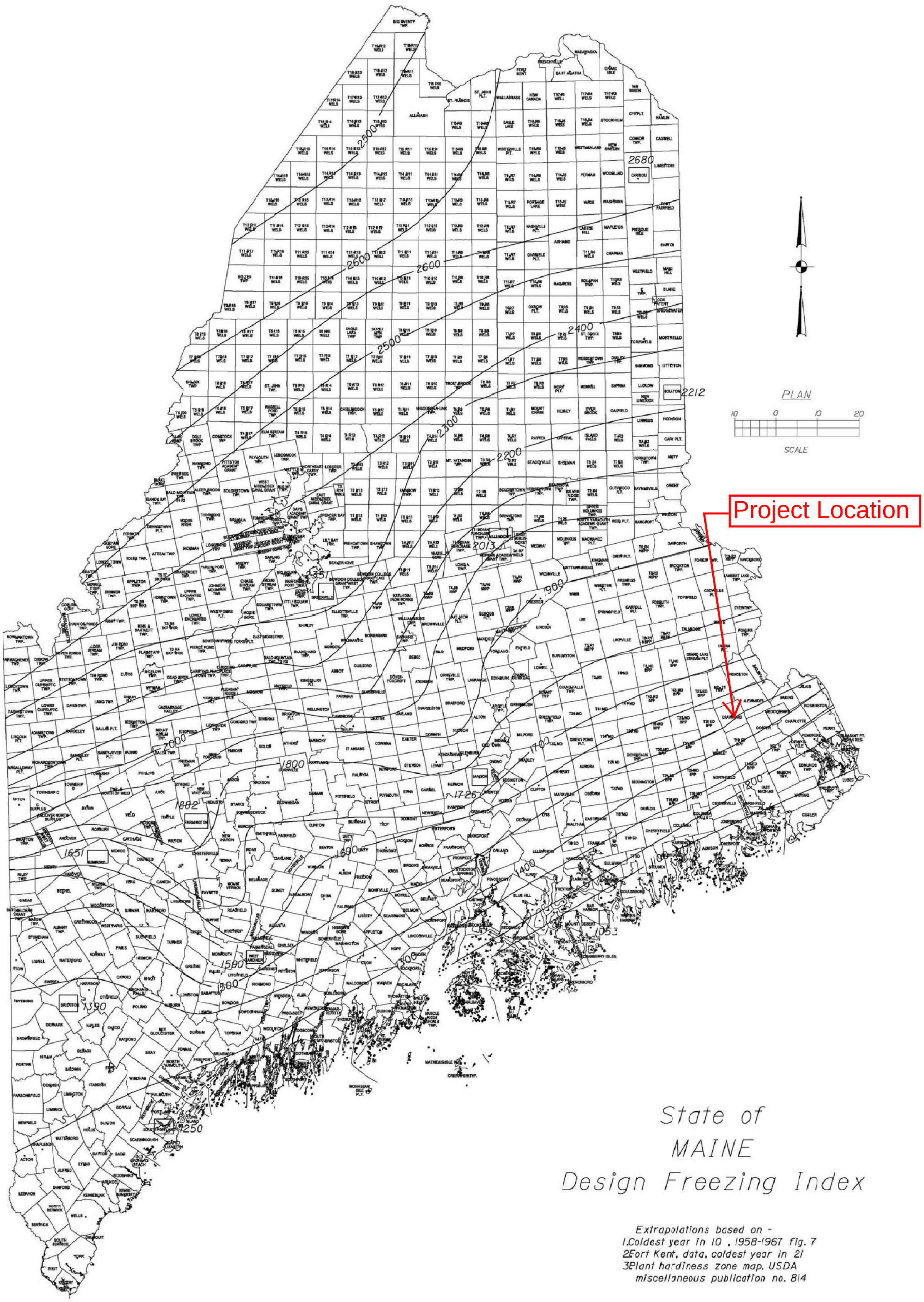

Figure 5-1 Maine Design Freezing Index Map

5.2 General

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