

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

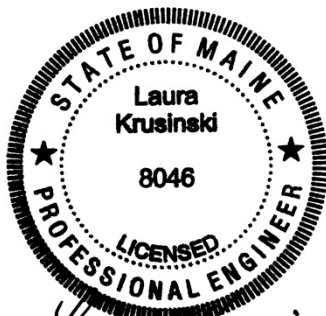
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**SOUCY BRIDGE
U.S. ROUTE 1 OVER THIBODEAU BROOK
GRAND ISLE, MAINE**

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Aroostook County
WIN 21884.00

Soils Report 2018-42
Bridge No. 2781

November 19, 2018
NHPP-2188(400)

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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 Soucy Bridge which carries U.S. Route 1 over Thibodeau Brook in Grand Isle, 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 1959. The culvert is approximately 125 feet long and 13.5 feet wide. The existing culvert replaced a 14.75-foot span concrete frame structure with mass concrete abutments constructed on a concrete mat foundation founded on soil. The preexisting concrete mat and portions of the concrete wingwalls are buried and abandoned in place. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the existing culvert is in overall poor condition (rating of 4) with heavy rusting and holes.

The proposed replacement structure is a 22-foot span and 12-foot rise, 144-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) or 2.25H:1V 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 1.

2.0 GEOLOGIC SETTING

The existing structure carries U.S. Route 1 over Thibodeau Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geologic Map of Maine, 1985, indicates the surficial soils in the vicinity of the bridge project consist of glacial till and stream alluvium. Glacial till is a heterogeneous mixture of sand, silt, clay, stones, and may include many boulders deposited directly by glacial ice. Stream alluvium is comprised of sand, gravel, and silt deposited on flood plains and stream beds by postglacial streams.

According to the MGS Bedrock Geology Map Maine, 1985, bedrock at the project site is comprised of undifferentiated interbedded pelite and sandstone.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the project location. Boring BB-GITB-101 was drilled south of the existing structure. Boring BB-GITB-102 was drilled north of the existing structure. The test boring locations are shown on Sheet 2 – Boring Location Plan.

The MaineDOT Drill Crew drilled the test borings on May 16 and 17, 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 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and open hole wash boring 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.

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 and surveyed by MaineDOT Region 5 survey personnel.

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 six standard grain size analyses with natural water content and two grain size analyses with hydrometer and natural moisture 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 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill and Glacial Till. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – 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 18.5 feet. The unit generally consisted of:

- Brown, moist, gravelly sand, trace silt;
- Brown, moist to damp, sand, some gravel, some to little silt;
- Brown, wet, sandy gravel, little to trace silt; and
- Cobbles.

Corrected SPT N-values in the fill unit ranged from 13 to 71 blows per foot (bpf), indicating the fill is medium dense to very dense in consistency. Four grain size analyses of the fill material resulted in the material being classified as A-1-a, A-1-b, and A-2-4 under the AASHTO Soil Classification System and GW-GM, GM, and SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 4 to 12 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill material in the borings. The thickness of the layer encountered in BB-GITB-101 was approximately 70.6 feet. Boring BB-GITB-102 terminated in the glacial till layer at approximately 41 feet below the roadway surface (brs). The deposit generally consisted of:

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

Corrected SPT N-values in the glacial till deposit ranged from 51 bpf to practical refusal indicating the glacial till is very dense or hard in consistency. Four grain size analyses conducted on samples from the glacial till deposit resulted in the soils being classified as A-1-a, and A-2-4 under the AASHTO Soil Classification System and GM, SM, and SC-SM under the USCS. The moisture content of the tested samples ranged from approximately 6 to 9 percent.

5.3 Groundwater

Groundwater measurements performed at the completion of the borings indicated groundwater levels ranging between 11 and 18 feet brs. Groundwater observations are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. Note that because water was introduced into the borehole during drilling operations, the groundwater measurements may not represent stabilized groundwater conditions. 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,
- concrete arch or frame supported by spread footings or piles,
- short span bridge supported by spread footings or piles, and
- a precast concrete or steel box culvert.

It was determined that an invert lining rehabilitation would increase the degradation of the stream by increasing flow velocities; therefore, the invert lining alternative was dismissed. The concrete arch or frame and short span bridge alternatives were dismissed because of the greater relative cost of the excavation, water control, and structure cost when compared to a precast concrete or steel box culvert. A precast concrete box was chosen over a steel box culvert because staged construction is required and steel box culverts can be difficult to construct in stages. As such, a 144-foot long precast concrete box culvert with a 22-foot span and 12-foot rise was found to meet the site hydraulic conditions, environmental considerations, and provide greater ease 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 a 144-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 be slope-tapered to match the 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. 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 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 453.0 to 449.5 feet. The coarse-grained glacial till encountered at this elevation is very dense in consistency. These soils are characterized as having adequate bearing resistance.

For a precast concrete box culvert with a base width of 24 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 15 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 6 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 $0.3f'_c$. See Appendix C – Calculations for supporting calculations.

7.2 Settlement

The fill unit encountered at the site is medium dense to very dense in consistency. The glacial till deposit at the bearing elevation is 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 nearly 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 fill or native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Grand Isle has a design freezing index (DFI) of approximately 2600 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 9.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Fort Kent, Maine lies near the 2600 F-degree isoline of Grand Isle and was used in the analysis because Grand Isle is not in the Modberg database. Fort Kent has a air DFI from the Modberg database of approximately 2837 F-degree days. A water content of 10% was assumed. These components correlate to a frost depth of approximately 9.6 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 9.1 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 4-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.28 - Heavy 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. Lateral earth support systems will need to accommodate the removal of the existing culvert and previous structure buried in the existing approaches.

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 very dense glacial till 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 replacement of Soucy Bridge in Grand Isle, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

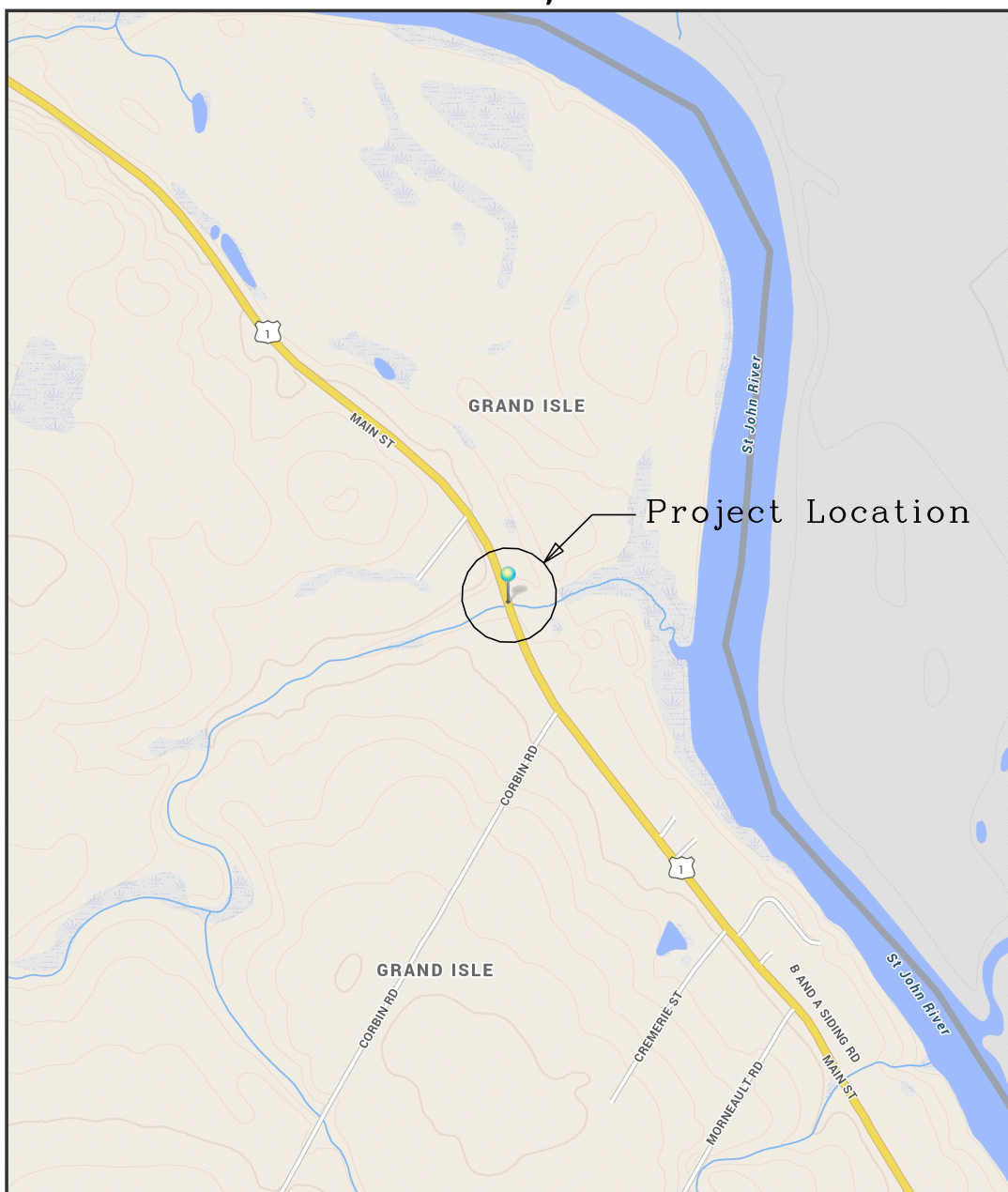
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes 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



GRAND ISLE, MAINE

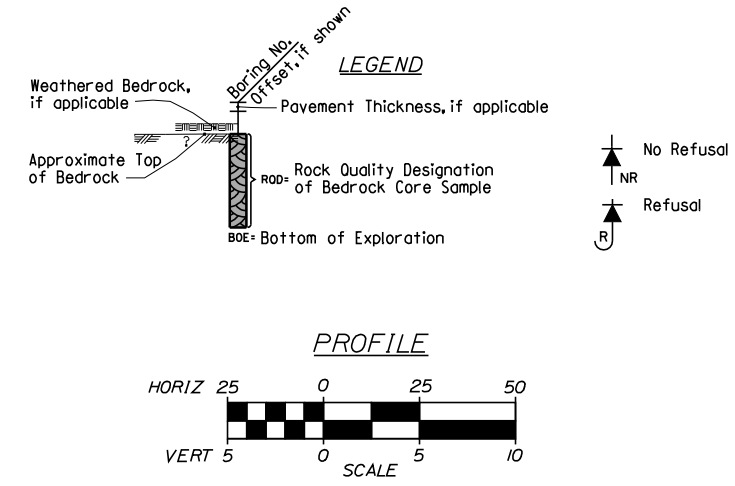
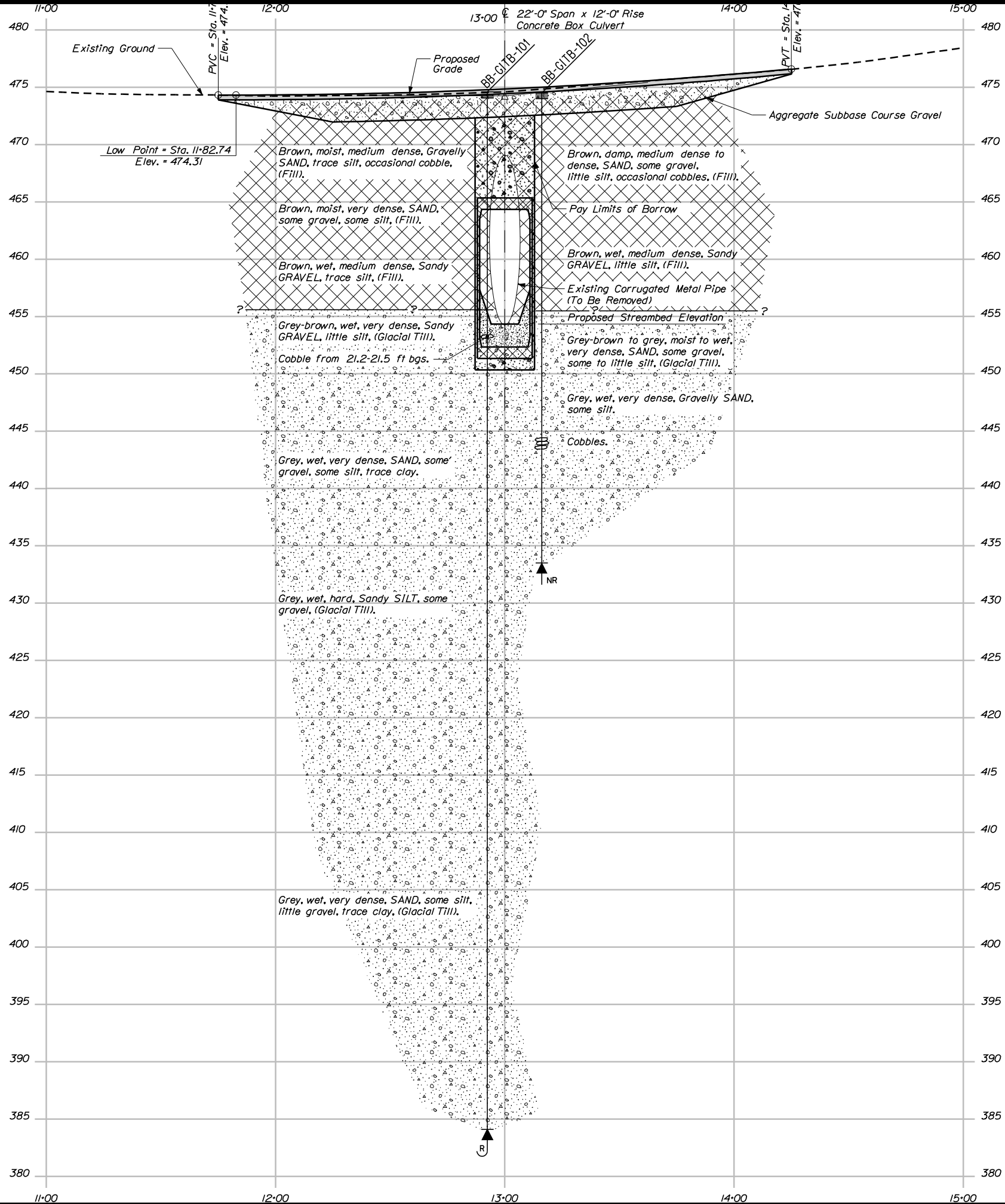


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0.25 Miles
1 inch = 0.28 miles

Date: 1/24/2018
Time: 8:10:10 AM

SHEET NUMBER 1 OF 4	SOUCY BRIDGE THIBODEAU BROOK GRAND ISLE AROOSTOOK COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
		NHPP-2188(400)	
	LOCATION MAP	WIN BRIDGE NO. 2781 21884.00 BRIDGE PLANS	



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 and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	SIGNATURE		DATE
	NHP-2188(400)		P.E. NUMBER
	WIN		DATE
BRIDGE NO. 2781		21884.00	
BRIDGE PLANS			
SOU CY BRIDGE THIBODEAU BROOK AROSTOOK COUNTY GRAND ISLE			
INTERPRETIVE SUBSURFACE PROFILE			
SHEET NUMBER			
3			
OF 4			

Maine Department of Transportation				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-101			
Soil/Bore Exploration Log US CUSTOMARY UNITS								WIN: 21884.00			
Drillers: MaineDOT				Elevation (ft.): 474.6				Auger ID/OD: 5" Solid Stem			
Operator(s): Travis/Dames/Rick				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: see Remarks				Drilling Method: Cased Wash Boring				Core Barrel: N/A			
Boring Location: 12+92.4, 11.3 ft LT.				Casing ID/OD: NW-3"				Water Level*: 18.0 ft bgs.			
Hammer Efficiency Factor 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathode ☐							
Derivations: B = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt g = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test PP = Pocket Penetration Test WSP = Weight of One Person				S _u = Peak Undrained Field Vane Undrained Shear Strength (psf) S _u (sp) = Lab. Vane Undrained Shear Strength (psf) S _u = Unconfined Compressive Strength (ksf) W = uncorrected = Raw Field SPT Blowcount Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT uncorrected Corrected for Hammer Efficiency N ₆₀ = Hammer Efficiency Factor/60/100 = uncorrected				T _c = 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			
Sample Information											
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows 1/6 in. Shear Strength or 100 (lb)	Uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ASTM and Unified Class
50	100	12/12	50.00 - 51.00	48/60	---					Similar to above.	
55	110	12/12	55.00 - 56.00	41/58	---					Gray, wet, very dense, SAND, some silt, little gravel, trace clay. (Glacial Till).	GM304389 A-2-4, SC-SM WC=9.0%
60	120	24/20	60.00 - 62.00	29/37/41/56	78	111				Similar to above.	
65	130	12/12	65.00 - 66.00	43/57	---					Similar to above.	
70	140	12/12	70.00 - 71.00	43/53	---					Similar to above.	
75											
80	150	24/18	80.00 - 82.00	25/44/42/58	86	122				Similar to above.	
85											
90	160	6/4	90.00 - 90.50	55	----			385.0 384.1		Weathered ROCK. Weathered ROCK in tip of Spoon. Bottom of Exploration at 90.5 feet below ground surface. REFUSAL	89.6 90.5
95											
100											
Remarks:											
-5/16/2017: 07:30-15:30 -5/17/2017: 07:00-10:00										Page 2 of 2	
Stratification lines represent approximate boundaries between soil type 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-GITB-101	

Maine Department of Transportation				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-G1TB-102																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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Date Start/Finish: 5/17/2017: 10:00-13:30				Drilling Method: Cased Water Boring				Core Barrel: N/A																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
Boring Location: 13+07.7, 11.7 ft Rt.				Casing ID/OD: NW-3"				Water Level#: 11.0 ft bgs.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
Hammer Efficiency Factor: 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
Derivations: S = Split Spoon Sample MU = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test PP = Pocket Penetration Test MW = Unsuccessful Field Vane Shear Test Attempt WSP = Weight of One Person				S _u = Peak/Undrained Field Vane Undrained Shear Strength (psf) S _{u(c)} = Lab. Vane Undrained Shear Strength (psf) S _{u(ub)} = Unconfined Compressive Strength (ksf) W-corrected = New Field SPT Result Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT Un-corrected Corrected for Hammer Efficiency N ₆₀ = Hammer Efficiency Factor/60/N ₆₀ uncorrected T ₂₀₀ = Pocket Torque Shear Strength (psf) W _c = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index C = Consolidation Test																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
Sample Information												Visual Description and Remarks		Laboratory Testing Results/ASTM and Unified Class																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blow Count (6 in. Shear)	Blow Count (12 in. Shear)	Blow Count (18 in. Shear)	Blow Count (24 in. Shear)	Blow Count (30 in. Shear)	Blow Count (36 in. Shear)	Blow Count (42 in. Shear)	Blow Count (48 in. Shear)	Blow Count (54 in. Shear)	Blow Count (60 in. Shear)	Blow Count (66 in. Shear)	Blow Count (72 in. Shear)	Blow Count (78 in. Shear)	Blow Count (84 in. Shear)	Blow Count (90 in. Shear)	Blow Count (96 in. Shear)	Blow Count (102 in. Shear)	Blow Count (108 in. Shear)	Blow Count (114 in. Shear)	Blow Count (120 in. Shear)	Blow Count (126 in. Shear)	Blow Count (132 in. Shear)	Blow Count (138 in. Shear)	Blow Count (144 in. Shear)	Blow Count (150 in. Shear)	Blow Count (156 in. Shear)	Blow Count (162 in. Shear)	Blow Count (168 in. Shear)	Blow Count (174 in. Shear)	Blow Count (180 in. Shear)	Blow Count (186 in. Shear)	Blow Count (192 in. Shear)	Blow Count (198 in. Shear)	Blow Count (204 in. Shear)	Blow Count (210 in. Shear)	Blow Count (216 in. Shear)	Blow Count (222 in. Shear)	Blow Count (228 in. Shear)	Blow Count (234 in. Shear)	Blow Count (240 in. Shear)	Blow Count (246 in. Shear)	Blow Count (252 in. Shear)	Blow Count (258 in. Shear)	Blow Count (264 in. Shear)	Blow Count (270 in. Shear)	Blow Count (276 in. Shear)	Blow Count (282 in. Shear)	Blow Count (288 in. Shear)	Blow Count (294 in. Shear)	Blow Count (300 in. Shear)	Blow Count (306 in. Shear)	Blow Count (312 in. Shear)	Blow Count (318 in. Shear)	Blow Count (324 in. Shear)	Blow Count (330 in. Shear)	Blow Count (336 in. Shear)	Blow Count (342 in. Shear)	Blow Count (348 in. Shear)	Blow Count (354 in. Shear)	Blow Count (360 in. Shear)	Blow Count (366 in. Shear)	Blow Count (372 in. Shear)	Blow Count (378 in. Shear)	Blow Count (384 in. Shear)	Blow Count (390 in. Shear)	Blow Count (396 in. Shear)	Blow Count (402 in. Shear)	Blow Count (408 in. Shear)	Blow Count (414 in. Shear)	Blow Count (420 in. Shear)	Blow Count (426 in. Shear)	Blow Count (432 in. Shear)	Blow Count (438 in. Shear)	Blow Count (444 in. Shear)	Blow Count (450 in. Shear)	Blow Count (456 in. Shear)	Blow Count (462 in. Shear)	Blow Count (468 in. Shear)	Blow Count (474 in. Shear)	Blow Count (480 in. Shear)	Blow Count (486 in. Shear)	Blow Count (492 in. Shear)	Blow Count (498 in. Shear)	Blow Count (504 in. Shear)	Blow Count (510 in. Shear)	Blow Count (516 in. Shear)	Blow Count (522 in. Shear)	Blow Count (528 in. Shear)	Blow Count (534 in. Shear)	Blow Count (540 in. Shear)	Blow Count (546 in. Shear)	Blow Count (552 in. Shear)	Blow Count (558 in. Shear)	Blow Count (564 in. Shear)	Blow Count (570 in. Shear)	Blow Count (576 in. Shear)	Blow Count (582 in. Shear)	Blow Count (588 in. Shear)	Blow Count (594 in. Shear)	Blow Count (600 in. Shear)	Blow Count (606 in. Shear)	Blow Count (612 in. Shear)	Blow Count (618 in. Shear)	Blow Count (624 in. Shear)	Blow Count (630 in. Shear)	Blow Count (636 in. Shear)	Blow Count (642 in. Shear)	Blow Count (648 in. Shear)	Blow Count (654 in. Shear)	Blow Count (660 in. Shear)	Blow Count (666 in. Shear)	Blow Count (672 in. Shear)	Blow Count (678 in. Shear)	Blow Count (684 in. Shear)	Blow Count (690 in. Shear)	Blow Count (696 in. Shear)	Blow Count (702 in. Shear)	Blow Count (708 in. Shear)	Blow Count (714 in. Shear)	Blow Count (720 in. Shear)	Blow Count (726 in. Shear)	Blow Count (732 in. Shear)	Blow Count (738 in. Shear)	Blow Count (744 in. Shear)	Blow Count (750 in. Shear)	Blow Count (756 in. Shear)	Blow Count (762 in. Shear)	Blow Count (768 in. Shear)	Blow Count (774 in. Shear)	Blow Count (780 in. Shear)	Blow Count (786 in. Shear)	Blow Count (792 in. Shear)	Blow Count (798 in. Shear)	Blow Count (804 in. Shear)	Blow Count (810 in. Shear)	Blow Count (816 in. Shear)	Blow Count (822 in. Shear)	Blow Count (828 in. Shear)	Blow Count (834 in. Shear)	Blow Count (840 in. Shear)	Blow Count (846 in. Shear)	Blow Count (852 in. Shear)	Blow Count (858 in. Shear)	Blow Count (864 in. Shear)	Blow Count (870 in. Shear)	Blow Count (876 in. Shear)	Blow Count (882 in. Shear)	Blow Count (888 in. Shear)	Blow Count (894 in. Shear)	Blow Count (900 in. Shear)	Blow Count (906 in. Shear)	Blow Count (912 in. Shear)	Blow Count (918 in. Shear)	Blow Count (924 in. Shear)	Blow Count (930 in. Shear)	Blow Count (936 in. Shear)	Blow Count (942 in. Shear)	Blow Count (948 in. Shear)	Blow Count (954 in. Shear)	Blow Count (960 in. Shear)	Blow Count (966 in. Shear)	Blow Count (972 in. Shear)	Blow Count (978 in. Shear)	Blow Count (984 in. Shear)	Blow Count (990 in. Shear)	Blow Count (996 in. Shear)	Blow Count (1002 in. Shear)	Blow Count (1008 in. Shear)	Blow Count (1014 in. Shear)	Blow Count (1020 in. Shear)	Blow Count (1026 in. Shear)	Blow Count (1032 in. Shear)	Blow Count (1038 in. Shear)	Blow Count (1044 in. Shear)	Blow Count (1050 in. Shear)	Blow Count (1056 in. Shear)	Blow Count (1062 in. Shear)	Blow Count (1068 in. Shear)	Blow Count (1074 in. Shear)	Blow Count (1080 in. Shear)	Blow Count (1086 in. Shear)	Blow Count (1092 in. Shear)	Blow Count (1098 in. Shear)	Blow Count (1104 in. Shear)	Blow Count (1110 in. Shear)	Blow Count (1116 in. Shear)	Blow Count (1122 in. Shear)	Blow Count (1128 in. Shear)	Blow Count (1134 in. Shear)	Blow Count (1140 in. Shear)	Blow Count (1146 in. Shear)	Blow Count (1152 in. Shear)	Blow Count (1158 in. Shear)	Blow Count (1164 in. Shear)	Blow Count (1170 in. Shear)	Blow Count (1176 in. Shear)	Blow Count (1182 in. Shear)	Blow Count (1188 in. Shear)	Blow Count (1194 in. Shear)	Blow Count (1200 in. Shear)	Blow Count (1206 in. Shear)	Blow Count (1212 in. Shear)	Blow Count (1218 in. Shear)	Blow Count (1224 in. Shear)	Blow Count (1230 in. Shear)	Blow Count (1236 in. Shear)	Blow Count (1242 in. Shear)	Blow Count (1248 in. Shear)	Blow Count (1254 in. Shear)	Blow Count (1260 in. Shear)	Blow Count (1266 in. Shear)	Blow Count (1272 in. Shear)	Blow Count (1278 in. Shear)	Blow Count (1284 in. Shear)	Blow Count (1290 in. Shear)	Blow Count (1296 in. Shear)	Blow Count (1302 in. Shear)	Blow Count (1308 in. Shear)	Blow Count (1314 in. Shear)	Blow Count (1320 in. Shear)	Blow Count (1326 in. Shear)	Blow Count (1332 in. Shear)	Blow Count (1338 in. Shear)	Blow Count (1344 in. Shear)	Blow Count (1350 in. Shear)	Blow Count (1356 in. Shear)	Blow Count (1362 in. Shear)	Blow Count (1368 in. Shear)	Blow Count (1374 in. Shear)	Blow Count (1380 in. Shear)	Blow Count (1386 in. Shear)	Blow Count (1392 in. Shear)	Blow Count (1398 in. Shear)	Blow Count (1404 in. Shear)	Blow Count (1410 in. Shear)	Blow Count (1416 in. Shear)	Blow Count (1422 in. Shear)	Blow Count (1428 in. Shear)	Blow Count (1434 in. Shear)	Blow Count (1440 in. Shear)	Blow Count (1446 in. Shear)	Blow Count (1452 in. Shear)	Blow Count (1458 in. Shear)	Blow Count (1464 in. Shear)	Blow Count (1470 in. Shear)	Blow Count (1476 in. Shear)	Blow Count (1482 in. Shear)	Blow Count (1488 in. Shear)	Blow Count (1494 in. Shear)	Blow Count (1500 in. Shear)	Blow Count (1506 in. Shear)	Blow Count (1512 in. Shear)	Blow Count (1518 in. Shear)	Blow Count (1524 in. Shear)	Blow Count (1530 in. Shear)	Blow Count (1536 in. Shear)	Blow Count (1542 in. Shear)	Blow Count (1548 in. Shear)	Blow Count (1554 in. Shear)	Blow Count (1560 in. Shear)	Blow Count (1566 in. Shear)	Blow Count (1572 in. Shear)	Blow Count (1578 in. Shear)	Blow Count (1584 in. Shear)	Blow Count (1590 in. Shear)	Blow Count (1596 in. Shear)	Blow Count (1602 in. Shear)	Blow Count (1608 in. Shear)	Blow Count (1614 in. Shear)	Blow Count (1620 in. Shear)	Blow Count (1626 in. Shear)	Blow Count (1632 in. Shear)	Blow Count (1638 in. Shear)	Blow Count (1644 in. Shear)	Blow Count (1650 in. Shear)	Blow Count (1656 in. Shear)	Blow Count (1662 in. Shear)	Blow Count (1668 in. Shear)	Blow Count (1674 in. Shear)	Blow Count (1680 in. Shear)	Blow Count (1686 in. Shear)	Blow Count (1692 in. Shear)	Blow Count (1698 in. Shear)	Blow Count (1704 in. Shear)	Blow Count (1710 in. Shear)	Blow Count (1716 in. Shear)	Blow Count (1722 in. Shear)	Blow Count (1728 in. Shear)	Blow Count (1734 in. Shear)	Blow Count (1740 in. Shear)	Blow Count (1746 in. Shear)	Blow Count (1752 in. Shear)	Blow Count (1758 in. Shear)	Blow Count (1764 in. Shear)	Blow Count (1770 in. Shear)	Blow Count (1776 in. Shear)	Blow Count (1782 in. Shear)	Blow Count (1788 in. Shear)	Blow Count (1794 in. Shear)	Blow Count (1800 in. Shear)	Blow Count (1806 in. Shear)	Blow Count (1812 in. Shear)	Blow Count (1818 in. Shear)	Blow Count (1824 in. Shear)	Blow Count (1830 in. Shear)	Blow Count (1836 in. Shear)	Blow Count (1842 in. Shear)	Blow Count (1848 in. Shear)	Blow Count (1854 in. Shear)	Blow Count (1860 in. Shear)	Blow Count (1866 in. Shear)	Blow Count (1872 in. Shear)	Blow Count (1878 in. Shear)	Blow Count (1884 in. Shear)	Blow Count (1890 in. Shear)	Blow Count (1896 in. Shear)	Blow Count (1902 in. Shear)	Blow Count (1908 in. Shear)	Blow Count (1914 in. Shear)	Blow Count (1920 in. Shear)	Blow Count (1926 in. Shear)	Blow Count (1932 in. Shear)	Blow Count (1938 in. Shear)	Blow Count (1944 in. Shear)	Blow Count (1950 in. Shear)	Blow Count (1956 in. Shear)	Blow Count (1962 in. Shear)	Blow Count (1968 in. Shear)	Blow Count (1974 in. Shear)	Blow Count (1980 in. Shear)	Blow Count (1986 in. Shear)	Blow Count (1992 in. Shear)	Blow Count (1998 in. Shear)	Blow Count (2004 in. Shear)	Blow Count (2010 in. Shear)	Blow Count (2016 in. Shear)	Blow Count (2022 in. Shear)	Blow Count (2028 in. Shear)	Blow Count (2034 in. Shear)	Blow Count (2040 in. Shear)	Blow Count (2046 in. Shear)	Blow Count (2052 in. Shear)	Blow Count (2058 in. Shear)	Blow Count (2064 in. Shear)	Blow Count (2070 in. Shear)	Blow Count (2076 in. Shear)	Blow Count (2082 in. Shear)	Blow Count (2088 in. Shear)	Blow Count (2094 in. Shear)	Blow Count (2100 in. Shear)	Blow Count (2106 in. Shear)	Blow Count (2112 in. Shear)	Blow Count (2118 in. Shear)	Blow Count (2124 in. Shear)	Blow Count (2130 in. Shear)	Blow Count (2136 in. Shear)	Blow Count (2142 in. Shear)	Blow Count (2148 in. Shear)	Blow Count (2154 in. Shear)	Blow Count (2160 in. Shear)	Blow Count (2166 in. Shear)	Blow Count (2172 in. Shear)	Blow Count (2178 in. Shear)	Blow Count (2184 in. Shear)	Blow Count (2190 in. Shear)	Blow Count (2196 in. Shear)	Blow Count (2202 in. Shear)	Blow Count (2208 in. Shear)	Blow Count (2214 in. Shear)	Blow Count (2220 in. Shear)	Blow Count (2226 in. Shear)	Blow Count (2232 in. Shear)	Blow Count (2238 in. Shear)	Blow Count (2244 in. Shear)	Blow Count (2250 in. Shear)	Blow Count (2256 in. Shear)	Blow Count (2262 in. Shear)	Blow Count (2268 in. Shear)	Blow Count (2274 in. Shear)	Blow Count (2280 in. Shear)	Blow Count (2286 in. Shear)	Blow Count (2292 in. Shear)	Blow Count (2298 in. Shear)	Blow Count (2304 in. Shear)	Blow Count (2310 in. Shear)	Blow Count (2316 in. Shear)	Blow Count (2322 in. Shear)	Blow Count (2328 in. Shear)	Blow Count (2334 in. Shear)	Blow Count (2340 in. Shear)	Blow Count (2346 in. Shear)	Blow Count (2352 in. Shear)	Blow Count (2358 in. Shear)	Blow Count (2364 in. Shear)	Blow Count (2370 in. Shear)	Blow Count (2376 in. Shear)	Blow Count (2382 in. Shear)	Blow Count (2388 in. Shear)	Blow Count (2394 in. Shear)	Blow Count (2400 in. Shear)	Blow Count (2406 in. Shear)	Blow Count (2412 in. Shear)	Blow Count (2418 in. Shear)	Blow Count (2424 in. Shear)	Blow Count (2430 in. Shear)	Blow Count (2436 in. Shear)	Blow Count (2442 in. Shear)	Blow Count (2448 in. Shear)	Blow Count (2454 in. Shear)	Blow Count (2460 in. Shear)	Blow Count (2466 in. Shear)	Blow Count (2472 in. Shear)	Blow Count (2478 in. Shear)	Blow Count (2484 in. Shear)	Blow Count (2490 in. Shear)	Blow Count (2496 in. Shear)	Blow Count (2502 in. Shear)	Blow Count (2508 in. Shear)	Blow Count (2514 in. Shear)	Blow Count (2520 in. Shear)	Blow Count (2526 in. Shear)	Blow Count (2532 in. Shear)	Blow Count (2538 in. Shear)	Blow Count (2544 in. Shear)	Blow Count (2550 in. Shear)	Blow Count (2556 in. Shear)	Blow Count (2562 in. Shear)	Blow Count (2568 in. Shear)	Blow Count (2574 in. Shear)	Blow Count (2580 in. Shear)	Blow Count (2586 in. Shear)	Blow Count (2592 in. Shear)	Blow Count (2598 in. Shear)	Blow Count (2604 in. Shear)	Blow Count (2610 in. Shear)	Blow Count (2616 in. Shear)	Blow Count (2622 in. Shear)	Blow Count (2628 in. Shear)	Blow Count (2634 in. Shear)	Blow Count (2640 in. Shear)	Blow Count (2646 in. Shear)	Blow Count (2652 in. Shear)	Blow Count (2658 in. Shear)	Blow Count (2664 in. Shear)	Blow Count (2670 in. Shear)	Blow Count (2676 in. Shear)	Blow Count (2682 in. Shear)	Blow Count (2688 in. Shear)	Blow Count (2694 in. Shear)	Blow Count (2700 in. Shear)	Blow Count (2706 in. Shear)	Blow Count (2712 in. Shear)	Blow Count (2718 in. Shear)	Blow Count (2724 in. Shear)	Blow Count (2730 in. Shear)	Blow Count (2736 in. Shear)	Blow Count (2742 in. Shear)	Blow Count (2748 in. Shear)	Blow Count (2754 in. Shear)	Blow Count (2760 in. Shear)	Blow Count (2766 in. Shear)	Blow Count (2772 in. Shear)	Blow Count (2778 in. Shear)	Blow Count (2784 in. Shear)	Blow Count (2790 in. Shear)	Blow Count (2796 in. Shear)	Blow Count (2802 in. Shear)	Blow Count (2808 in. Shear)	Blow Count (2814 in. Shear)	Blow Count (2820 in. Shear)	Blow Count (2826 in. Shear)	Blow Count (2832 in. Shear)	Blow Count (2838 in. Shear)	Blow Count (2844 in. Shear)	Blow Count (2850 in. Shear)	Blow Count (2856 in. Shear)	Blow Count (2862 in. Shear)	Blow Count (2868 in. Shear)	Blow Count (2874 in. Shear)	Blow Count (2880 in. Shear)	Blow Count (2886 in. Shear)	Blow Count (2892 in. Shear)	Blow Count (2898 in. Shear)	Blow Count (2904 in. Shear)	Blow Count (2910 in. Shear)	Blow Count (2916 in. Shear)	Blow Count (2922 in. Shear)	Blow Count (2928 in. Shear)	Blow Count (2934 in. Shear)	Blow Count (2940 in. Shear)	Blow Count (2946 in. Shear)	Blow Count (2952 in. Shear)	Blow Count (2958 in. Shear)	Blow Count (2964 in. Shear)	Blow Count (2970 in. Shear)	Blow Count (2976 in. Shear)	Blow Count (2982 in. Shear)	Blow Count (2988 in. Shear)	Blow Count (2994 in. Shear)	Blow Count (3000 in. Shear)	Blow Count (3006 in. Shear)	Blow Count (3012 in. Shear)	Blow Count (3018 in. Shear)	Blow Count (3024 in. Shear)	Blow Count (3030 in. Shear)	Blow Count (3036 in. Shear)	Blow Count (3042 in. Shear)	Blow Count (3048 in. Shear)	Blow Count (3054 in. Shear)	Blow Count (3060 in. Shear)	Blow Count (3066 in. Shear)	Blow Count (3072 in. Shear)	Blow Count (3078 in. Shear)	Blow Count (3084 in. Shear)	Blow Count (3090 in. Shear)	Blow Count (3096 in. Shear)	Blow Count (3102 in. Shear)	Blow Count (3108 in. Shear)	Blow Count (3114 in. Shear)	Blow Count (3120 in. Shear)	Blow Count (3126 in. Shear)	Blow Count (3132 in. Shear)	Blow Count (3138 in. Shear)	Blow Count (3144 in. Shear)	Blow Count (3150 in. Shear)	Blow Count (3156 in. Shear)	Blow Count (3162 in. Shear)	Blow Count (3168 in. Shear)	Blow Count (3174 in. Shear)	Blow Count (3180 in. Shear)	Blow Count (3186 in. Shear)	Blow Count (3192 in. Shear)	Blow Count (3198 in. Shear)	Blow Count (3204 in. Shear)	Blow Count (3210 in. Shear)	Blow Count (3216 in. Shear)	Blow Count (3222 in. Shear)	Blow Count (3228 in. Shear)	Blow Count (3234 in. Shear)	Blow Count (3240 in. Shear)	Blow Count (3246 in. Shear)	Blow Count (3252 in. Shear)	Blow Count (3258 in. Shear)	Blow Count (3264 in. Shear)	Blow Count (3270 in. Shear)	Blow Count (3276 in. Shear)	Blow Count (3282 in. Shear)	Blow Count (3288 in. Shear)	Blow Count (3294 in. Shear)	Blow Count (3300 in. Shear)	Blow Count (3306 in. Shear)	Blow Count (3312 in. Shear)	Blow Count (3318 in. Shear)	Blow Count (3324 in. Shear)	Blow Count (3330 in. Shear)	Blow Count (3336 in. Shear)	Blow Count (3342 in. Shear)	Blow Count (3348 in. Shear)	Blow Count (3354 in. Shear)	Blow Count (3360 in. Shear)	Blow Count (3366 in. Shear)	Blow Count (3372 in. Shear)	Blow Count (3378 in. Shear)	Blow Count (3384 in. Shear)	Blow Count (3390 in. Shear)	Blow Count (3396 in. Shear)	Blow Count (3402 in. Shear)	Blow Count (3408 in. Shear)	Blow Count (3414 in. Shear)	Blow Count (3420 in. Shear)	Blow Count (3426 in. Shear)	Blow Count (3432 in. Shear)	Blow Count (3438 in. Shear)	Blow Count (3444 in. Shear)	Blow Count (3450 in. Shear)	Blow Count (3456 in. Shear)	Blow Count (3462 in. Shear)	Blow Count (3468 in. Shear)	Blow Count (3474 in. Shear)	Blow Count (3480 in. Shear)	Blow Count (3486 in. Shear)	Blow Count (3492 in. Shear)	Blow Count (3498 in. Shear)	Blow Count (3504 in. Shear)	Blow Count (3510 in. Shear)	Blow Count (3516 in. Shear)	Blow Count (3522 in. Shear)	Blow Count (3528 in. Shear)	Blow Count (3534 in. Shear)	Blow Count (3540 in. Shear)	Blow Count (3546 in. Shear)	Blow Count (3552 in. Shear)	Blow Count (3558 in. Shear)	Blow Count (3564 in. Shear)	Blow Count (3570 in. Shear)	Blow Count (3576 in. Shear)	Blow Count (3582 in. Shear)	Blow Count (3588 in. Shear)	Blow Count (3594 in. Shear)	Blow Count (3600 in. Shear)	Blow Count (3606 in. Shear)	Blow Count (3612 in. Shear)	Blow Count (3618 in. Shear)	Blow Count (3624 in. Shear)	Blow Count (3630 in. Shear)	Blow Count (3636 in. Shear)	Blow Count (3642 in. Shear)	Blow Count (3648 in. Shear)	Blow Count (3654 in. Shear)	Blow Count (3660 in. Shear)	Blow Count (3666 in. Shear)	Blow Count (3672 in. Shear)	Blow Count (3678 in. Shear)	Blow Count (3684 in. Shear)	Blow Count (3690 in. Shear)	Blow Count (3696 in. Shear)	Blow Count (3702 in. Shear)	Blow Count (3708 in. Shear)	Blow Count (3714 in. Shear)	Blow Count (3720 in. Shear)	Blow Count (3726 in. Shear)	Blow Count (3732 in. Shear)	Blow Count (3738 in. Shear)	Blow Count (3744 in. Shear)	Blow Count (3750 in. Shear)	Blow Count (3756 in. Shear)	Blow Count (3762 in. Shear)	Blow Count (3768 in. Shear)	Blow Count (3774 in. Shear)	Blow Count (3780 in. Shear)	Blow Count (3786 in. Shear)	Blow Count (3792 in. Shear)	Blow Count (3798 in. Shear)	Blow Count (3804 in. Shear)	Blow Count (3810 in. Shear)	Blow Count (3816 in. Shear)	Blow Count (3822 in. Shear)	Blow Count (3828 in. Shear)	Blow Count (3834 in. Shear)	Blow Count (3840 in. Shear)	Blow Count (3846 in. Shear)	Blow Count (3852 in. Shear)	Blow Count (3858 in. Shear)	Blow Count (3864 in. Shear)	Blow Count (3870 in. Shear)	Blow Count (3876 in. Shear)	Blow Count (3882 in. Shear)	Blow Count (3888 in. Shear)	Blow Count (3894 in. Shear)	Blow Count (3900 in. Shear)	Blow Count (3906 in. Shear)	Blow Count (3912 in. Shear)	Blow Count (3918 in. Shear)	Blow Count (3924 in. Shear)	Blow Count (3930 in. Shear)	Blow Count (3936 in. Shear)	Blow Count (3942 in. Shear)	Blow Count (3948 in. Shear)	Blow Count (3954 in. Shear)	Blow Count (3960 in. Shear)	Blow Count (3966 in. Shear)	Blow Count (3972 in. Shear)	Blow Count (3978 in. Shear)	Blow Count (3984 in. Shear)	Blow Count (3990 in. Shear)	Blow Count (3996 in. Shear)	Blow Count (4002 in. Shear)	Blow Count (4008 in. Shear)	Blow Count (4014 in. Shear)	Blow Count (4020 in. Shear)	Blow Count (4026 in. Shear)	Blow Count (4032 in. Shear)	Blow Count (4038 in. Shear)	Blow Count (4044 in. Shear)	Blow Count (4050 in. Shear)	Blow Count (4056 in. Shear)	Blow Count (4062 in. Shear)	Blow Count (4068 in. Shear)	Blow Count (4074 in. Shear)	Blow Count (4080 in. Shear)	Blow Count (4086 in. Shear)	Blow Count (4092 in. Shear)	Blow Count (4098 in. Shear)	Blow Count (4104 in. Shear)	Blow Count (4110 in. Shear)	Blow Count (4116 in. Shear)	Blow Count (4

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.	Descriptive Term		Portion of Total (%)	
		(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	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).			
		(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	Density of Cohesionless Soils			
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.	Density of Cohesionless Soils		Standard Penetration Resistance N-Value (blows per foot)	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Very loose	0 - 4		
				Loose	5 - 10			
				Medium Dense	11 - 30			
				Dense	31 - 50			
	SILTS AND CLAYS (liquid limit greater than 50)	OL	Organic silts and organic silty clays of low plasticity.	Very Dense	> 50			
		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	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.				
		CH	Inorganic clays of high plasticity, fat clays.					
		OH	Organic clays of medium to high plasticity, organic silts.	Approximate Undrained Shear Strength (psf)				
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	Consistency of Cohesive soils		Field Guidelines			
			Very Soft	SPT N-Value (blows per foot)				
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					Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *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 Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information								

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-101 WIN: 21884.00					
Driller: MaineDOT				Elevation (ft.): 474.6				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: see Remarks				Drilling Method: Cased Wash Boring				Core Barrel: N/A					
Boring Location: 12+92.4, 11.3 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 18.0 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 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	474.1		6.5" HMA.	0.5	G#304385 A-2-4, SM WC=5.1% G#304386 A-1-a, GW-GM WC=11.5% G#304387 A-1-a, GM WC=9.0%	
5	1D	24/14	5.00 - 7.00	10/11/9/13	20	28				Brown, moist, medium dense, Gravelly fine to coarse SAND, trace silt, occasional cobble, (Fill).			
10	2D	24/14	10.00 - 12.00	22/25/25/21	50	71	29			Brown, moist, very dense, SAND, some gravel, some silt.			
							57						
							64						
							48						
							44						
15	3D	24/10	15.00 - 17.00	5/5/4/4	9	13	15			Brown, wet, medium dense, Sandy GRAVEL, trace silt.			
							21						
							25						
							21						
							46						
20	4D	14.4/10	20.00 - 21.20	7/10/30(2.4")	---		10	455.6		Grey-brown, wet, very dense, Sandy GRAVEL, little silt, (Glacial Till).			
							67			Cobble from 21.2-21.5 ft bgs.			
							76			Roller Coned ahead to 23.0 ft bgs.			
							132						
25							20						
Remarks: -5/16/2017; 07:30-15:30 -5/17/2017; 07:00-10:00													
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 4 Boring No.: BB-GITB-101			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-101 WIN: 21884.00					
Driller: MaineDOT				Elevation (ft.): 474.6				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: see Remarks				Drilling Method: Cased Wash Boring				Core Barrel: N/A					
Boring Location: 12+92.4, 11.3 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 18.0 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 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 _u = 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						
25	5D	24/18	25.00 - 27.00	28/37/34/43	71	101	13		Grey, wet, very dense, SAND, some gravel, some silt, trace clay, (Glacial Till). Roller Coned ahead to 27.0 ft bgs.	G#304388 A-2-4, SC-SM WC=6.4%			
							53						
							123						
							209						
30	6D	12/9	29.00 - 30.00	41/53	---		OPEN HOLE					Similar to above. Roller Coned ahead to 35.0 ft bgs.	
35	7D	12/12	35.00 - 36.00	43/68	---				Similar to above.				
40	8D	24/19	40.00 - 42.00	43/26/32/53	58	83			Grey, wet, hard, fine to coarse Sandy SILT, some gravel, (Glacial Till).				
45	9D	18/12	45.00 - 46.50	49/45/59	104	148			Similar to above.				
50													

Remarks:
 -5/16/2017; 07:30-15:30
 -5/17/2017; 07:00-10:00

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.

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Boring No.: BB-GITB-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-101 WIN: 21884.00																																																																																																																																																																																																																											
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<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</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>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><th>Elevation (ft.)</th></tr><tr><td rowspan="4">50</td><td>10D</td><td>12/12</td><td>50.00 - 51.00</td><td>48/60</td><td>---</td><td></td><td></td><td></td><td rowspan="16"></td><td>Similar to above.</td><td rowspan="16">G#304389 A-2-4, SC-SM WC=9.0%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">55</td><td>11D</td><td>12/12</td><td>55.00 - 56.00</td><td>41/58</td><td>---</td><td></td><td></td><td></td><td>Grey, wet, very dense, SAND, some silt, little gravel, trace clay, (Glacial Till).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">60</td><td>12D</td><td>24/20</td><td>60.00 - 62.00</td><td>29/37/41/56</td><td>78</td><td>111</td><td></td><td></td><td>Similar to above.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">65</td><td>13D</td><td>12/12</td><td>65.00 - 66.00</td><td>43/57</td><td>---</td><td></td><td></td><td></td><td>Similar to above.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">70</td><td>14D</td><td>12/12</td><td>70.00 - 71.00</td><td>43/53</td><td>---</td><td></td><td></td><td></td><td>Similar to above.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>75</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												Depth (ft.)	Sample Information								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	Elevation (ft.)	50	10D	12/12	50.00 - 51.00	48/60	---					Similar to above.	G#304389 A-2-4, SC-SM WC=9.0%																									55	11D	12/12	55.00 - 56.00	41/58	---				Grey, wet, very dense, SAND, some silt, little gravel, trace clay, (Glacial Till).																												60	12D	24/20	60.00 - 62.00	29/37/41/56	78	111			Similar to above.																												65	13D	12/12	65.00 - 66.00	43/57	---				Similar to above.																												70	14D	12/12	70.00 - 71.00	43/53	---				Similar to above.																												75											
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Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-101 WIN: 21884.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
Driller: MaineDOT				Elevation (ft.) 474.6				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: see Remarks				Drilling Method: Cased Wash Boring				Core Barrel: N/A																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
Boring Location: 12+92.4, 11.3 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 18.0 ft bgs.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
Hammer Efficiency Factor: 0.854				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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90.50</td><td>55</td><td>---</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>95</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>100</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												Depth (ft.)	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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook Location: Grand Isle, Maine				Boring No.: BB-GITB-102 WIN: 21884.00				
Driller: MaineDOT				Elevation (ft.): 474.5				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: 5/17/2017; 10:00-13:30				Drilling Method: Cased Wash Boring				Core Barrel: N/A				
Boring Location: 13+07.7, 11.7 ft Rt.				Casing ID/OD: NW-3"				Water Level*: 11.0 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 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.
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0							SSA	474.0		6" HMA.	0.5'	G#304390 A-1-b, SM WC=3.9%
5	1D	24/19	5.00 - 7.00	12/16/12/10	28	40				Brown, damp, dense, SAND, some gravel, little silt, occasional cobble, (Fill).		
10										Similar to above, except medium dense, (Fill).		
15	3D	24/14	15.00 - 17.00	5/8/12/12	20	28	6			Brown, wet, medium dense, Sandy GRAVEL, little silt, (Fill).	19.0'	G#304391 A-1-a, GM WC=12.1%
20	4D	24/9	20.00 - 22.00	19/21/15/14	36	51	29	455.5	Grey-brown, moist, very dense, fine to coarse SAND, some gravel, little silt, (Glacial Till). Washed ahead to 22.0 ftb gs.		G#304392 A-2-4, SM WC=7.6%	
	5D	24/15	22.00 - 24.00	12/19/29/28	48	68	53		Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).			
25							172					
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 2 Boring No.: BB-GITB-102		

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:

Soucy Bridge #2781 carries US Route 1 over Thibodeau Brook

Location:

Grand Isle, Maine

Boring No.:

BB-GITB-102

WIN:

21884.00

Driller:

MaineDOT

Elevation (ft.):

474.5

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:

5/17/2017; 10:00-13:30

Drilling Method:

Cased Wash Boring

Core Barrel:

N/A

Boring Location:

13+07.7, 11.7 ft Rt.

Casing ID/OD:

NW-3"

Water Level*:

11.0 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

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_u = 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

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				
25	6D	12/6	25.00 - 26.00	42/75	---		OPEN HOLE			Grey, wet, very dense, Gravelly fine to coarse SAND, some silt, (Glacial Till).	
										Drill action indicates cobbles.	
30	7D	18/12	30.00 - 31.50	29/44/63	107	152				Similar to above.	
40	8D	12/8	40.00 - 41.00	443/60	---					Similar to above.	
								433.5		Bottom of Exploration at 41.0 feet below ground surface. NO REFUSAL	41.0
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 2 of 2

Boring No.: BB-GITB-102

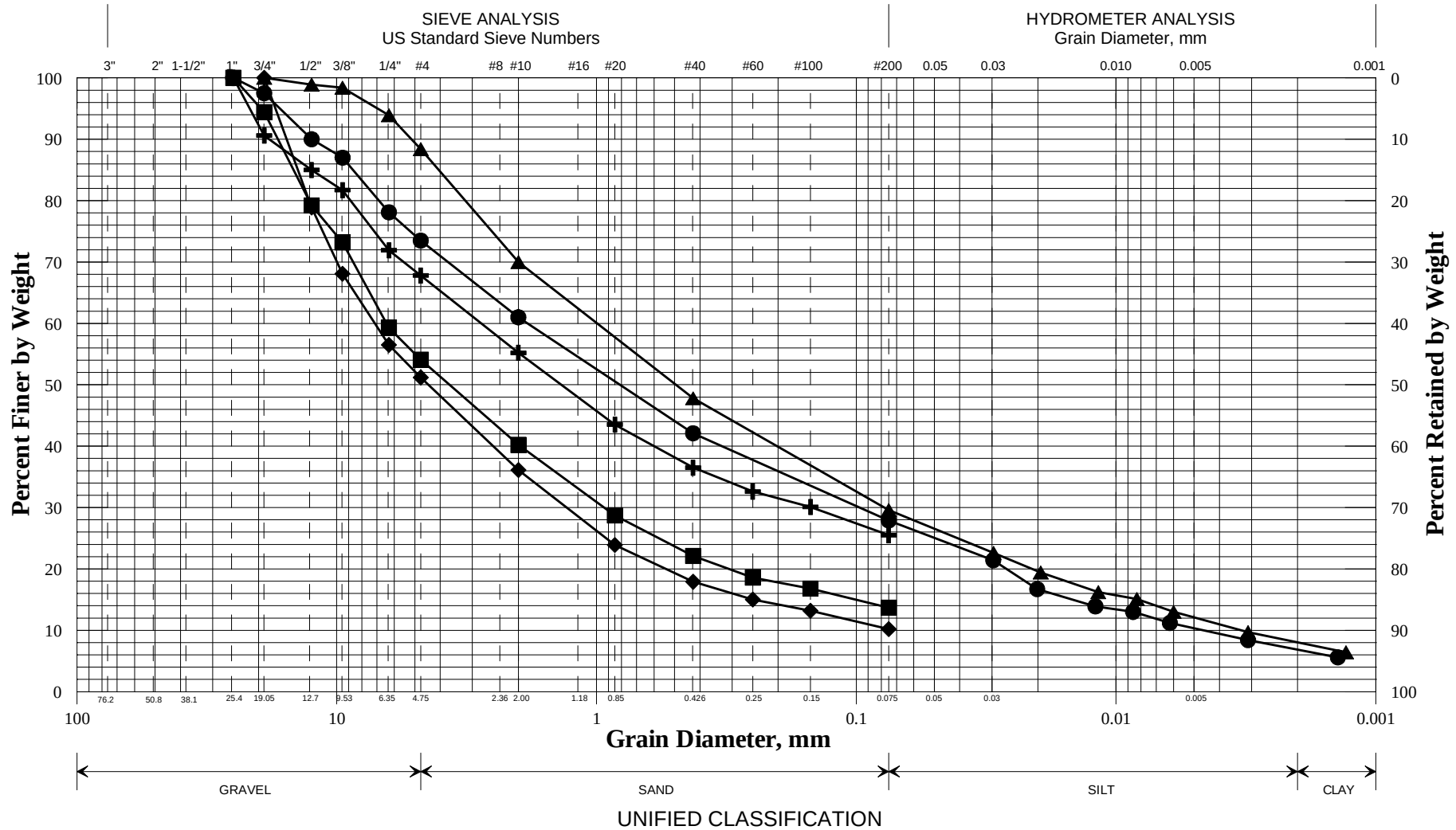
Appendix B

Laboratory Test Results

Work Number: 21884.00

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

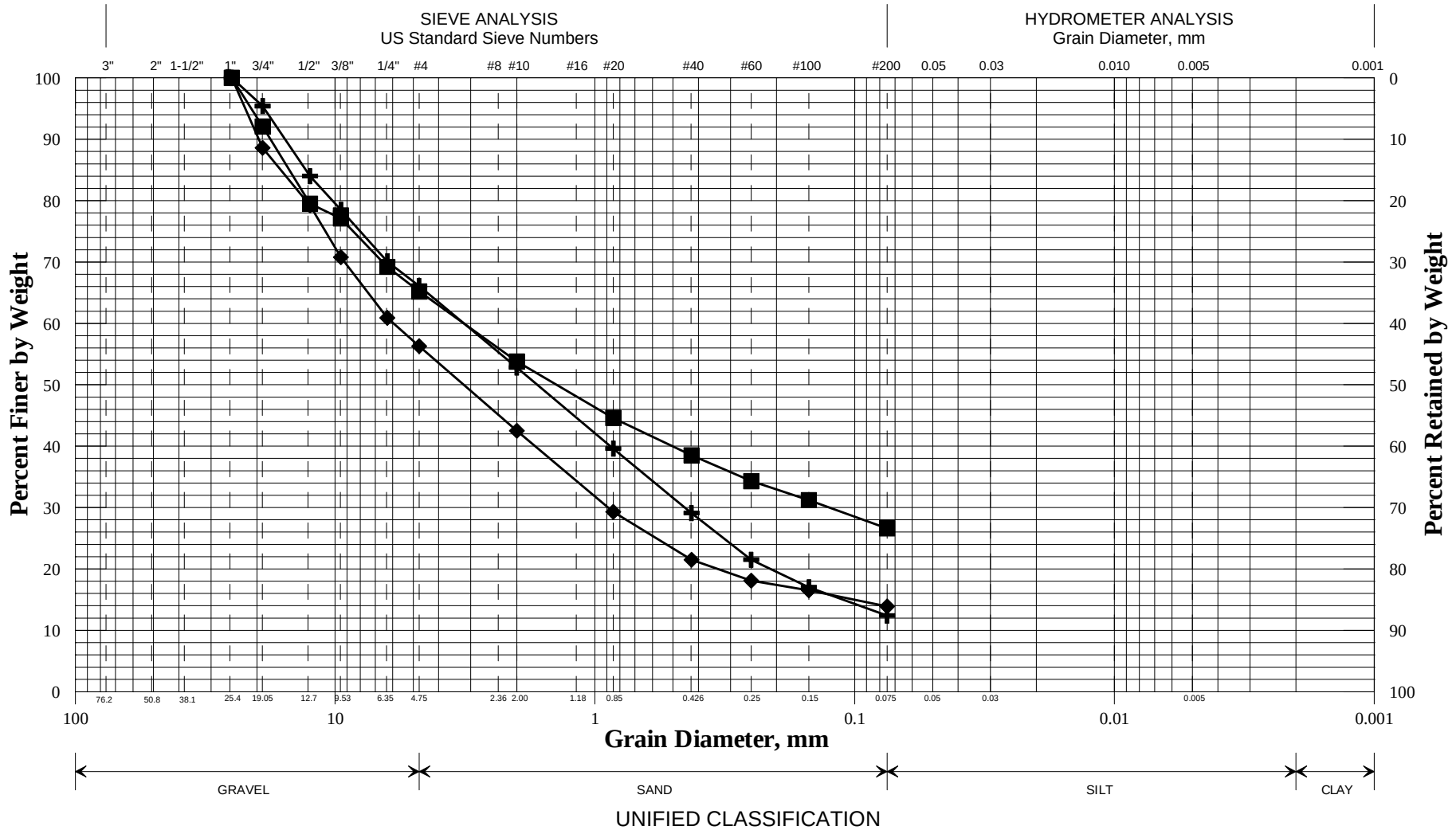
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-GITB-101/2D	12+92.4	11.3 LT	10.0-12.0	SAND, some gravel, some silt.	5.1			
◆	BB-GITB-101/3D	12+92.4	11.3 LT	15.0-17.0	Sandy GRAVEL, trace silt.	11.5			
■	BB-GITB-101/4D	12+92.4	11.3 LT	20.0-21.2	Sandy GRAVEL, little silt.	9.0			
●	BB-GITB-101/5D	12+92.4	11.3 LT	25.0-27.0	SAND, some gravel, some silt, trace clay.	6.4			
▲	BB-GITB-101/11D	12+92.4	11.3 LT	55.0-56.0	SAND, some silt, little gravel, trace clay.	9.0			
×									

WIN
021884.00
Town
Grand Isle
Reported by/Date
WHITE, TERRY A 6/22/2017

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-GITB-102/1D	13+07.7	11.7 RT	5.0-7.0	SAND, some gravel, little silt.	3.9			
◆	BB-GITB-102/3D	13+07.7	11.7 RT	15.0-17.0	Sandy GRAVEL, little silt.	12.1			
■	BB-GITB-102/5D	13+07.7	11.7 RT	22.0-24.0	SAND, some gravel, some silt.	7.6			
●									
▲									
×									

WIN	
021884.00	
Town	
Grand Isle	
Reported by/Date	
WHITE, TERRY A	6/22/2017

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 $\gamma := 125 \cdot \text{pcf}$

Internal friction angle $\phi := 32 \cdot \text{deg}$

Cohesion $c := 0 \cdot \text{psf}$

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:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

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 :

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

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:

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

Soil type	Coefficient of Lateral Earth Pressure			
	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.

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 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

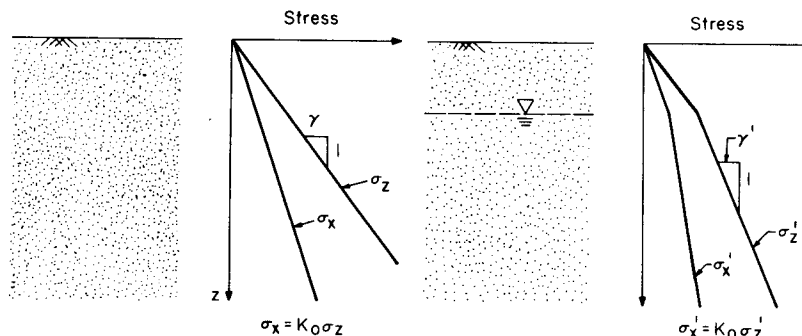


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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 $\beta = 0^\circ$, 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:

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

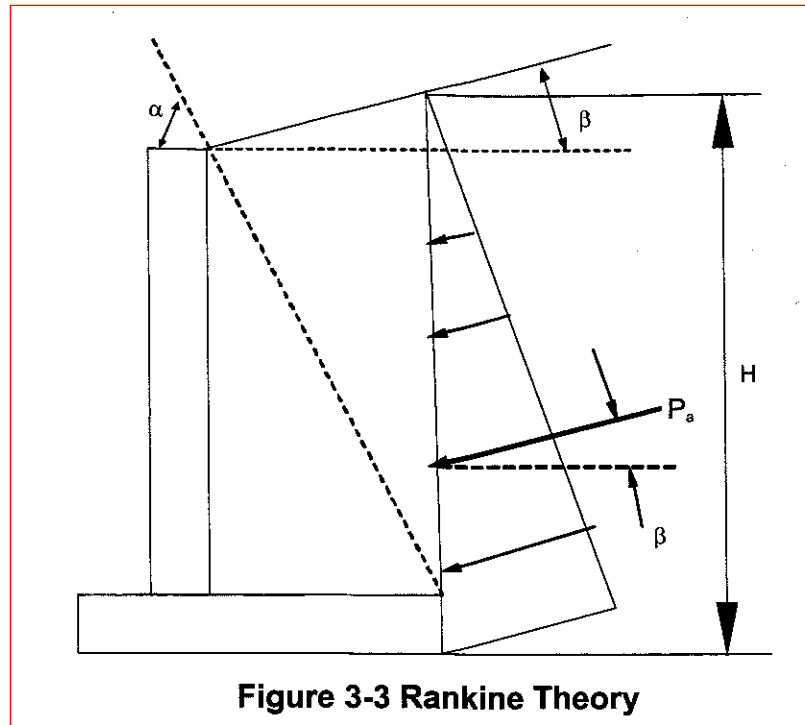


Figure 3-3 Rankine Theory

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 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 considered.
3. The proposed bearing elevation ranges from approximately 453 to 449.5 feet.
4. Proposed finish roadway grade elevation is approximately the same as existing.
5. Proposed precast concrete box base is 24 feet wide.
6. Proposed precast concrete box is 112 feet long, which ignores the 14 and 18 foot long sections with slope tapered walls.
7. 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 25 bpf for the consistency of compacted Granular Borrow.
8. 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
Coarse to medium sand, and with little gravel (SW, SP)	Medium dense to dense	4-8	6

Recommend 6 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

$$B := 24\text{ft}$$

$$D_f := 1.0\cdot\text{ft}$$

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

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

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

Foundation soils:

$$\gamma_{1d} := 125 \cdot \text{pcf}$$

MaineDOT Soil Type 4 - Granular Borrow

$$w_{\text{sat}} := .10$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Dry, dense angular sand - natural moisture
content in a saturated state

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

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

$$\gamma_{1\text{sat}} = 137.5 \cdot \text{pcf}$$

$$N_{\text{design}} := 25$$

$$\phi := 34 \cdot \text{deg}$$

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 $\phi = 34$ degrees.

$$N_c := 42.14$$

$$N_q := 29.4$$

$$N_{\gamma} := 31.1$$

Nominal Bearing Resistance per Terzaghi equation

$$q := D_f \cdot (\gamma_{1\text{sat}} - \gamma_w) \quad q = 0.075 \cdot \text{ksf}$$

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

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

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

$$q_n = 30.2 \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.45$$

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

$$q_r = 13.6 \cdot \text{ksf}$$

for

$$B = 24 \cdot \text{ft}$$

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)

$$N_c := 42.2$$

$$N_q := 29.4$$

$$N_\gamma := 41.1$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$L := 112 \cdot \text{ft} \quad B := 24 \cdot \text{ft}$$

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

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

$$s_\gamma = 0.91$$

$$s_q = 1.14$$

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.

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

Load Inclination factors

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

$$i_c := 1.0$$

$$i_\gamma := 1.0$$

$$i_q := 1.0$$

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

$$\frac{D_f}{B} = 0.042$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 42.2$$

$$N_{\gamma m} = 37.577$$

$$N_{qm} = 33.649$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 33.1 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 14.9 \cdot \text{ksf}$$

Recommend 15 ksf for the factored bearing resistance at the strength limit state for precast concrete box culverts 24 ft wide or greater on compacted Granular Borrow.

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.

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

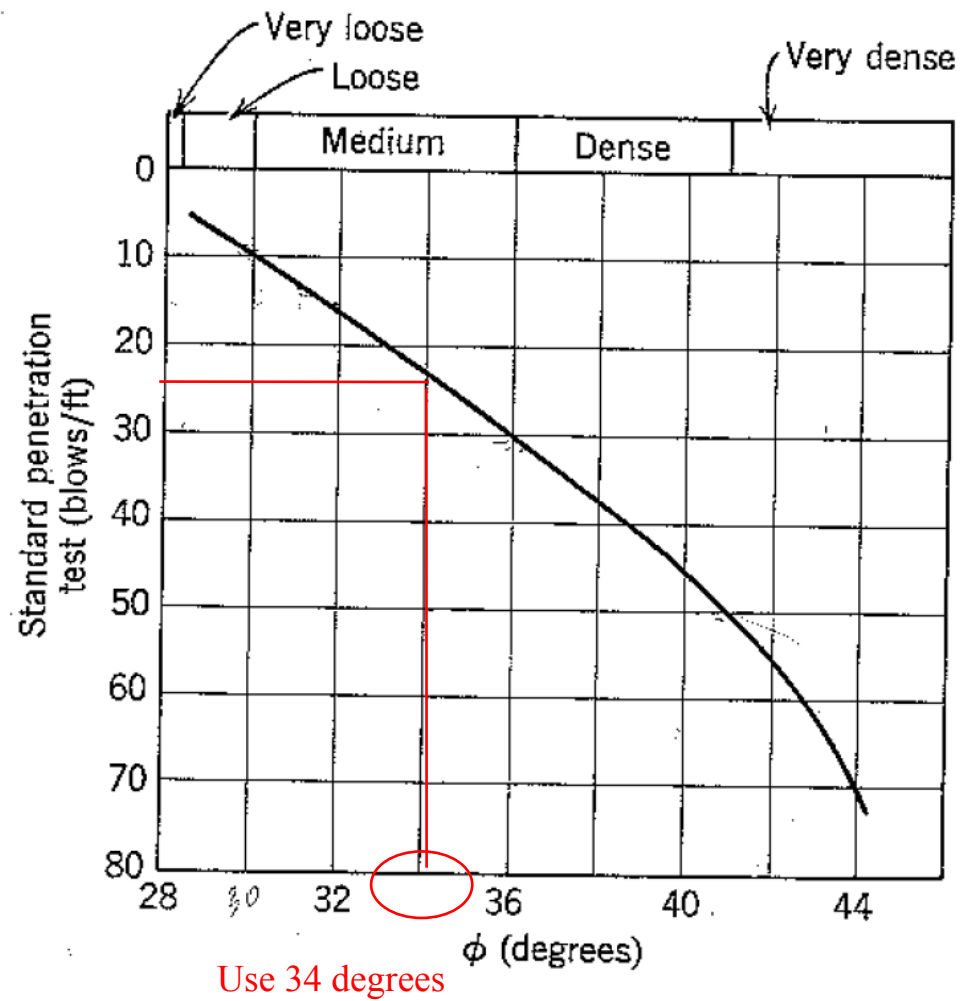


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

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 $i_i < 1$.

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 interfacing. 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.

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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.

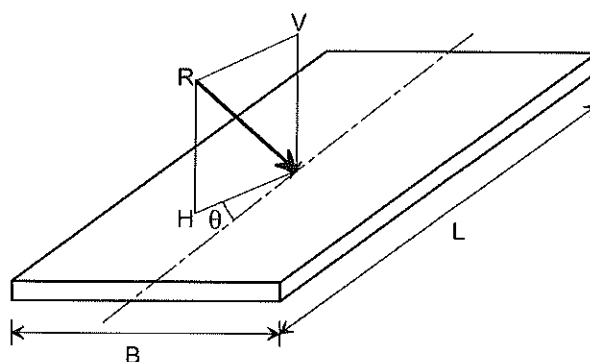


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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: **Grand isle, Maine**

DFI = 1500 degree-days.

Case 1 - coarse grained granular fill soils W=10% (assumed).

For DFI = 2600

$d := 109.1 \cdot \text{in}$

Depth of Frost Penetration

$d := 109.1 \text{ in}$

$d = 9.1 \cdot \text{ft}$

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: Fort Kent, Maine

Air Design Freezing Index = 2837 F-days

N-Factor = 0.80

Surface Design Freezing Index = 2270 F-days

Mean Annual Temperature = 37.8 deg F

Design Length of Freezing Season = 164 days

Layer

#:Type	t	w%	d	Cf	Cu	Kf	Ku	L
--------	---	----	---	----	----	----	----	---

1-Coarse	115.5	10.0	125.0	28	34	2.0	1.6	1,800
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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).

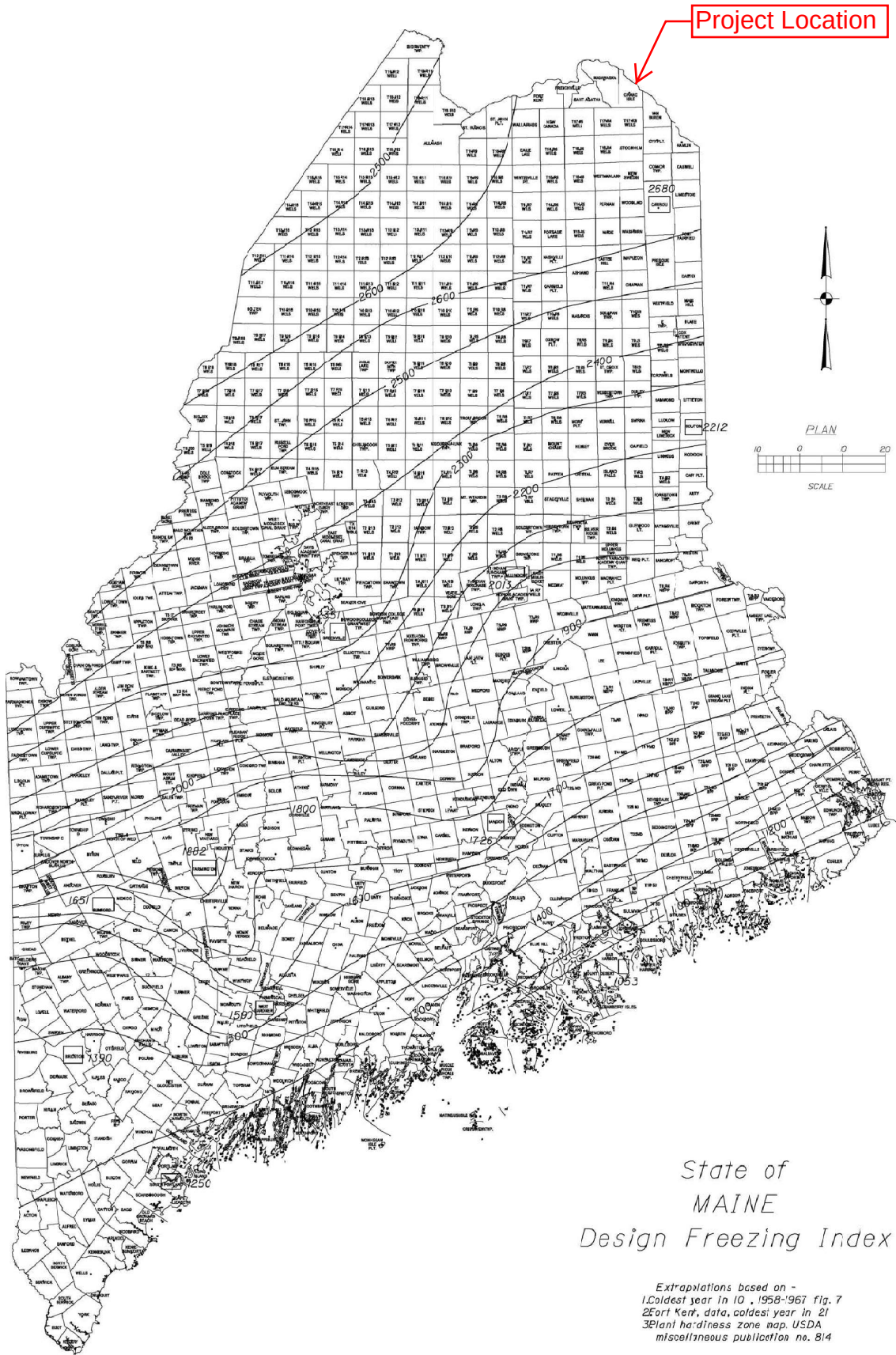
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

L = Latent heat of fusion, in BTU / cubic ft.

Total Depth of Frost Penetration = 9.63 ft = 115.5 in.

Recommendation: 9.1 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