

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

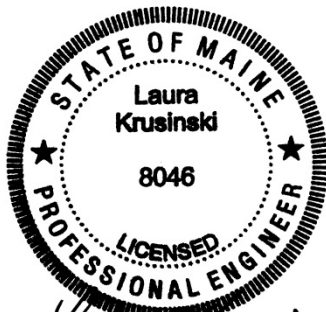
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**OTTER BROOK BRIDGE
U.S. ROUTE 1 OVER OTTER BROOK
CARIBOU, MAINE**

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Aroostook County
WIN 22228.00

Soils Report 2019-30
Bridge No. 5902

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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 Otter Brook Bridge which carries U.S. Route 1 over Otter Brook in Caribou, 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 corrugated steel pipe arch constructed in 1960. The culvert is approximately 80 feet long and 10.7 feet wide. The existing culvert replaced a stream crossing which was 250 feet south of the existing culvert when Otter Brook east of Route 1 was diverted. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the existing culvert is in overall poor condition (rating of 4) with deep pitting and rust holes.

The proposed replacement structure is a 22-foot span and 8-foot rise, 90-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 the calculated scour depth. The upstream and downstream ends of the culvert will be slope-tapered to match the 1.75H:1V (horizontal:vertical) sideslopes. The box culvert invert will be embedded 2 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill bearing on compacted native soils.

The new box culvert will be located on nearly the same horizontal alignment as the existing bridge. The finished grade over the proposed precast box culvert will closely match the existing. A contractor-designed one lane temporary detour will relocate State Route 1 upstream of the existing stream crossing to allow for construction of the new box culvert.

2.0 GEOLOGIC SETTING

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

The Maine Geological Survey (MGS) Reconnaissance Surficial Geologic Map of the Caribou Quadrangle, Open-File No. 86-59 (1986), indicates the surficial soils near the bridge project consist of swamp and marsh deposits. Swamp and marsh deposits are generally comprised of peat, silt, clay, and sand. The deposits are generally form flat landforms formed by an accumulation of sediments and organic material in depressions and other poorly drained areas.

The MGS Preliminary Aquifer Boundaries Map of the New Sweden Quadrangle, Open-File No. 02-122 (2002), indicates the bridge project is near a boundary of surficial deposits with significant saturated thickness where potential ground-water yield is moderate to excellent. These deposits primarily consist of glacial sand and gravel, but may include areas of sandy till and alluvium.

According to the MGS Geologic Map of the Caribou and Northern Presque Isle Quadrangles, Open File No. 87-2 (1987), bedrock at the project site is mapped as thinly bedded argillaceous micrite and calcareous phyllite of the Carys Mills Formation Upper Member.

3.0 SUBSURFACE INVESTIGATION

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

The test borings were drilled on May 24 and 25, 2018. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and open hole wash boring techniques. Boring BB-COB-101 was completed by backfilling and compacting the borehole with drilling spoils. Boring BB-COB-102 was completed by sealing the borehole with a bentonite-slurry mixture prior to backfilling and compacting the borehole with drilling spoils. 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 drill rig that performed the borings was 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 2018. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.918 to the raw field N-values. This hammer efficiency factor (0.918) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

A geotechnical engineer logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified field-testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of four standard grain size analyses with natural water content and four 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 3 – 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 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A fill unit was encountered in the test borings. The thickness was approximately 9.6 to 9.7 feet. The unit generally consisted of brown, damp to moist, sand, some to little gravel, some to trace silt, trace organics.

Corrected SPT N-values in the fill unit ranged from 11 to 87 blows per foot (bpf), indicating the fill unit is medium dense to very dense in consistency. Two grain size analyses of the fill material resulted in the material being classified as A-1-b under the AASHTO Soil Classification System and SW-SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 6 to 7 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill layer in the borings. Boring BB-COB-101 terminated within the glacial till layer at approximately 25 feet below the roadway surface (brs). Boring BB-COB-102 terminated within the glacial till layer at approximately 32 feet brs. The deposit generally consisted of:

- Brown, wet, sandy gravel, little silt;
- Grey, wet, gravelly sand, little silt;
- Grey, wet, gravel, some clay, some silt, little sand;
- Grey, wet, sandy gravel, little silt, trace clay;
- Grey, wet, sand, some gravel, little silt, trace clay;
- Grey, wet, clay, some silt, some gravel, trace sand; and
- Brown, wet, gravel, some sand, little silt.

The top of a water bearing layer with high permeability was encountered at approximate El. 523.5. Overlying the water bearing layer was a low permeability, confining layer creating positive pressure within the water bearing layer.

Corrected SPT N-values in the coarse-grained glacial till ranged from 32 to 77 bpf indicating the coarse-grained glacial till is dense to very dense in consistency. One Corrected SPT N-value in the fine-grained glacial till was 32 bpf indicating the fine-grained glacial till is hard in consistency. Six grain size analyses conducted on samples from the glacial till deposit resulted in the soils being classified as A-1-a, A-1-b, A-2-4, and A-4 under the AASHTO Soil Classification System and GM, SM, CL, GC-GM, and SC-SM under the USCS. The moisture content of the tested samples ranged from approximately 8 to 12 percent.

5.3 Groundwater

Groundwater measurements performed during the borings indicate groundwater levels ranged between 9 and 10 feet bbs. Groundwater observations are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Note that because water was introduced into the boreholes 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.

In addition to the recorded groundwater measurements, significant artesian pressure was observed during drilling boring BB-COB-102. The artesian pressure resulted in groundwater flowing out of the drill casing and onto the roadway surface at an estimated rate of 100 gallons per minute following sample BB-COB-102/9D, between 30 and 32 feet bbs.

6.0 FOUNDATION ALTERNATIVES

Given the span length of 22 feet, it was determined that a buried structure is the most suitable structure type to satisfy the purpose and need of this bridge replacement project. A precast concrete box culvert was chosen because of the structure's increased durability, ease and speed of construction, and economic advantage over similar buried structures. As such, a 90-foot long precast concrete box culvert with a 22-foot span and 8-foot rise is the preferred replacement structure. 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 90-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 a minimum of 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 approximate El. 540.0 to 539.0 feet. The glacial till encountered at this elevation is 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 28 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 dense to very dense in consistency. These coarse-grained materials are cohesionless and undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment will remain 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, Caribou 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, Caribou WSO Airport, Maine, has an air DFI from the Modberg database of approximately 2462 F-degree days. A water content of 10% was assumed. These components correlate to a frost depth of approximately 8.8 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 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

7.5 Seismic Design Considerations

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

7.6 Construction Considerations

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

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

The bottom of the proposed culvert excavation is underlain by a clay layer with low permeability that is confining water within a layer of high permeability. The Contractor must design, install, and operate dewatering and groundwater control systems so that the upward pressure within the layer of high permeability does not cause mass uplift of the soil or rupture the confining layer creating preferential seepage pathways into the excavation. Failure to install and operate proper

dewatering and groundwater control systems will significantly affect the stability of the excavation bottom.

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 Otter Stream Bridge in Caribou, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

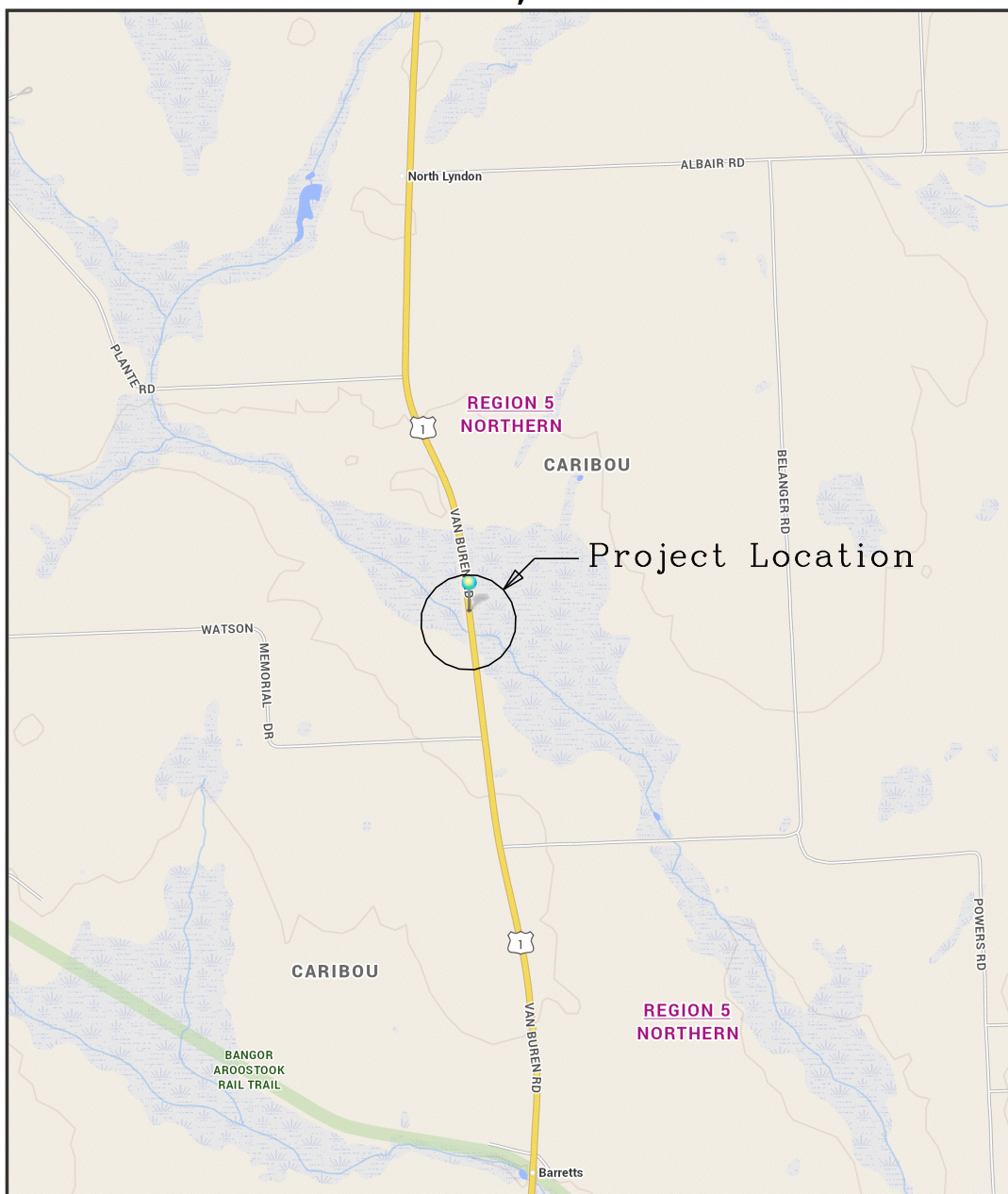
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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



CARIBOU, MAINE

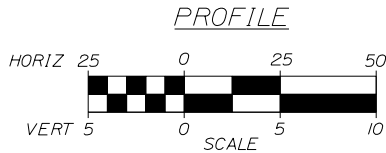
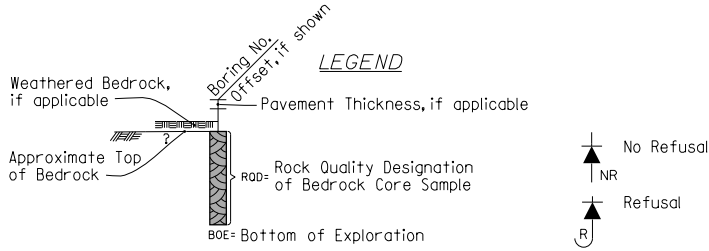
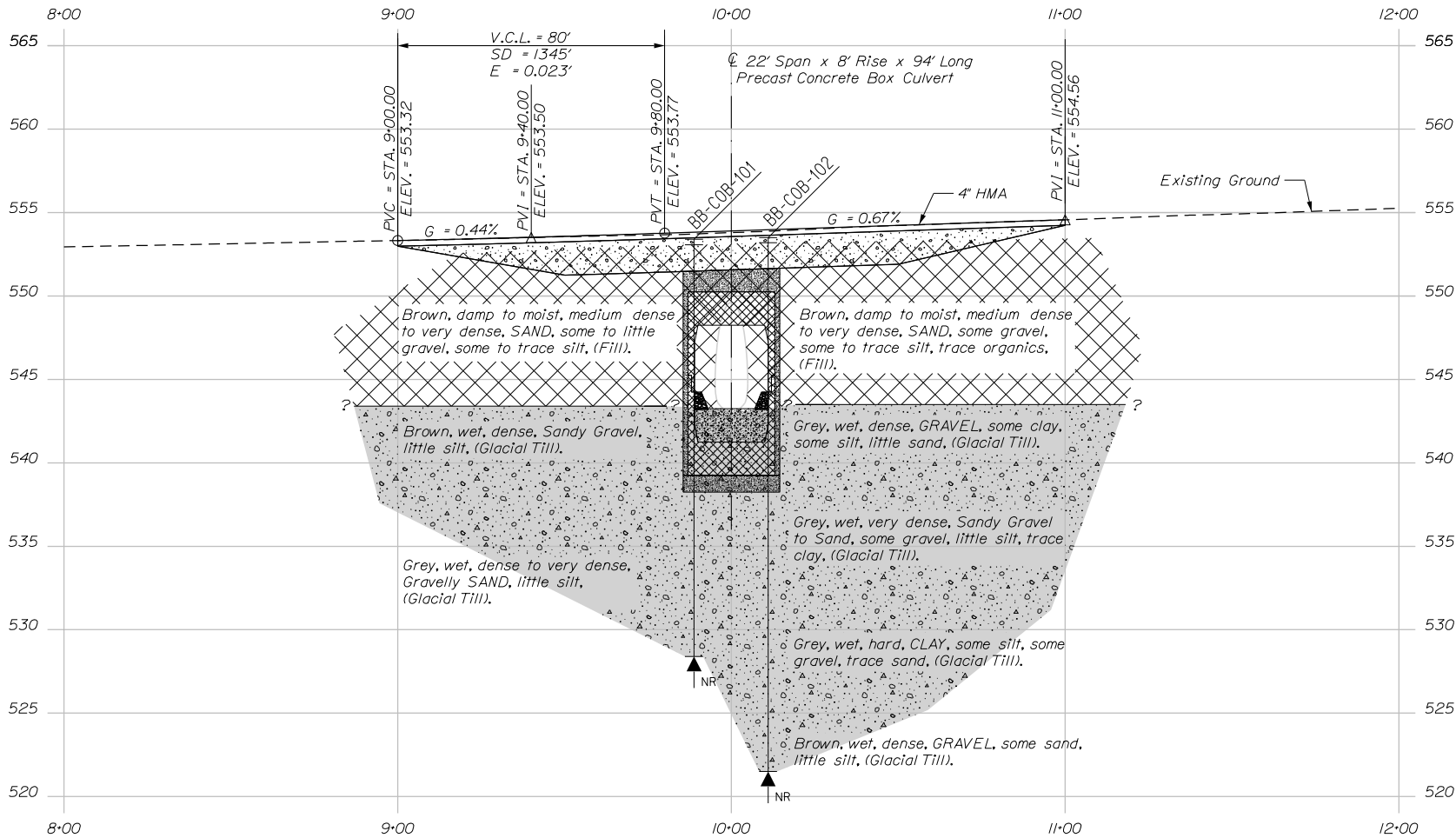
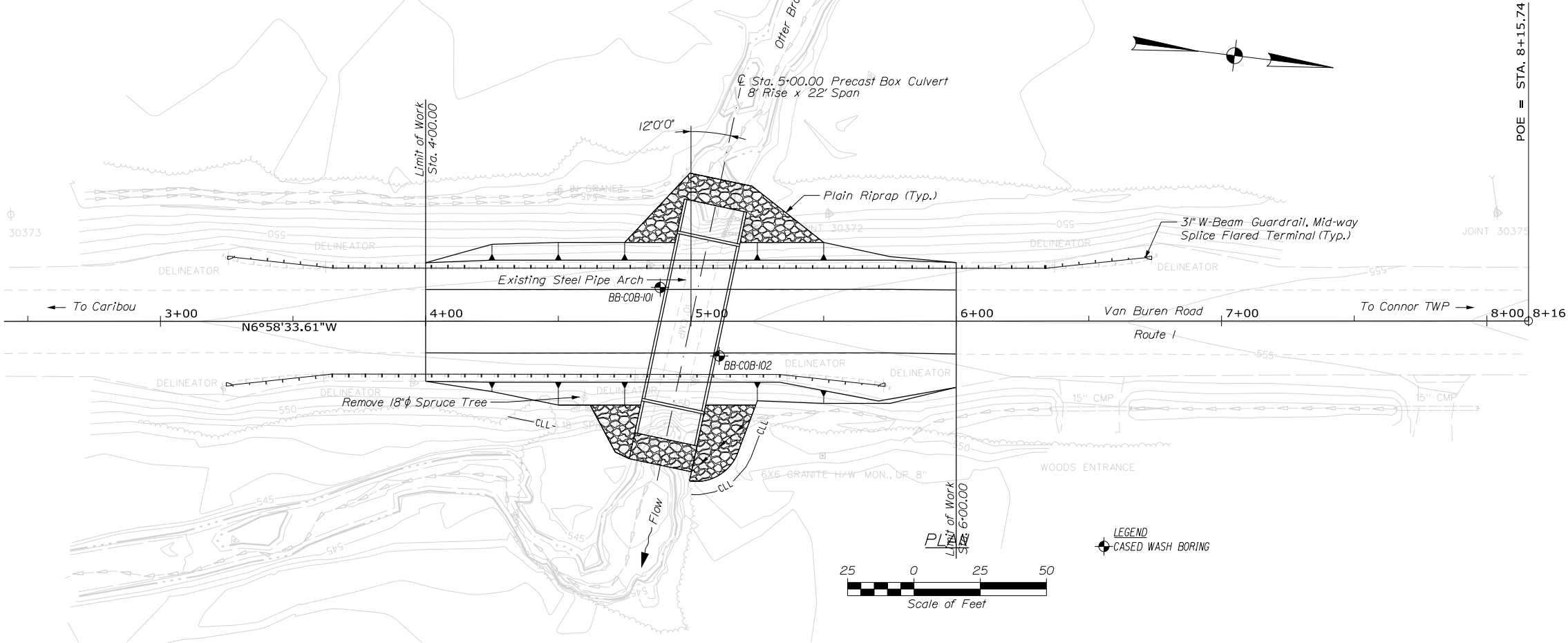


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1 inch = 0.54 miles

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SHEET NUMBER 1 OF 3	OTTER BROOK BRIDGE OTTER BROOK		STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
	CARIBOU AROOSTOOK COUNTY		022228.00	
	LOCATION MAP		WIN	
			BRIDGE NO. 5002	022228.00 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.

PROJ. MANAGER	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
CHECKED-REVIEWED					
DESIGNS DETAILER	T. WHITE	JAN 2019			
DESIGNS DETAILER					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

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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	<u>Coarse-grained soils</u> (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	<u>Density of Cohesionless Soils</u> Very loose Loose Medium Dense Dense Very Dense															
	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.	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.	<u>Fine-grained soils</u> (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.	<u>Approximate Undrained Shear Strength (psf)</u> Very Soft Soft Medium Stiff Stiff Very Stiff Hard															
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	<u>Field Guidelines</u> Fist easily penetrates Thumb easily penetrates Thumb penetrates with moderate effort Indented by thumb with great effort Indented by thumbnail Indented by thumbnail with difficulty																	
<u>Desired Soil Observations (in this order, if applicable):</u> 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					<u>Rock Quality Designation (RQD):</u> RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^*}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table><tr><th>Rock Mass Quality</th><th>RQD (%)</th></tr><tr><td>Very Poor</td><td>≤25</td></tr><tr><td>Poor</td><td>26 - 50</td></tr><tr><td>Fair</td><td>51 - 75</td></tr><tr><td>Good</td><td>76 - 90</td></tr><tr><td>Excellent</td><td>91 - 100</td></tr></table> <u>Desired Rock Observations (in this order, if applicable):</u> 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))				Rock Mass Quality	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100
Rock Mass Quality	RQD (%)																			
Very Poor	≤25																			
Poor	26 - 50																			
Fair	51 - 75																			
Good	76 - 90																			
Excellent	91 - 100																			
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					<u>Sample Container Labeling Requirements:</u> WIN Bridge Name / Town Boring Number Sample Number Sample Depth															
					Blow Counts Sample Recovery Date Personnel Initials															

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-101 WIN: 22228.00				
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.4				Auger ID/OD: 5" Solid Stem				
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon				
Logged By: N. Strout				Rig Type: Diedrich D-50				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 5/25/2018				Drilling Method: Cased-Wash				Core Barrel: N/A				
Boring Location: 9+88.8, 13.0 ft Lt.				Casing ID/OD: 4" / 4.5"				Water Level*: 10.0' (during drilling)				
Hammer Efficiency Factor: 0.918				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	1D	24/18	0.50 - 2.50	24/25/22/20	47	72	SSA	553.1		4" of Pavement.	G#296354 A-1-b, SW-SM WC=6.4%	
										Brown, damp, very dense, SAND, some gravel, trace silt, (Fill).		
	2D	24/20	2.50 - 4.50	16/13/10/9	23	35				Brown, moist, dense, SAND, little gravel, little silt, (Fill),		
5	3D	24/11	5.00 - 7.00	8/8/7/8	15	23				Brown, moist, medium dense, SAND, some silt, little gravel, (Fill).		
	4D	24/16	7.00 - 9.00	8/6/6/5	12	18				Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).		
10	5D	24/19	10.00 - 12.00	5/12/15/21	27	41	52	543.4		Brown, wet, dense, Sandy GRAVEL, little silt, (Glacial Till).		G#296356 A-1-a, GM WC=11.9%
							79					
							88					
							68					
							99					
15	6D	24/12	15.00 - 17.00	21/18/15/14	33	50	59			Grey, wet, dense, Gravelly SAND, little silt, (Glacial Till).	G#296357 A-1-b, SM WC=10.0%	
							56					
							51					
							75					
							113					
20							open					
	7D	24/19	21.00 - 23.00	8/11/12/19	23	35			Similar to above.			
	8D	24/22	23.00 - 25.00	16/21/25/26	46	70			Similar to above except very dense.			
25												
Remarks: Automatic Hammer SN 367 Casing driven using 140# Auto-Hammer												
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-COB-101		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-101 WIN: 22228.00			
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.4				Auger ID/OD: 5" Solid Stem			
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon			
Logged By: N. Strout				Rig Type: Diedrich D-50				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/25/2018				Drilling Method: Cased-Wash				Core Barrel: N/A			
Boring Location: 9+88.8, 13.0 ft Lt.				Casing ID/OD: 4" / 4.5"				Water Level*: 10.0' (during drilling)			
Hammer Efficiency Factor: 0.918				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value 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								528.4		Bottom of Exploration at 25.0 feet below ground surface. 25.0 Bottom of Exploration at 25.0 feet below ground surface. No refusal.	
50											
Remarks: Automatic Hammer SN 367 Casing driven using 140# Auto-Hammer											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2	
* 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-COB-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-102 WIN: 22228.00				
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.5				Auger ID/OD: 5" Solid Stem				
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon				
Logged By: N. Strout				Rig Type: Diedrich D-50				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 5/24/2018				Drilling Method: Cased-Wash				Core Barrel: N/A				
Boring Location: 10+11, 12.8 ft Rt.				Casing ID/OD: 4" / 4.5"				Water Level*: 9.0' (during drilling)				
Hammer Efficiency Factor: 0.918				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												
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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/18	0.50 - 2.50	22/28/29/25	57	87	SSA	553.1		3.75" of Pavement.	G#296358 A-1-b, SW-SM WC=6.7% G#296359 A-4, CL WC=12.1% G#296552 A-1-b, GC-GM WC=7.9% G#296553 A-2-4, SC-SM WC=8.3%	
										Brown, damp, very dense, SAND, some gravel, trace silt, (Fill).		
	2D	24/20	2.50 - 4.50	18/11/10/8	21	32				Similar to above except dense and little silt.		
5	3D	24/20	5.00 - 7.00	5/4/4/5	8	12				Brown, damp, medium dense, SAND, some gravel, trace silt, (Fill).		
	4D	24/2	7.00 - 9.00	5/4/3/2	7	11				Brown and grey, moist, medium dense, SAND, some gravel, some silt, trace organics, (Fill).		
10	5D	24/15	10.00 - 12.00	9/14/14/15	28	43	91	543.5		Grey, wet, dense, GRAVEL, some clay, some silt, little sand, (Glacial Till).		
							84					
							94					
							105					
							114					
15	6D	24/9	15.00 - 17.00	22/25/25/20	50	77	104			Grey, wet, very dense, Sandy GRAVEL, little silt, trace clay, (Glacial Till).		
							80					
							79					
							105					
							123					
20	7D	24/7	20.00 - 22.00	9/11/24/20	35	54	184			Grey, wet, very dense, SAND, some gravel, little silt, trace clay, (Glacial Till).		
							161					
							131					
							143					
25							134					
Remarks: Automatic Hammer SN 367 Casing driven using 140# Auto-Hammer bgs = below ground surface Artesian water flow at 30 ft bgs, following sample 9D (30-32 ft), flow at 100 gallons/minute												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2		
* 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-COB-102		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-102 WIN: 22228.00																																																																																																																																																																																																																																																																																																						
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.5				Auger ID/OD: 5" Solid Stem																																																																																																																																																																																																																																																																																																						
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon																																																																																																																																																																																																																																																																																																						
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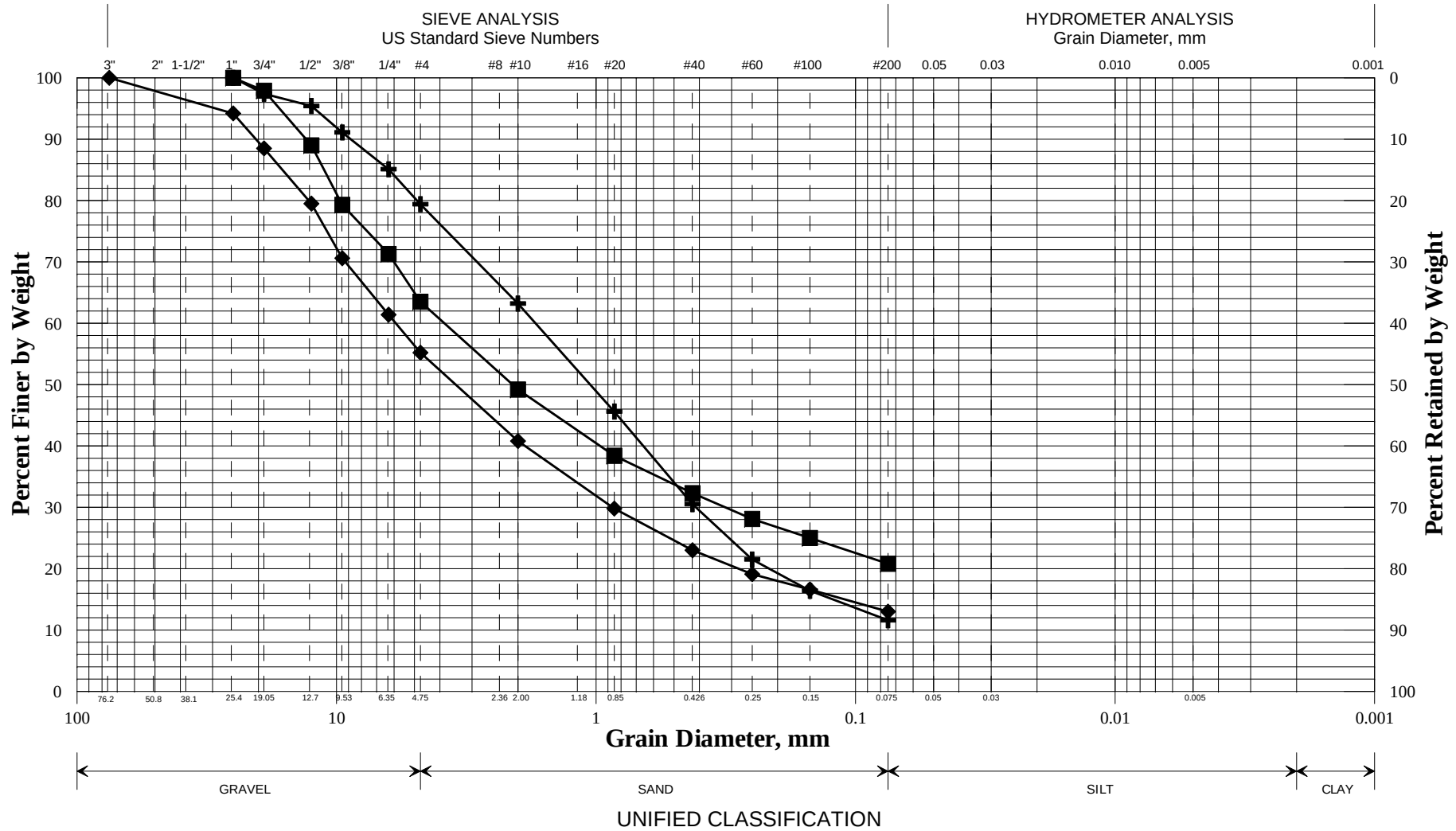
Appendix B

Laboratory Test Results

Work Number: 22228.00

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

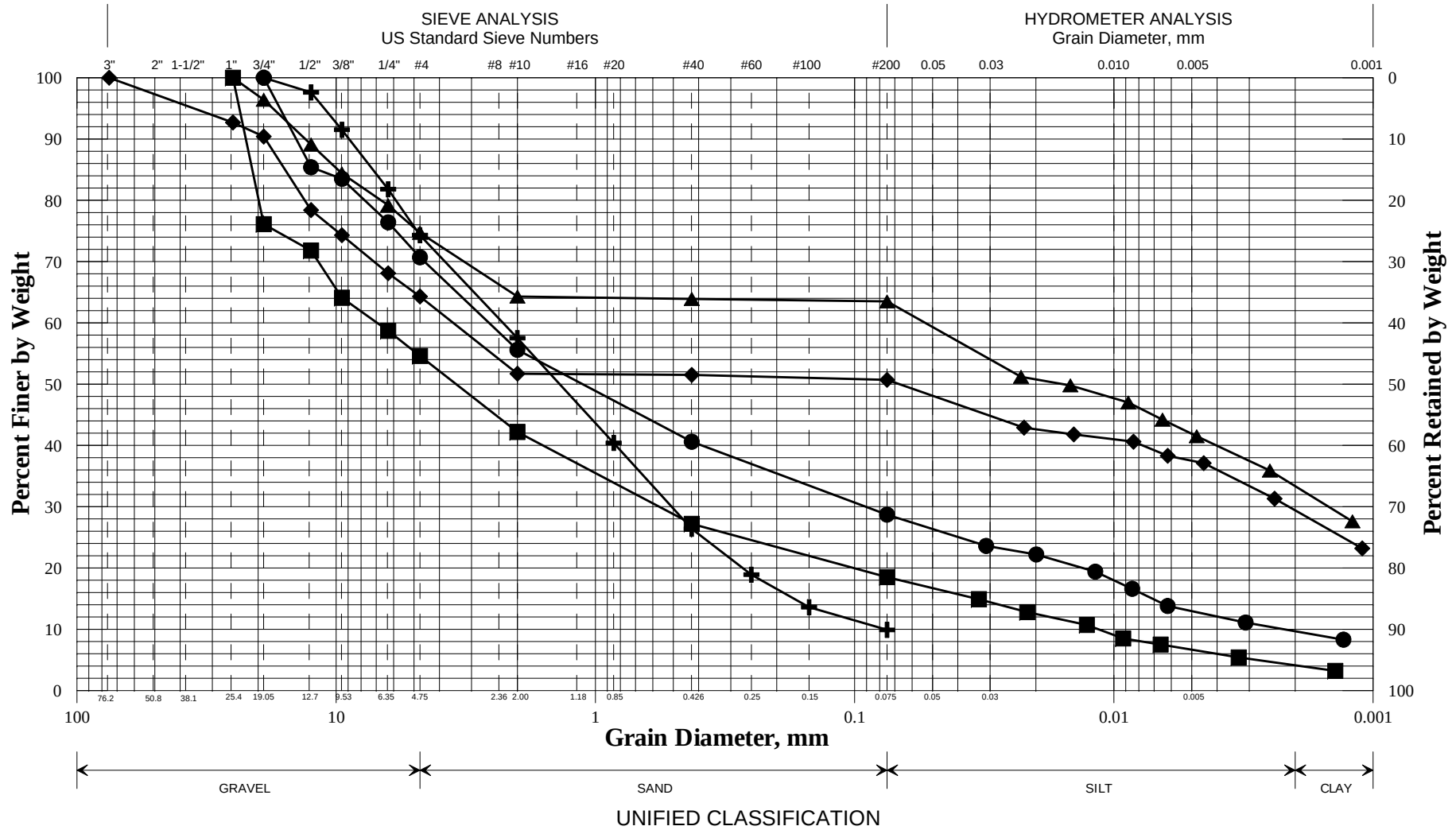
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-COB-101/2D	9+88.8	13.0 LT	2.5-4.5	SAND, little gravel, little silt.	6.4			
◆	BB-COB-101/5D	9+88.8	13.0 LT	10.0-12.0	Sandy GRAVEL, little silt.	11.9			
■	BB-COB-101/6D	9+88.8	13.0 LT	15.0-17.0	Gravelly SAND, little silt.	10.0			
●									
▲									
×									

WIN	
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Town	
Caribou	
Reported by/Date	
WHITE, TERRY A	7/23/2018

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-COB-102/3D	10+11	12.8 RT	5.0-7.0	SAND, some gravel, trace silt.	6.7			
◆	BB-COB-102/5D	10+11	12.8 RT	10.0-12.0	GRAVEL, some clay, some silt, little sand.	12.1			
■	BB-COB-102/6D	10+11	12.8 RT	15.0-17.0	Sandy GRAVEL, little silt, trace clay.	7.9			
●	BB-COB-102/7D	10+11	12.8 RT	20.0-22.0	SAND, some gravel, little silt, trace clay.	8.3			
▲	BB-COB-102/8D	10+11	12.8 RT	25.0-27.0	CLAY, some silt, some gravel, trace sand.	9.4			
×									

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Reported by/Date
WHITE, TERRY A 7/23/2018

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 and headwalls 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 $\beta := 26.6 \cdot \text{deg}$

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

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

Recommend: Active Earth Pressure Coefficient, $K_a = 0.46$

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

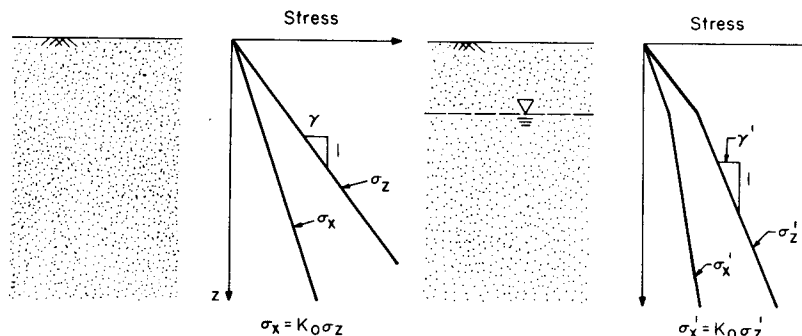


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

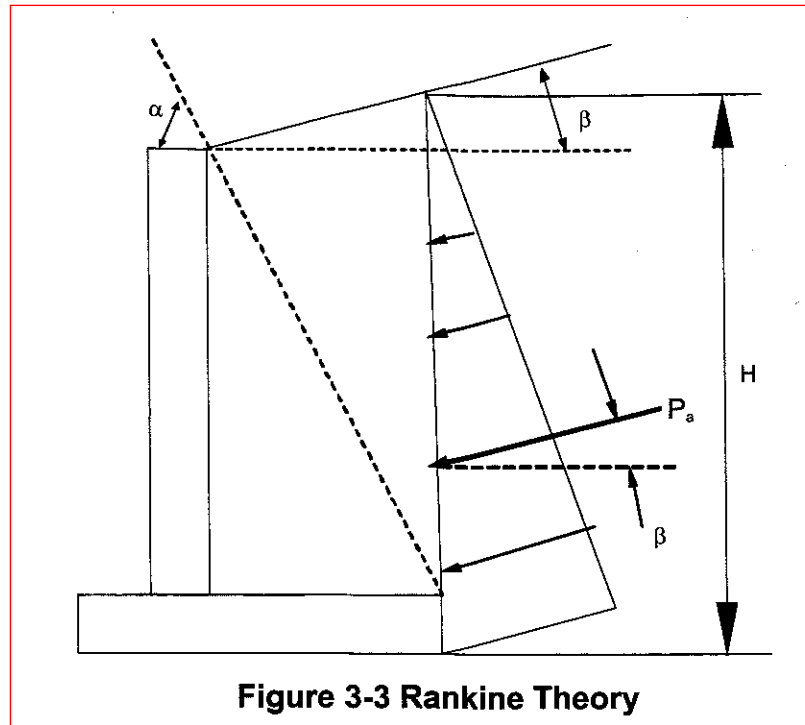


Figure 3-3 Rankine Theory

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 ignored.
3. The proposed bearing elevation is approximately 539.5 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 approximately 64.5 feet long (excluding 12.25 foot long slope tapered sections).
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 41 bpf for the consistency of the soils encountered at the box bearing elevation.
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
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Medium Dense to Dense	4-8	6

The glacial till deposit is dense. 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

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

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

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

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

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:

$$\gamma_{1d} := 134 \cdot \text{pcf}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Glacial Till

$$w_{\text{sat}} := .12$$

Lab results from BB-COB-101;5D

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

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

$$\gamma_{1\text{sat}} = 150.08 \cdot \text{pcf}$$

$$N_{\text{design}} := 41$$

$$\phi := 38 \cdot \text{deg}$$

Lambe and Whitman, Soil Mechanics Fig. 11.14

$$c := 0$$

Cohesion

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

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

$$s_{\gamma} := 0.8$$

$$s_c := 1.3$$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223), $\phi = 38$ degrees.

$$N_c := 61.31$$

$$N_q := 48.9$$

$$N_{\gamma} := 64.0$$

Nominal Bearing Resistance per Terzaghi equation

$$q := D_f \cdot (\gamma_{\text{above}} - \gamma_w)$$

$$q = 0.063 \cdot \text{ksf}$$

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

$$q_n = 56.9 \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.45$$

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

$$q_r = 25.6 \cdot \text{ksf}$$

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)

$$N_c := 61.4$$

$$N_q := 48.9$$

$$N_\gamma := 78.0$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$L := 64.5 \cdot \text{ft} \quad B := 24 \cdot \text{ft}$$

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

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

$$s_\gamma = 0.85$$

$$s_q = 1.29$$

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

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

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

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

$$N_{cm} = 79.82$$

$$N_{\gamma m} = 66.391$$

$$N_{qm} = 63.116$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 63.7 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 28.7 \cdot \text{ksf}$$

Recommend 28 ksf for the factored bearing resistance at the strength limit state for precast concrete box culverts 24 ft wide or greater.

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

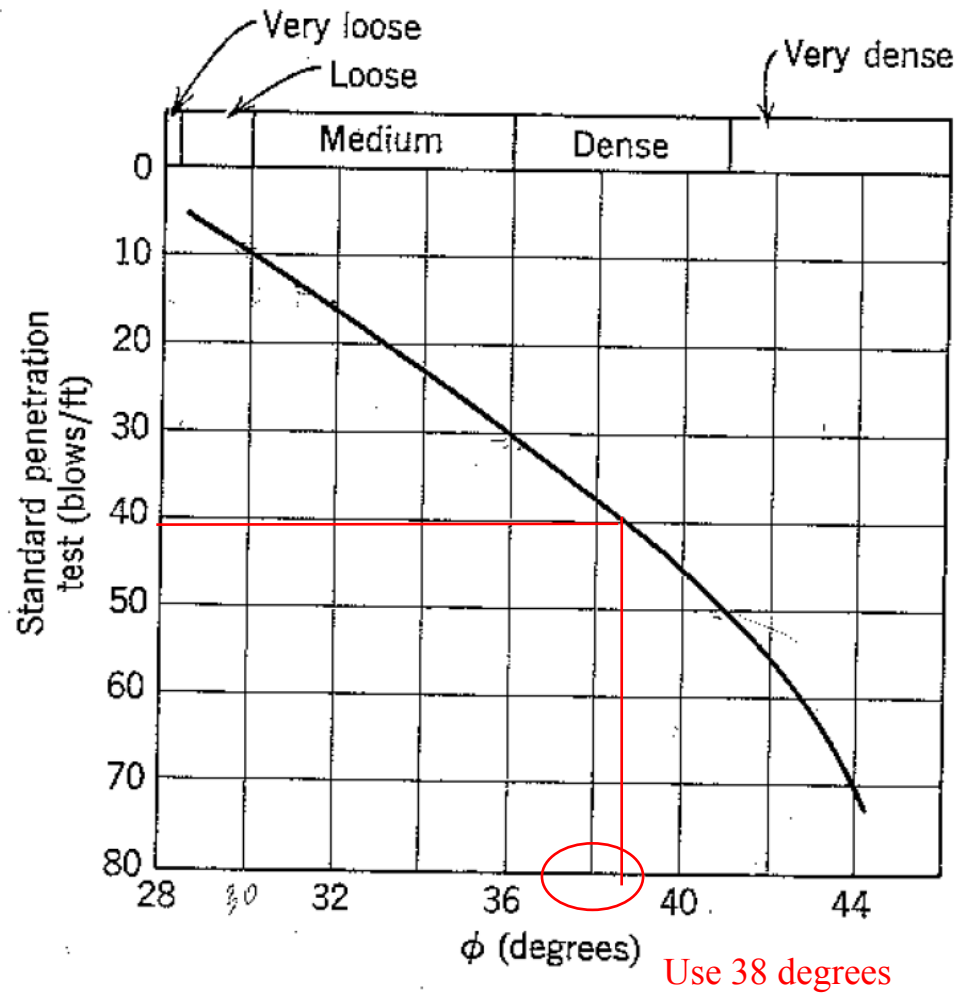


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

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.

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

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

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

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

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

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

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

$$\text{when } \phi = 0$$

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

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

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

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

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.

So, the allowable load per unit area of the foundation is

$$q_{\text{all}} = \frac{q_u}{\text{FS}} = \frac{1078.29}{3} \approx 359.5 \text{ kN/m}^2$$

Thus, the total allowable gross load is

$$Q = (359.5) B^2 = (359.5) (2 \times 2) = 1438 \text{ kN}$$

3.5 Modification of Bearing Capacity Equations for Water Table

Equations (3.3) and (3.7) through (3.11) give the ultimate bearing capacity, based on the assumption that the water table is located well below the foundation. However, if the water table is close to the foundation, some modifications of the bearing capacity equations will be necessary. (See Figure 3.6.)

Case I. If the water table is located so that $0 \leq D_1 \leq D_f$, the factor q in the bearing capacity equations takes the form

$$q = \text{effective surcharge} = D_1 \gamma + D_2 (\gamma_{\text{sat}} - \gamma_w) \quad (3.16)$$

where

γ_{sat} = saturated unit weight of soil

γ_w = unit weight of water

Also, the value of γ in the last term of the equations has to be replaced by $\gamma' = \gamma_{\text{sat}} - \gamma_w$.

Case II. For a water table located so that $0 \leq d \leq B$,

$$q = \gamma D_f \quad (3.17)$$

In this case, the factor γ in the last term of the bearing capacity equations must be replaced by the factor

$$\bar{\gamma} = \gamma' + \frac{d}{B} (\gamma - \gamma') \quad (3.18)$$

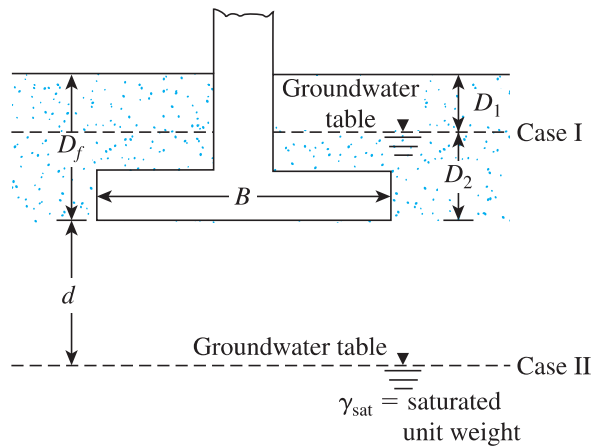


Figure 3.6 Modification of bearing capacity equations for water table

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

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

10.5.5.2.2—Spread Footings

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

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

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

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

C10.5.5.2.2

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

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

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

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

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

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

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

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

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

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

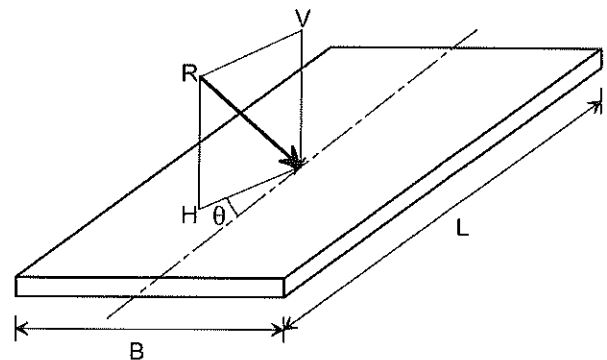


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

Frost

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

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

DFI = 2600 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

$d = 109 \cdot \text{in}$

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

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Use Caribou WSO Airport data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Caribou WSO Airport, Maine

Air Design Freezing Index = 2462 F-days

N-Factor = 0.80

Surface Design Freezing Index = 1970 F-days

Mean Annual Temperature = 49.2 deg F

Design Length of Freezing Season = 156 days

Layer

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

1-Coarse	105.0	10.0	125.0	28	34	2.0	1.6	1,800
----------	-------	------	-------	----	----	-----	-----	-------

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).

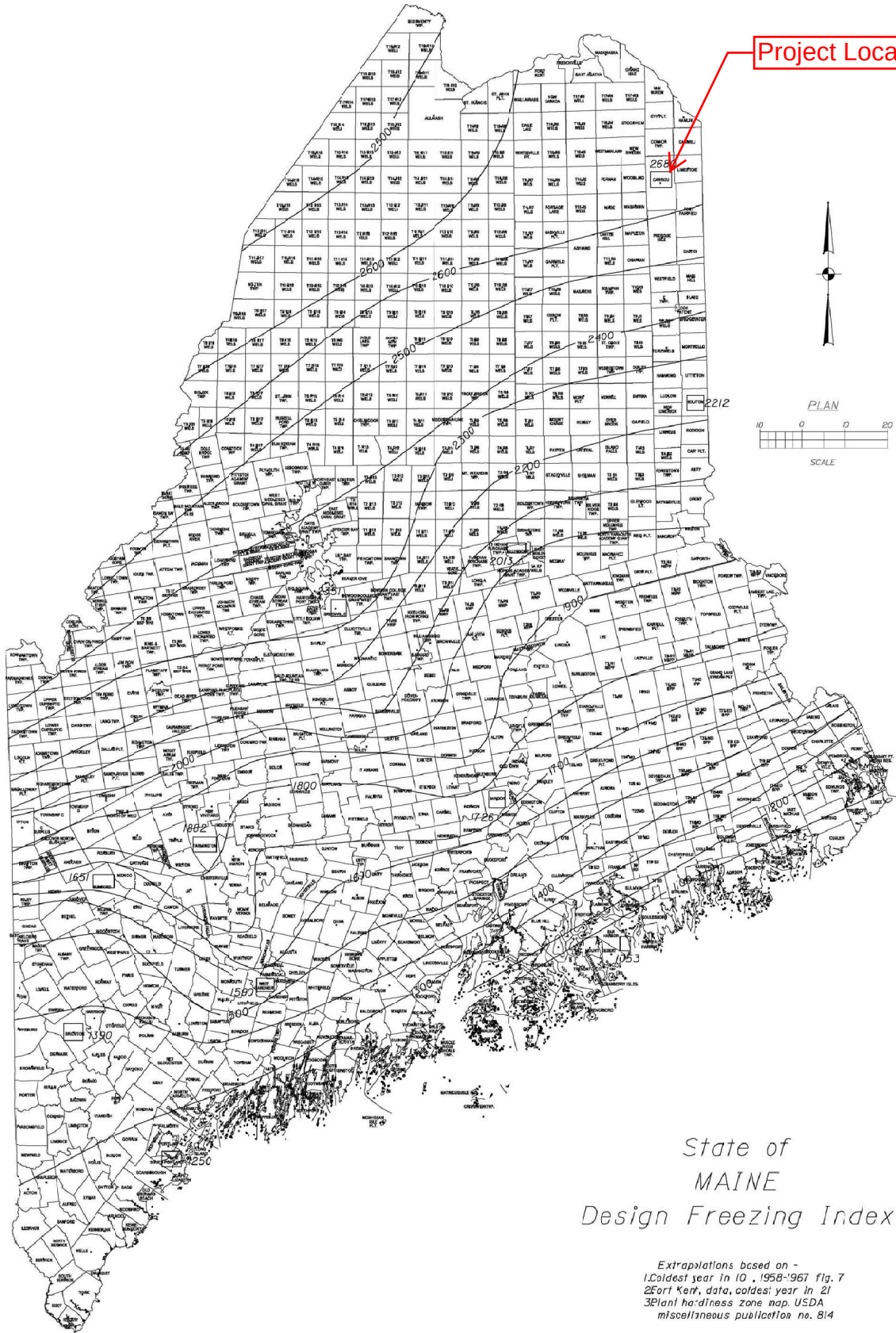
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

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

Total Depth of Frost Penetration = 8.75 ft = 105.0 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

Basal Heave

Objective:

Evaluate the potential for basal heave during excavation for the box culvert.

Given:

1. Flow estimate of 100 gpm.
2. Steel (HW) casing has a 4 inch inside diameter.
3. Top of high permeability zone at Elevation 521.5.
4. Road surface Elevation 553.5.

Assumptions:

1. Water temperature is 40 degrees.
2. No head loss due to friction occurs.
3. Casing stick up of approx. one foot above road surface to an elevation of 554.5.
4. The high permeability zone is large and stagnate so that the water surface inside the aquifer does not change by outflow and the velocity is near zero.
5. The pressure within the high permeability zone is constant.

A. Find velocity of water flowing out of the casing at 100 gpm.

Flow out of HW casing

Area of HW casing

$$Q := 100 \frac{\text{gal}}{\text{min}} = 2.2 \times 10^{-1} \cdot \frac{\text{ft}^3}{\text{s}}$$

$$A := \pi \cdot (2\text{in})^2 = 8.7 \times 10^{-2} \cdot \text{ft}^2$$

Velocity of water exiting casing

$$V := \frac{Q}{A} = 2.553 \cdot \frac{\text{ft}}{\text{s}}$$

B. Solve Bernoulli's Equation to find pressure within the high perm zone, P_1 .

Weight of 40 Degree water

Gravitational force

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

$$g := 32.2 \frac{\text{ft}}{\text{s}^2}$$

Velocity of water surface within aquifer

Velocity of water exiting casing

$$V_1 := 0 \frac{\text{ft}}{\text{s}}$$

$$V_2 := 2.553 \frac{\text{ft}}{\text{s}}$$

Elevation of water surface within aquifer

Elevation of water surface exiting the casing

$$Z_1 := 523.5 \text{ft}$$

$$Z_2 := 554.5 \text{ft}$$

Pressure at top of casing

Velocity of water surface within aquifer

$$P_2 := 0 \text{psi}$$

$$V_1 := 0 \frac{\text{ft}}{\text{s}}$$

$$P_1 := \gamma_w \left[\frac{P_2}{\gamma} + \frac{(V_2)^2 - (V_1)^2}{2 \cdot g} + Z_2 - Z_1 \right] \quad \text{Ref: Fluid Mechanics Eq 5-75}$$

$$P_1 = 13.5 \cdot \text{psi}$$

C. Find Force of water acting on one square foot of the low permeability confining layer.

$$F := P_1 \cdot 1 \text{ ft} \cdot \frac{144 \text{ in}^2}{1 \text{ ft}} = 1940.7 \cdot \text{lbf}$$

D. Estimate the remaining overburden pressure on one square foot of the low perm confining layer after excavation.

Bottom of Excavation elevation

$$Z_3 := 539 \text{ ft}$$

Dry unit weight of glacial till

$$\gamma_{1d} := 134 \cdot \text{pcf}$$

Moisture content at saturated condition

$$w_{1\text{sat}} := .12$$

Saturated unit weight of glacial till

$$\gamma_{1\text{sat}} := \gamma_{1d} \cdot (1 + w_{1\text{sat}}) = 150.1 \cdot \text{pcf}$$

Effective unit weight of glacial till

$$\gamma_{1'} := \gamma_{1\text{sat}} - \gamma_w = 87.7 \cdot \text{pcf}$$

Thickness of Glacial Till

$$h_1 := Z_3 - Z_4 = 10.5 \cdot \text{ft}$$

Bottom of confining layer elevation

$$Z_5 := 523.5 \text{ ft}$$

Dry unit weight of clay

$$\gamma_{2d} := 108 \text{ pcf}$$

Moisture content at saturated condition

$$w_{2\text{sat}} := .09$$

Saturated unit weight of clay

$$\gamma_{2\text{sat}} := \gamma_{2d} \cdot (1 + w_{2\text{sat}}) = 117.7 \cdot \text{pcf}$$

Effective unit weight of clay

$$\gamma_{2'} := \gamma_{2\text{sat}} - \gamma_w = 55.3 \cdot \text{pcf}$$

Thickness of confining layer

$$h_2 := Z_4 - Z_5 = 5 \cdot \text{ft}$$

Top of confining layer elevation

$$Z_4 := 528.5 \text{ ft}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Glacial Till

Lab results from BB-COB-101;5D

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

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Stiff Clay

Lab results from BB-COB-102;8D

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

Effective overburden pressure remaining over one square foot of the confining layer

$$\sigma' := (\gamma_1' \cdot h_1 + \gamma_2' \cdot h_2) \cdot 1 \text{ ft}^2 = 1197 \cdot \text{lbf}$$

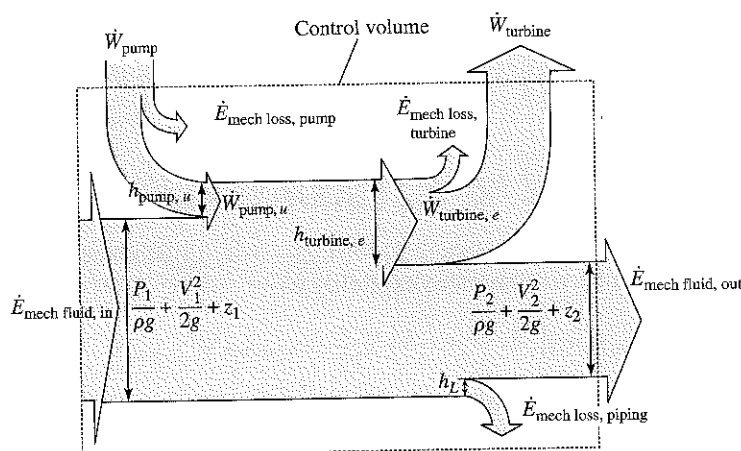
Pressure difference

$$\Delta p := F - \sigma' = 743 \cdot \text{lbf}$$

Pressure within the highly permeable zone acting on the confining layer that is larger than the pressure over the layer of low permeability will force water through the layer of low permeability and create heave the bottom of the excavation and/or create preferential flow paths. Heave and flow through preferential flow paths will significantly affect the stability of the excavation.

FIGURE 5-51

Mechanical energy flow chart for a fluid flow system that involves a pump and a turbine. Vertical dimensions show each energy term expressed as an equivalent column height of fluid, i.e., *head*, corresponding to each term of Eq. 5-74.



The *pump head* is zero if the piping system does not involve a pump, a fan, or a compressor, and the *turbine head* is zero if the system does not involve a turbine. Also, the *head loss* h_L can sometimes be ignored when the frictional losses in the piping system are negligibly small compared to the other terms in Eq. 5-74.

Special Case: Incompressible Flow with No Mechanical Work Devices and Negligible Friction

When piping losses are negligible, there is negligible dissipation of mechanical energy into thermal energy, and thus $h_L = e_{\text{mech loss, piping}}/g \cong 0$, as shown later in Example 5-11. Also, $h_{\text{pump, u}} = h_{\text{turbine, e}} = 0$ when there are no mechanical work devices such as fans, pumps, or turbines. Then Eq. 5-74 reduces to

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 \quad \text{or} \quad \frac{P}{\rho g} + \frac{V^2}{2g} + z = \text{constant} \quad (5-75)$$

which is the Bernoulli equation derived earlier using Newton's second law of motion.

Kinetic Energy Correction Factor, α

The average flow velocity V_{avg} was defined such that the relation $\rho V_{\text{avg}} A$ gives the actual mass flow rate. Therefore, there is no such thing as a correction factor for mass flow rate. However, as Gaspard Coriolis (1792–1843) showed, the kinetic energy of a fluid stream obtained from $V^2/2$ is not the same as the actual kinetic energy of the fluid stream since the square of a sum is not equal to the sum of the squares of its components (Fig. 5-52). This error can be corrected by replacing the kinetic energy terms $V^2/2$ in the energy equation by $\alpha V_{\text{avg}}^2/2$, where α is the kinetic energy correction factor. By using equations for the variation of velocity with the radial distance, it can be shown that the correction factor is 2.0 for fully developed laminar pipe flow, and it ranges between 1.04 and 1.11 for fully developed turbulent flow in a round pipe.

The kinetic energy correction factors are often ignored (i.e., α is set equal to 1) in an elementary analysis since (1) most flows encountered in practice are turbulent, for which the correction factor is near unity, and (2) the

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-102 WIN: 22228.00				
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.5				Auger ID/OD: 5" Solid Stem				
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon				
Logged By: N. Strout				Rig Type: Diedrich D-50				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 5/24/2018				Drilling Method: Cased-Wash				Core Barrel: N/A				
Boring Location: 10+11, 12.8 ft Rt.				Casing ID/OD: 4" / 4.5"				Water Level*: 9.0' (during drilling)				
Hammer Efficiency Factor: 0.918				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	1D	24/18	0.50 - 2.50	22/28/29/25	57	87	SSA	553.1		3.75" of Pavement.	G#296358 A-1-b, SW-SM WC=6.7%	
										Brown, damp, very dense, SAND, some gravel, trace silt, (Fill).		
	2D	24/20	2.50 - 4.50	18/11/10/8	21	32				Similar to above except dense and little silt.		
5	3D	24/20	5.00 - 7.00	5/4/4/5	8	12				Brown, damp, medium dense, SAND, some gravel, trace silt, (Fill).		G#296359 A-4, CL WC=12.1%
	4D	24/2	7.00 - 9.00	5/4/3/2	7	11				Brown and grey, moist, medium dense, SAND, some gravel, some silt, trace organics, (Fill).		
10	5D	24/15	10.00 - 12.00	9/14/14/15	28	43	91	543.5		Grey, wet, dense, GRAVEL, some clay, some silt, little sand, (Glacial Till).		G#296552 A-1-b, GC-GM WC=7.9%
							84					
							94					
							105					
15	6D	24/9	15.00 - 17.00	22/25/25/20	50	77	104			Grey, wet, very dense, Sandy GRAVEL, little silt, trace clay, (Glacial Till).	G#296553 A-2-4, SC-SM WC=8.3%	
							80					
							79					
							105					
20	7D	24/7	20.00 - 22.00	9/11/24/20	35	54	184			Grey, wet, very dense, SAND, some gravel, little silt, trace clay, (Glacial Till).		
							161					
							131					
							143					
25							134					
Remarks: Automatic Hammer SN 367 Casing driven using 140# Auto-Hammer bgs = below ground surface Artesian water flow at 30 ft bgs, following sample 9D (30-32 ft), flow at 100 gallons/minute												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2		
* 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-COB-102		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Otter Brook Bridge #5902 carries Route 1 over Otter Brook Location: Caribou, Maine				Boring No.: BB-COB-102 WIN: 22228.00																																																																																																																																																																																																																																																																																																							
Driller: S. W. Cole Explorations, Inc.				Elevation (ft.): 553.5				Auger ID/OD: 5" Solid Stem																																																																																																																																																																																																																																																																																																							
Operator: K. Hanscom				Datum: NAVD88				Sampler: Standard Split-Spoon																																																																																																																																																																																																																																																																																																							
Logged By: N. Strout				Rig Type: Diedrich D-50				Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																																																																							
Date Start/Finish: 5/24/2018				Drilling Method: Cased-Wash				Core Barrel: N/A																																																																																																																																																																																																																																																																																																							
Boring Location: 10+11, 12.8 ft Rt.				Casing ID/OD: 4" / 4.5"				Water Level*: 9.0' (during drilling)																																																																																																																																																																																																																																																																																																							
Hammer Efficiency Factor: 0.918				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																																																											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value 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																																																																																																																																																																																																																																																																																																			
Sample Information <table><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Size (in.)</th><th>Location</th><th>Thick Log</th><th>Visual Description and Remarks</th><th>Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><td rowspan="4">25</td><td>8D</td><td>24/16</td><td>25.00 - 27.00</td><td>8/10/11/11</td><td>21</td><td>32</td><td>78</td><td></td><td rowspan="4"> EI 528.5 </td><td rowspan="4">Grey, wet, hard, CLAY, some silt, some gravel, trace sand, (Glacial Till).</td><td rowspan="4">G#296360 A-4, CL WC=9.4%</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">30</td><td>9D</td><td>24/8</td><td>30.00 - 32.00</td><td>8/10/11/15</td><td>21</td><td>32</td><td></td><td></td><td rowspan="4"> EI 523.5 </td><td rowspan="4">Brown, wet, dense, GRAVEL, some sand, little silt.</td><td rowspan="4">high perm layer</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="12">35</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12">521.5</td><td rowspan="12">Bottom of Exploration at 32.0 feet below ground surface. No refusal.</td><td rowspan="12">32.0</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></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></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></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">40</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="4"></td><td rowspan="4"></td><td rowspan="4"></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">45</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="4"></td><td rowspan="4"></td><td rowspan="4"></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">50</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="4"></td><td rowspan="4"></td><td rowspan="4"></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></table>												Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Size (in.)	Location	Thick Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	25	8D	24/16	25.00 - 27.00	8/10/11/11	21	32	78		EI 528.5	Grey, wet, hard, CLAY, some silt, some gravel, trace sand, (Glacial Till).	G#296360 A-4, CL WC=9.4%																									30	9D	24/8	30.00 - 32.00	8/10/11/15	21	32			EI 523.5	Brown, wet, dense, GRAVEL, some sand, little silt.	high perm layer																									35									521.5	Bottom of Exploration at 32.0 feet below ground surface. No refusal.	32.0																																																																																									40																																				45																																				50																																			
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