

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

GEOTECHNICAL DESIGN REPORT

For the Construction of

**GREAT BROOK BRIDGE
FOREST AVENUE
ORONO, MAINE**

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Penobscot County
WIN 27196.00

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Soils Report 2026-43
Bridge No. 6795

Additional review of this report provided by
S.W.Cole, see stamped memo dated June 2026.

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of existing twin culverts (#914698) which carries Forest Avenue over an unnamed stream in Orono, Maine. A subsurface investigation has been completed at the site to evaluate subsurface conditions and to develop geotechnical design and construction recommendations for the replacement structure. This report presents the subsurface information obtained during the subsurface investigation and soil laboratory testing programs, and provides geotechnical design parameters and construction recommendations for the large culvert replacement.

The existing crossing consists of dual 36-inch diameter, approximately 70-foot-long corrugated metal pipe (CMP) culverts. The CMPs are in poor condition and need replacement both from an infrastructure and environmental standpoint. No original construction plans for the existing crossing were available for review. Forest Avenue is a Highway Corridor Priority 4 road.

The proposed replacement structure will be an approximately 12-foot span by 6-foot rise by 74-foot-long precast concrete box culvert. The box culvert shall have 2-foot deep toe walls at the inlet and outlet. The invert of the proposed culvert is approximately 10.8 feet below the existing road grade at the roadway centerline. The box culvert invert will be embedded 2 feet into the streambed. The roadway embankment slopes at the proposed culvert inlet and outlet shall be no steeper than 1.75H:1V to protect against erosion.

The new box culvert will be located on horizontal and vertical alignments that will approximately match existing alignments. The roadway will be closed during construction to facilitate removal of the existing culvert and installation of the proposed box culvert, and traffic will be detoured onto state and local roads.

2.0 GEOLOGIC SETTING

The existing culvert carries Forest Avenue over an unnamed stream in Orono and is located approximately 0.12 miles west of Essex Street as shown on Sheet 1 – Location Map.

According to the Maine Geological Survey (MGS) map titled Surficial Geology of the Pushaw Lake Quadrangle, Maine, Open File 09-18 (2009) the surficial soils at the site consist of Presumpscot Formation and Glacial Till. Presumpscot Formation consists of silt and clay. Glacial Till consists of gravel, sand, silt, and clay.

According to the map titled Bedrock Geologic Map of Maine (1985) published by the MGS, the bedrock in the vicinity of the site consists of calcareous sandstone, interbedded sandstone, and impure limestone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

One (1) boring (HB-ORN-101) and one (1) probe (HB-ORN-102) were drilled for this project on October 23, 2023, by the MaineDOT drill crew using a trailer-mounted drill rig. Boring HB-ORN-101 was drilled behind the existing culvert at the southwest corner, and Probe HB-ORN-102 was drilled behind the existing culvert at the northeast corner. The boring was terminated 18.4 feet below ground surface (bgs) in bedrock, and the probe was terminated at a depth 15.3 feet bgs. Exploration locations are shown on Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile with Boring Logs. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on the Boring Logs in Appendix A.

Boring HB-ORN-101 was drilled using solid stem auger, cased wash boring, and rock core drilling techniques. Soil samples were obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in November 2022. All N-values discussed in this report are corrected values (N_{60}) computed by applying an average energy transfer factor of 0.906 to the raw field N-values. This hammer efficiency factor (0.906) and both the raw field N-value and corrected N-value (N_{60}) are shown on the Boring Logs in Appendix A. Probe HB-ORN-102 was drilled using solid stem auger techniques. No soil samples were obtained in the probe.

The MaineDOT Geotechnical Team member selected the boring and probe locations, drilling methods, designated type and depth of sampling, reviewed field logs for accuracy and identified field and laboratory testing requirements. A NorthEast Transportation Training and Certification (NETTCP) certified Subsurface Investigator logged the subsurface conditions encountered. The boring and probe were located in the field by taping to surveyed site features after completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted to assist in soil classification, evaluation of engineering properties of the soils and geologic assessment of the project site. Laboratory testing consisted of two (2) standard grain size analyses with natural water content. The results of the laboratory testing program are discussed in the following section and are included in Appendix B – Laboratory Test Results. Laboratory test information is also shown on the Boring Logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test boring and probe generally consisted of fill, underlain by native clayey silt, underlain by glacial till, underlain by bedrock. An interpretive subsurface profile depicting the generalized soil stratigraphy at the boring location is shown on Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile with Boring Logs.

Boring HB-ORN-101 was drilled to depth of approximately 13.4 feet bgs where bedrock was encountered. The total boring depth was 18.4 feet bgs, including a bedrock core approximately 5.0 feet in length. Probe HB-ORN-102 was drilled to depth of approximately 15.3 feet bgs, where a refusal surface was encountered. The exact nature of the refusal surface was not determined in the probe. Weathered bedrock was encountered in probe HB-ORN-102 at depth of approximately 14.0 feet bgs.

The table below summarizes the field and laboratory information obtained in boring HB-ORN-101:

Approx. Depth BGS ¹ (feet)	Soil Description	AASHTO ² Classification	USCS ³	WC% ⁴
0.0 – 7.5	Fill: Brown, moist to wet, fine to coarse sand, some gravel, little silt.	A-1-b	SM	7.2
7.5 – 9.5	Presumpscot Formation: Grey, wet, clayey silt, trace fine sand.	--	--	--
9.5 – 13.4	Glacial Till: Grey, wet, silty gravel, little fine to coarse sand.	A-2-4	GM	10.1
13.4 – 18.4	Bedrock	--	--	--

¹BGS = below ground surface

²AASHTO = American Association of State Highway and Transportation Officials

³USCS = Unified Soil Classification System

⁴WC% = Water content in percent

One (1) N₆₀-value obtained in the sand fill was 21 blows per foot (bpf) indicating that the fill is medium dense. One (1) N₆₀-value obtained in the glacial till was 35 bpf, indicating that the till is dense. The Rock Quality Designation (RQD) of the bedrock was determined to be 80 percent in boring HB-ORN-101 which correlates to a Rock Mass Quality of Good.

Groundwater was recorded at a depth 5.8 feet bgs in boring HB-ORN-101. Groundwater levels can be expected to fluctuate subject to seasonal variations, local soil conditions, topography, precipitation, and construction activity.

6.0 FOUNDATION ALTERNATIVE

A precast concrete box culvert was the only bridge replacement alternative considered for this project. A precast concrete box culvert satisfies the purpose and need of this project because of the structure's durability, ease and speed of construction, and economic advantages.

7.0 GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

The following sections discuss geotechnical recommendations for the design and construction of the proposed culvert. Geotechnical engineering evaluations were made in accordance with 2024 AASHTO LRFD Bridge Design Specifications 10th Edition (LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through June 2018 (BGD).

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 12-foot span by 6-foot rise by 74-foot-long precast concrete box culvert. To prevent undermining, the box culvert will have 2-foot inlet and outlet toe walls (cutoff walls) embedded at or below the bottom of riprap aprons and culvert support materials. 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.

The proposed structure shall be bedded on a 1-foot-thick layer of Granular Borrow, which shall meet the requirements of Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The Granular Borrow bedding and backfill material shall be placed in 6-inch lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor.

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. To limit future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

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 and reinforcement. The design shall be 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, 10th Edition 2024 (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.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will not be included in the culvert design.

7.1.2 Precast Concrete Box Inlet and Outlet walls

The precast concrete box culvert's base, ceiling, and full height outlet and inlet walls will extend the full length of the culvert. The left and right outlet walls are integral to the precast culvert and will share the same precast base slab. The full height inlet and 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 inlet and outlet walls shall be designed to resist lateral earth pressures, vehicular loads and forces resulting from creep, temperature and shrinkage deformations of the concrete box culvert. The inlet and 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. Passive pressure resulting from the embedment of the box culvert and walls with engineered streambed, or any other material shall not contribute to resisting forces.

The inlet and outlet walls are fixed to the box culvert for the entire length, and should be designed to resist movement using an at-rest earth pressure coefficient, K_o , of 0.47.

7.1.3 Precast Concrete Inlet and outlet Toe Walls

Toe walls (cutoff 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 and should bear on natural soils at/below the bottom of the Culvert Bedding Stone and riprap apron materials.

7.1.4 Bearing Resistance

To provide a stable subgrade, it is recommended that the precast concrete box culvert be underlain by a 1-foot-thick layer of Granular Borrow placed on the native soil subgrade.

The factored bearing resistance for the precast concrete box culvert bearing on compacted granular bedding material placed on dense glacial till native soils at the service and strength limit states are presented in the table below. Supporting calculations in accordance with LRFD are provided in Appendix C – Calculations.

Limit State	Resistance Factor ϕ_b	AASHTO LRFD Reference	Factored Bearing Resistance (ksf)
Service	1.0	Article 10.5.5.1	5.0
Strength	0.45	Table 10.5.5.2.2-1	12.5

7.1.5 Modulus of Subgrade Reaction

A modulus of subgrade reaction (k_s) equal to 140 pounds per cubic inch shall be used for the structural design of the box culvert's base slab. Calculations are included in Appendix C – Calculations.

7.2 Settlement

The proposed box culvert will bear on dense glacial till. These materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment will approximately match the existing. As a result, little to no increase in vertical overburden pressure is anticipated and any settlement is anticipated to be small and will occur quickly as load is applied to the structure.

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, Orono has a design freezing index (DFI) of approximately 1750 F-degree days. A water content of 10 percent was used for coarse-grained soils. These components correlate to a frost depth of 7.4 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 7.4 feet for frost protection. However, there are no requirements for embedment of precast concrete boxes at stream crossings in the BDG. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.4 Scour and Riprap

Where required, slopes shall be armored with 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 conforming to MaineDOT Standard Specification 722.03 – 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 2H:1V extending from the edge of the roadway down to the existing ground surface.

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.

8.0 CONSTRUCTION CONSIDERATIONS

Excavations for culvert foundations are anticipated to extend approximately 13 feet below existing grades. The foundation subgrade are primarily anticipated to consist of natural glacial till, and weathered rock and bedrock beneath portions of the proposed culvert footprint. Open cut excavation techniques are anticipated to be suitable for this project. Conventional cantilever sheet pile support is not feasible due to the shallow bedrock. If excavation support is needed, it would require either

drilled elements into bedrock (e.g., socketed soldier piles or drilled toe pins) or internal bracing.

Damming and diversion and/or temporary dewatering are anticipated to be necessary to control groundwater and/or river inflow in excavations. Depending on permitting and water levels at the time of construction, we anticipate that it would be possible to dam the brook with sandbags and an impermeable membrane, and temporarily divert the flow through a pipe so the contractor can construct foundations in the dry. It may also be necessary to employ localized pumping from sumps to maintain dewatering. It is anticipated that inflow of surface water or runoff to excavations can be handled by open pumping from sumps installed at the bottoms of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices. Regardless of the method of excavation, all excavations and earth support systems shall meet all applicable OSHA regulations.

The approximate base elevation at the culvert inlet and outlet are El. 110.4 and 109.6 respectively. Based on the results of the boring and probe, weathered rock is estimated at elevation El. 107, and competent bedrock is estimated at elevations ranging from El. 106 to 108, at the location of the upstream and downstream explorations respectively. Therefore, the materials encountered at the box subgrade elevation are likely to consist of dense Silty Gravel (Glacial Till) with areas of weathered or competent bedrock. Placing the precast box on dissimilar materials is to be avoided. Isolated points of contact of the bottom slab and bedrock or cobbles can create localized bearing stress. To mitigate these concerns, high points in the bedrock surface should be excavated down to allow for all box segments to be installed on the 12-inch-thick mat of bedding material.

The bedrock surface, where encountered, shall be prepared in accordance with MaineDOT standard practices. The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the excavation for the precast concrete box culvert is made. Construction activities should not be permitted to create any open fissures in the bedrock to remain. Any irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with crushed stone to the bottom of the required bedding material. Considering the limited extent of bedrock removal, blasting is not recommended, and bedrock should be removed by mechanical methods (e.g., hoe ramming).

Any unsuitable soils (i.e. low strength silts, loose sands, and/or organic material) that may be encountered at subgrade elevation should be excavated down to expose competent, firm material and replaced with compacted granular borrow. Prior to placing and compacting the granular borrow bedding layer, the subgrade shall be proof compacted where it consists of soil. Rock should be removed to a depth of at least 1 foot below the base of the culvert and toe walls.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary 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.

Using the excavated native soils as backfill around the culvert shall not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the vicinity of the culvert. These materials should not be used to re-base the roadway. Excavated subbase sand and gravel may be used as fill below roadway subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Highway Program for specific application to the proposed replacement of an existing culvert (#914698) under Forest Avenue in Orono, 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 a limited subsurface investigation at discrete exploratory location 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 a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Sheets



ORONO, MAINE

Project Location

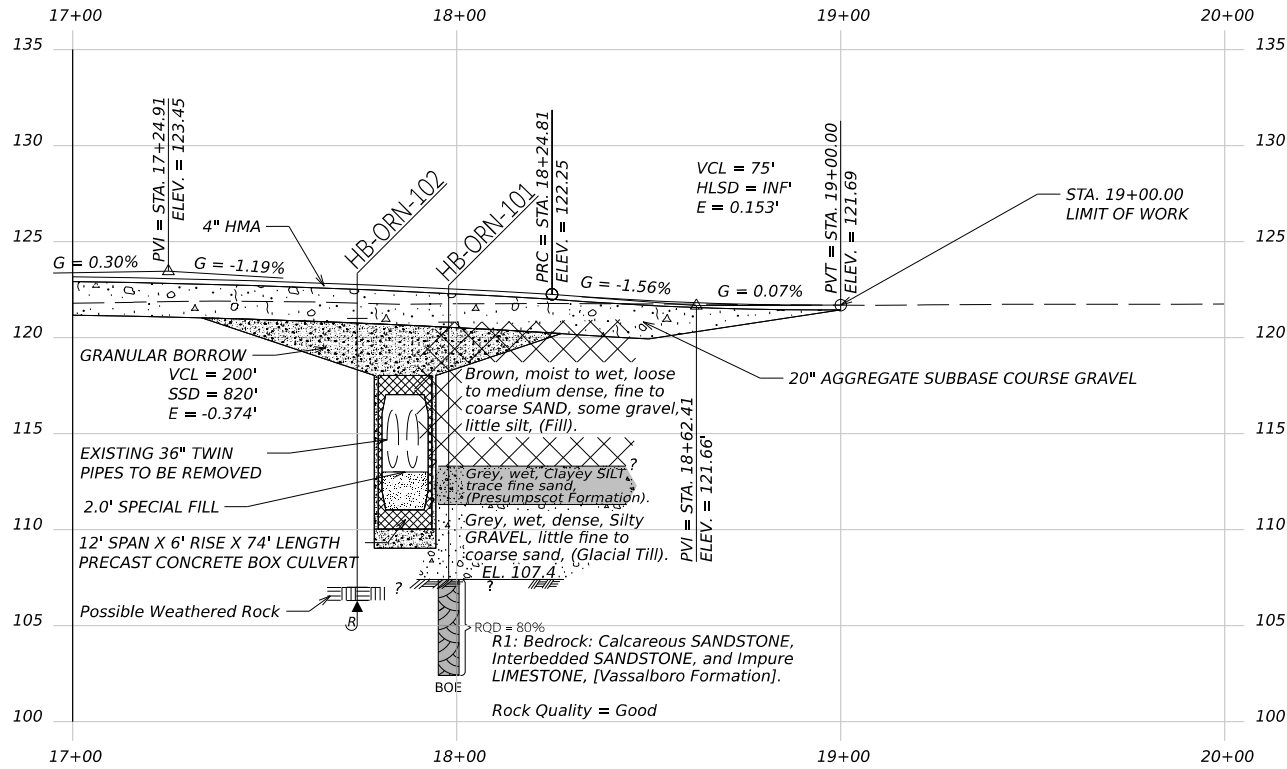
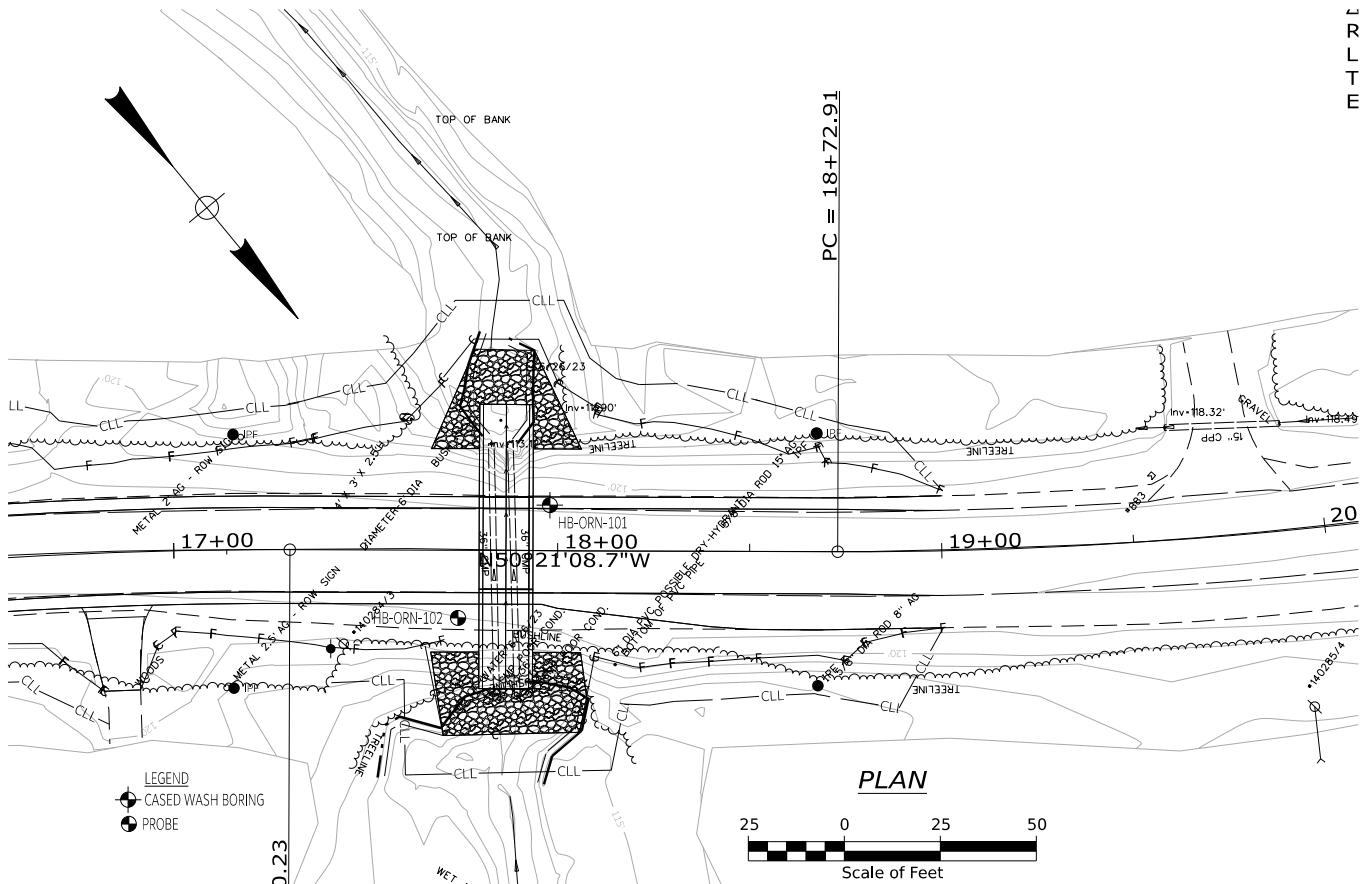


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1 inch = 0.14 miles

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SHEET NUMBER 1 OF 2	ORONO FOREST AVENUE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
		27196.00	
	LOCATION MAP	WIN 27196.00	HIGHWAY 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

Maine Department of Transportation				Project: Large Culvert Replacement on Forest Avenue Location: Orono, Maine		Boring No.: HB-ORN-101	
Soil/Rock Exploration Log				US CUSTOMARY UNITS		WIN: 27196.00	
Driller: MaheDOT		Elevation (ft.): 120.8		Auger ID/OD: 5" Solid Stem			
Operator: Daggett/Andrie		Datum: NAVD88		Sampler: Standard Split Spoon			
Logged By: B. W. Lior		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 10/23/2023-10/23/2023		Drilling Method: Cased Wash Boring		Core Barrels: MD-2"			
Boring Location: 17+47.5, 11.3 ft Lt.		Coating ID/OD: 16-3"		Water Level: 5.8 ft bgs.			
Hammer Efficiency Factors 0.906		Hammer Type: Automatic 88		Hydraulic 10		Rope & Cathode 10	
Soil/Rock Sample		Soil/Rock Sample		Soil/Rock Sample		Soil/Rock Sample	
S = Split Stem Sample		S = Split Stem Sample		S = Split Stem Sample		S = Split Stem Sample	
W = Unsuccessful Split Stem Sample Attempt		W = Unsuccessful Split Stem Sample Attempt		W = Unsuccessful Split Stem Sample Attempt		W = Unsuccessful Split Stem Sample Attempt	
U = Unsuccessful Split Stem Sample		U = Unsuccessful Split Stem Sample		U = Unsuccessful Split Stem Sample		U = Unsuccessful Split Stem Sample	
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P = Pushed Penetration (PPC) - Weight of Rod or Coating		P = Pushed Penetration (PPC) - Weight of Rod or Coating		P = Pushed Penetration (PPC) - Weight of Rod or Coating		P = Pushed Penetration (PPC) - Weight of Rod or Coating	
R = Refusal at 100 lb. Hammer		R = Refusal at 100 lb. Hammer		R = Refusal at 100 lb. Hammer		R = Refusal at 100 lb. Hammer	
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Appendix A

Boring Logs

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

MODIFIED BURMISTER SYSTEM			
<u>Descriptive Term</u>		<u>Portion of Total (%)</u>	
trace		0 - 10	
little		11 - 20	
some		21 - 35	
adjective (e.g. Sandy, Clayey)		36 - 50	
TERMS DESCRIBING DENSITY/CONSISTENCY			
<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).			
<u>Density of Cohesionless Soils</u>		<u>Standard Penetration Resistance</u> N ₆₀ -Value (blows per foot)	
Very loose		0 - 4	
Loose		5 - 10	
Medium Dense		11 - 30	
Dense		31 - 50	
Very Dense		> 50	
<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.			
<u>Consistency of Cohesive soils</u>		<u>SPT N₆₀-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>
Very Soft		WOH, WOR, WOP, <2	0 - 250
Soft		2 - 4	250 - 500
Medium Stiff		5 - 8	500 - 1000
Stiff		9 - 15	1000 - 2000
Very Stiff		16 - 30	2000 - 4000
Hard		>30	over 4000
<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>Rock Quality Designation (RQD):</u>			
RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^* > 4 \text{ inches}}{\text{length of core advance}}$			
*Minimum NQ rock core (1.88 in. OD of core)			
<u>Rock Quality Based on RQD</u>			
<u>Rock Quality</u>		<u>RQD (%)</u>	
Very Poor		≤25	
Poor		26 - 50	
Fair		51 - 75	
Good		76 - 90	
Excellent		91 - 100	
<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 quality (very poor, poor, etc.)			
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12			
Recovery (inch/inch and percentage)			
Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
<u>Sample Container Labeling Requirements:</u>			
WIN		Blow Counts	
Bridge Name / Town		Sample Recovery	
Boring Number		Date	
Sample Number		Personnel Initials	
Sample Depth			

Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Large Culvert Replacement on Forest Avenue Location: Orono, Maine				Boring No.: HB-ORN-101 WIN: 27196.00																																																																																																																																																																																																																																																																																																																																																	
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Operator: Daggett/Andrle				Datum: NAVD88				Sampler: Standard Split Spoon																																																																																																																																																																																																																																																																																																																																																	
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																																																																																																																	
Date Start/Finish: 10/23/2023; 12:30-14:00				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"																																																																																																																																																																																																																																																																																																																																																	
Boring Location: 17+97.9, 11.9 ft Lt.				Casing ID/OD: NW-3"				Water Level*: 5.8 ft bgs..																																																																																																																																																																																																																																																																																																																																																	
Hammer Efficiency Factor: 0.906				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 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																																																																																																																																																																																																																																																																																																																																																									
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="7">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) 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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Large Culvert Replacement on Forest Aveune Location: Orono, Maine				Boring No.: HB-ORN-102 WIN: 27196.00			
Drilling Contractor: MaineDOT				Elevation (ft.): 121.0				Auger ID/OD: 5" Dia.			
Operator: Daggett/Andrle				Datum: NAVD88				Sampler: N/A			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: N/A			
Date Start/Finish: 10/23/2023-10/23/2023				Drilling Method: Solid Stem Auger				Core Barrel: N/A			
Boring Location: 17+74.1, 17.6 ft Rt.				Casing ID/OD: N/A				Water Level*: None Observed			
Definitions: D = Spilt Spoon Sample MU = Unsuccessful Thin Wall Tube Sample Attempt WO1P = Weight of 1 Person S = Sample off Auger Flights R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) B = Bucket Sample off Auger Flights SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) LL = Liquid Limit MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) PL = Plastic Limit U = Thin Wall Tube Sample RC = Roller Cone N-value = Raw Field SPT N-value G = Grain Size Analysis MV = Unsuccessful Field Vane Shear Test Attempt WOH = Weight of 140lb. Hammer T_v = Pocket Torvane Shear Strength (psf) V = Field Vane Shear Test PP= Pocket Penetrometer WOR/C = Weight of Rods or Casing WC = Water Content, percent ≐ = Similar or Equal too C = Consolidation Test											
Depth (ft.)	Sample Information								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-value	Casing Blows	Elevation (ft.)	Graphic Log			
0						SSA			Probe, no material samples taken.		
5											
10											
15									Possible Weathered Rock from 14.0-15.3 ft bgs.		
20									Bottom of Exploration at 15.3 feet below ground surface. REFUSAL		
25											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.											
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											
Page 1 of 1											
Boring No.: HB-ORN-102											

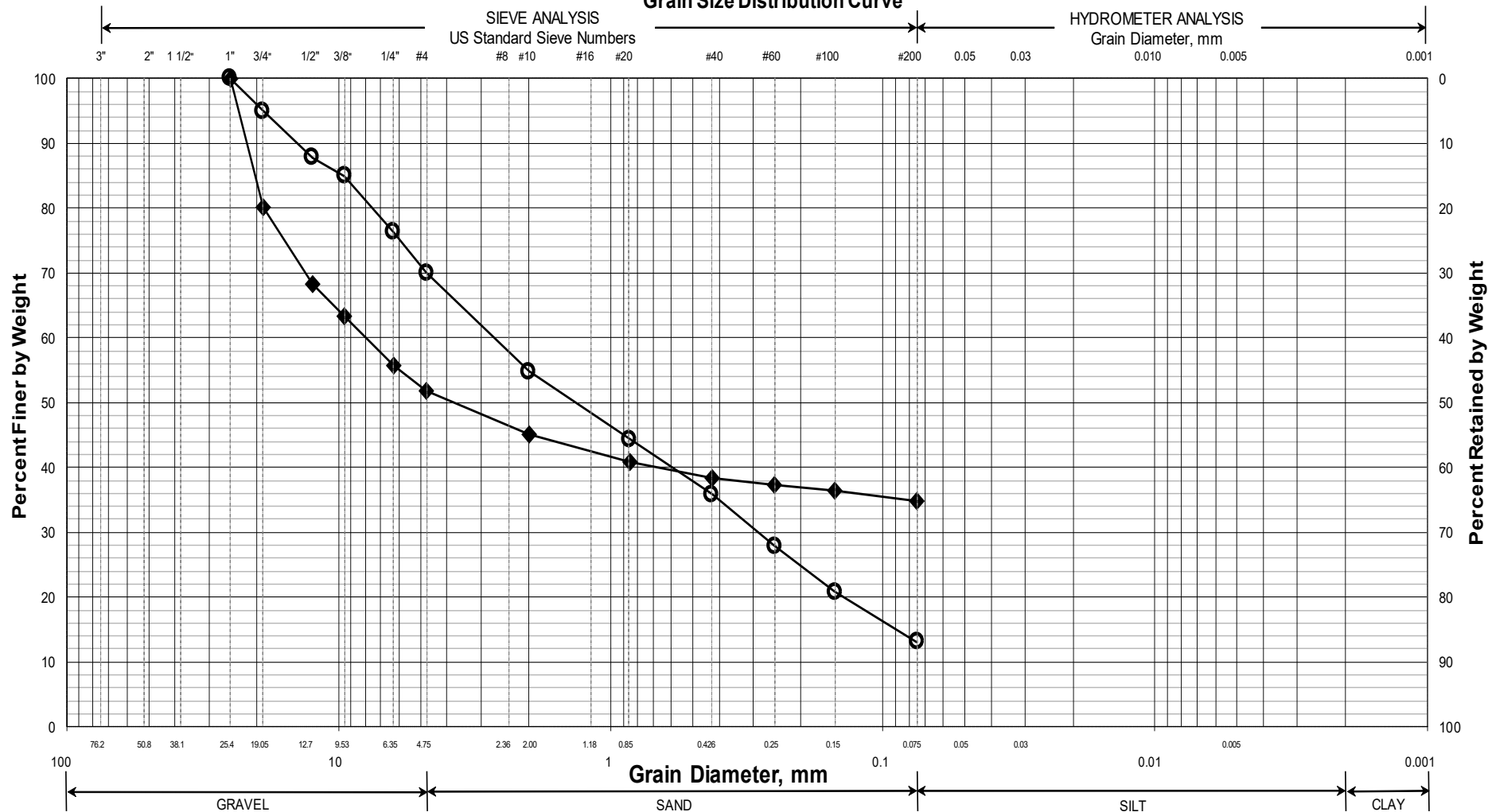
Appendix B

Laboratory Test Results

Work Number: 27196.00

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
O	HB-ORN-101/1D	17+97.9	11.9 LT	0.0-2.0	SAND, some gravel, little silt.	7.2			
◆	HB-ORN-101/3D	17+97.9	11.9 LT	10.0-12.0	Silty GRAVEL, little sand.	10.1			
■									
●									
▲									
X									

WIN	
027196.00	
Town	
Orono	
Reported by/Date	
WHITE, TERRY A	5/1/2026



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **379690** Boring No./Sample No. **HB-ORN-101/1D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **10/23/2023** Received **1/23/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **17+97.9** Offset, ft: **11.9** LT Dbfg, ft: **0.0-2.0**

WIN/Town **027196.00 - ORONO** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	95.0
½ in. [12.5 mm]	87.8
⅜ in. [9.5 mm]	85.0
¼ in. [6.3 mm]	76.3
No. 4 [4.75 mm]	70.0
No. 10 [2.00 mm]	54.8
No. 20 [0.850 mm]	44.3
No. 40 [0.425 mm]	35.8
No. 60 [0.250 mm]	27.9
No. 100 [0.150 mm]	20.8
No. 200 [0.075 mm]	13.1

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	7.2

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **1/25/2024**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **379691** Boring No./Sample No. **HB-ORN-101/3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **10/23/2023** Received **1/23/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **17+97.9** Offset, ft: **11.9** LT Dbfg, ft: **10.0-12.0**

WIN/Town **027196.00 - ORONO** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	80.2
½ in. [12.5 mm]	68.2
⅜ in. [9.5 mm]	63.4
¼ in. [6.3 mm]	55.8
No. 4 [4.75 mm]	51.8
No. 10 [2.00 mm]	45.1
No. 20 [0.850 mm]	40.8
No. 40 [0.425 mm]	38.4
No. 60 [0.250 mm]	37.3
No. 100 [0.150 mm]	36.4
No. 200 [0.075 mm]	34.8

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	10.1

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**Date Reported: **1/26/2024**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

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

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

$$K_o = 0.47$$

Fang, Foundation
Engineering Handbook
2nd ed. Pg. 224, Eq. 6.2
Formula for normally
consolidated soils.

Outlet walls free to rotate - Active Earth Pressure - Rankine Theory

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

For cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

6.1 AT-REST LATERAL PRESSURES

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

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

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

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

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

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

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

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

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

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

^a Unloading cycle.

be enough to know that the K_0 during reloading falls about halfway between that for unloading and normally consolidated conditions. Also, K_0 might be directly determined through in-situ testing methods.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

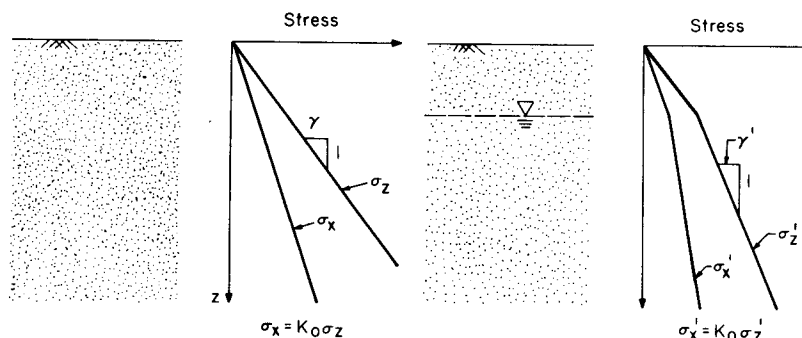


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

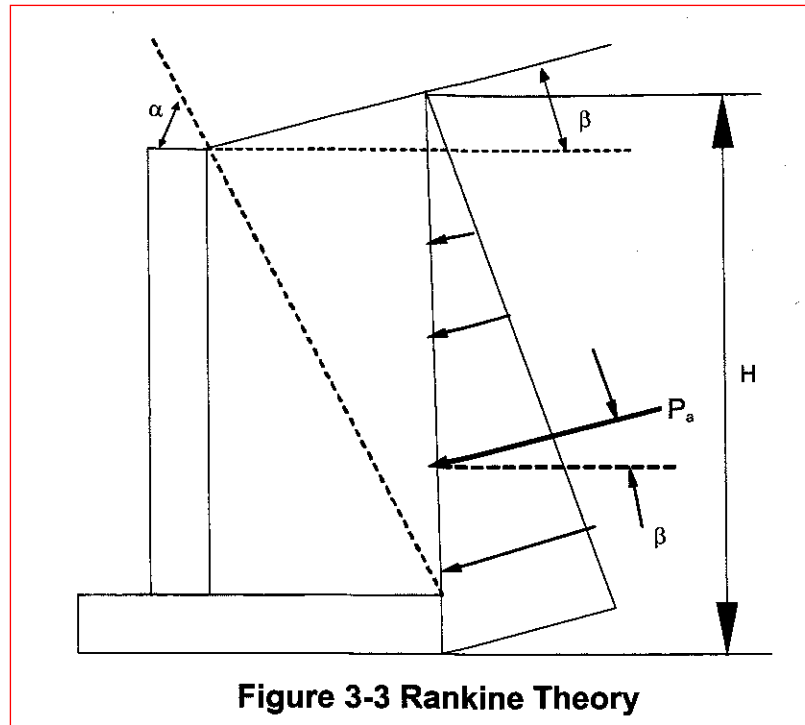
where:

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

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

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

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



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

Bearing Resistance - Existing Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - Box Culvert on Glacial Till

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 10th Edition 2024

Table C10.6.2.5.1-1 Presumptive Bearing Resistances for Spread Footings at the

Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Glacial Till (dense Silty Gravel), conservatively use dense silty sand values

Based on N-values, soils are dense near the bearing elevation

Density In Place: medium dense to dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use:

$$q_{nom} := 5 \cdot \text{ksf}$$

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1)

$$\phi_{\text{service_bc}} := 1.0$$

$$q_{\text{factored_service_bc}} := q_{nom} \cdot \phi_{\text{service_bc}}$$

$$q_{\text{factored_service_bc}} = 5 \cdot \text{ksf}$$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - Box Culvert on Glacial Till

Reference: AASHTO LRFD Bridge Design Specifications 10th Edition 2024 - Article 10.6.3.1

Assumptions:

1. The box will be founded at ~ Elev 111.43 feet

Bottom of Construction will be 2 feet below box invert

$$D_{\text{footing}} := 2.0 \cdot \text{ft}$$

2. Assumed parameters for glacial till:

Saturated unit weight: $\gamma_s := 135 \cdot \text{pcf}$

Internal friction angle: $\phi_{ns} := 33 \cdot \text{deg}$

Undrained shear strength: $c_{ns} := 0 \cdot \text{psf}$

3. Box Culvert parameters

Width of box culvert, B $B_{\text{box}} := 15 \cdot \text{ft}$

Length of box culvert, L $L_{\text{box}} := 74 \cdot \text{ft}$

Nominal Bearing Resistance per LRFD Equation 10.6.3.1.2a-1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{ym} C_{wy}$$

Bearing Capacity Factors - LRFD Table 10.6.3.1.2a-1

$$\text{For } \phi=33 \text{ deg} \quad N_c := 38.6 \quad N_q := 26.1 \quad N_\gamma := 35.2$$

Shape Correction Factors LRFD Table 10.6.3.1.2a-3

for $\phi=33$ degrees

$$s_c := 1 + \left(\frac{B_{\text{box}}}{L_{\text{box}}} \right) \left(\frac{N_q}{N_c} \right) \quad s_c = 1.14$$

$$s_\gamma := 1 - 0.4 \left(\frac{B_{\text{box}}}{L_{\text{box}}} \right) \quad s_\gamma = 0.9189$$

$$s_q := 1 + \left(\frac{B_{\text{box}}}{L_{\text{box}}} \cdot \tan(\phi_{\text{ns}}) \right) \quad s_q = 1.13$$

Load Inclination Factors:

Assume all are 1.0 (LRFD Article C10.6.3.1.2a)

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

Depth Correction

Factor

$$d_q := 1 + 2 \cdot \tan(\phi_{\text{ns}}) \cdot (1 - \sin(\phi_{\text{ns}}))^2 \cdot \tan\left(\frac{D_{\text{footing}}}{B_{\text{box}}}\right)^{-1}$$

$$d_q = 3.0079$$

LRFD Eq.
10.6.3.1.2a-10

$$N_{\text{cm}} := N_c \cdot s_c \cdot i_c \quad N_{\text{cm}} = 43.8905 \quad \text{LRFD Eq. 10.6.3.1.2a-2}$$

$$N_{\text{qm}} := N_q \cdot s_q \cdot d_q \cdot i_q \quad N_{\text{qm}} = 88.84 \quad \text{LRFD Eq. 10.6.3.1.2a-3}$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \quad N_{\gamma m} = 32.35 \quad \text{LRFD Eq. 10.6.3.1.2a-4}$$

Coefficients for Groundwater Depths LRFD Table 10.6.3.1.2a-2

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

$$\text{Depth the water table: } D_w := 0 \cdot \text{ft} \quad C_{wq} := 0.5 \quad C_{w\gamma} := 0.5$$

$$q_{\text{nominal}} := c_{\text{ns}} \cdot N_{\text{cm}} + \gamma_s \cdot D_{\text{footing}} \cdot N_{\text{qm}} \cdot C_{wq} + 0.5(\gamma_s) B_{\text{box}} \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$$q_{\text{nominal}} = 28.4 \cdot \text{ksf}$$

Factored Bearing Resistance for Strength Limit State

$$\text{Resistance Factor: } \phi_b := 0.45 \quad \text{LRFD Table 10.5.5.2.2-1}$$

$$q_{\text{factored}} := q_{\text{nominal}} \cdot \phi_b$$

$$q_{\text{factored}} = 12.8 \cdot \text{ksf}$$

Recommend a limiting factored bearing resistance of 12.5 ksf for the Strength Limit State.

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.

Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

10.6.2.5.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

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
		Theoretical method (Munfakh et al., 2001), in sand, using CPT
		Theoretical method (Munfakh et al., 2001), in sand, using SPT
		Semi-empirical methods (Meyerhof, 1957), all soils
		Footings on rock
		Plate Load Test
Sliding	ϕ_τ	Precast concrete placed on sand
		Cast-in-place concrete on sand
		Cast-in-place or precast concrete on clay
		Soil on soil
	ϕ_{ep}	Passive earth pressure component of sliding resistance

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.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3—Driven Piles

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

C10.5.5.2.3

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method (see Articles 10.7.3.8.2, 10.7.3.8.3, 10.7.3.8.4, or 10.7.3.8.5). The actual penetration depth where the pile is stopped using the driving criterion (e.g., a blow count measured during pile driving) will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the nominal pile bearing resistance is dependent on the reliability of the method used to verify the nominal resistance during pile installation (see Allen,

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

C10.6.3.1.2a

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a-1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a-2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a-3)$$

$$N_{\gamma m} = N_{\gamma} s_{\gamma} i_{\gamma} \quad (10.6.3.1.2a-4)$$

where:

- c = cohesion, taken as undrained shear strength (ksf)
- N_c = cohesion term (undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)
- N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 through 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

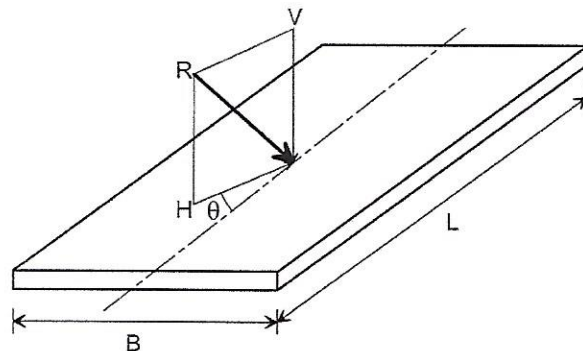


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c , s_γ , s_q	$\phi_f = 0$	$1 + \left(\frac{B}{5L} \right)$	1.0	1.0
	$\phi_f > 0$	$1 + \left(\frac{B}{L} \right) \left(\frac{N_c}{N_c} \right)$	$1 - 0.4 \left(\frac{B}{L} \right)$	$1 + \left(\frac{B}{L} \tan \phi_f \right)$

$$d_q = 1 + 2 \tan \phi_f (1 - \sin \phi_f)^2 \arctan \left(\frac{D_f}{B} \right) \quad (10.6.3.1.2a-10)$$

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle, ϕ_f , of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

where:

- d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation(dim)
- ϕ_f = angle of internal friction of soil (degrees)
- D_f = footing embedment depth (ft)
- B = footing width (ft)

Arctan (D_f/B) is in radians.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. The depth correction factor, d_q , shall not exceed 1.4.

10.6.3.1.2b—Considerations for Punching Shear

C10.6.3.1.2b

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b-1)$$

$$\phi^* = \tan^{-1}(0.67 \tan \phi_f) \quad (10.6.3.1.2b-2)$$

where:

- c^* = reduced effective stress soil cohesion for punching shear (ksf)
- ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

Modulus of Subgrade Reaction

Objective:

Estimate the modulus of subgrade reaction for the box culvert base slab design.

Given:

1. Limited lab data and SPT N-values.

Assumptions:

1. The proposed bearing elevation of base slab is approximately 110 feet.
2. Proposed finished roadway grade is approximately 122.7.
3. Proposed precast concrete box is 15 feet wide and approximately 74 feet long.
4. The subsurface conditions present at the proposed bearing elevation is glacial till comprised of dense Silty Gravel, little sand. (Modeled as medium dense to dense sand)
5. The bottom of the box culvert will be submerged for the structure's design life.

Published values of subgrade modulus

Published values of subgrade modulus in medium dense to dense sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:

Range of modulus of subgrade reaction for medium dense sand: 35 to 295 pci

Choose upper limit: $k_s = 295$ pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:

Saturated, dense, coarse grained glacial till, K_{v1} , $259 \text{ pci} / 2 = 129.5 \text{ pci}$

Das Principles of Foundation Engineering, 7th ed. Table 6.2:

Typical subgrade reaction values for 0.3 m x 0.3 m plate

Saturated sand, medium dense, 129 - 147 pci, use upper limit: $k_{0.3} (k_1) = 147 \text{ pci}$

Adjust Published values for dimensions of base slab

Published range for medium dense to dense sand subgrade is: 129.5-295

Assume a subgrade modulus of 191 pci, average of 129.5, 147, and 295 pci.

Value of k_{s1} = 191 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base
(Width B = 15 ft, Length L = 74 ft).

Square to rectangle base adjustment:

$$k_{s1} := 191 \text{ pci} \quad B := 15 \text{ ft} \quad \underset{\sim}{L} := 74 \text{ ft}$$

$$k := \frac{k_{s1} \cdot \left[1 + 0.5 \left(\frac{B}{L} \right) \right]}{1.5}$$

Das, Principles of Foundation
Engineering 7th Ed. P. 311 Eqn. 6.44

$$k = 140.2 \cdot \text{pci}$$

Recommend a subgrade modulus of 140 pci

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \quad (9-10)$$

for either horizontal or vertical members

for depth variation

interest below ground

to give k_s the best fit (if load test or other data are available)

variation may be zero; at the ground surface A_s is zero for a lateral k_s .
 > 0 . For footings and mats (plates in general), $A_s > 0$ and $B_s \approx 0$.
 used with the proper interpretation of the bearing-capacity equations (the d_i factors dropped) to give

$$q_{ult} = cN_{cs} + \gamma ZN_{qs} + 0.5\gamma BN_{\gamma s} \quad (9-10a)$$

$$cN_{cs} + 0.5\gamma BN_{\gamma s} \quad \text{and} \quad B_s Z^1 = C(\gamma N_{qs})Z^1$$

to estimate k_s . In these equations the Terzaghi or Hansen bearing capacity factors are used. The C factor is 40 for SI units and 12 for Fps, using the same values for A_s and B_s at a 0.0254-m and 1-in. settlement but with no SF, since this equation is used where there is concern that k_s does not increase without bound with increasing Z . The $B_s Z$ term by one of two simple methods:

$$\text{Method 1: } B_s \tan^{-1} \frac{Z}{D}$$

$$\text{Method 2: } \frac{B_s}{D^n} Z^n = B'_s Z^n$$

depth of interest, say, the length of a pile

depth of interest

estimate of the exponent

to estimate a value of k_s to determine the correct order of magnitude of n . This can be obtained using one of the approximations given here. Obviously if a value of k_s is three times larger than the table range indicates, the computations will have a possible gross error. Note, however, if you use a reduced value of k_s (or 12 mm) instead of 0.0254 m you may well exceed the table range. If a computational error (or a poor assumption) is found then use judgment to adjust the table values are intended as guides. The reader should not use, say, a value of k_s even as a "good" estimate.

shown in Fig. 9-9c (and used in your diskette program FADBEMLP as described) is estimated at some small value of, say, 6 to 25 mm, or from inspection of a load test was done. It might also be estimated from a triaxial compression test or at the maximum pressure from the stress-strain plot.

compute

$$X_{max} = \epsilon_{max}(1.5 \text{ to } 2B)$$

TABLE 9-1

Range of modulus of subgrade reaction k_s

reaction k_s

Use values as guide and for comparison when using approximate equations

$$\frac{kN}{M^3} \rightarrow \frac{lb}{in^3} : \frac{224.8 lb}{1 kN} * \frac{1 M^3}{61023.7 in^3} = .003684 \frac{kN}{M^3} = 1 \frac{lb}{in^3}$$

Soil	k_s , kN/m ³	k_s , lb/in ³
Loose sand	4800-16,000	18 - 59
Medium dense sand	9600-80,000	35 - 295
Dense sand	64,000-128,000	236 - 472
Clayey medium dense sand	32,000-80,000	118 - 295
Silty medium dense sand	24,000-48,000	88 - 177
Clayey soil:		
$q_a \leq 200$ kPa	12,000-24,000	44 - 88
$200 < q_a \leq 800$ kPa	24,000-48,000	88 - 177
$q_a > 800$ kPa	$> 48,000$	> 177

295 pci

The 1.5 to 2B dimension is an approximation of the depth of significant stress-strain influence (Boussinesq theory) for the structural member. The structural member may be either a footing or a pile.

Example 9-5. Estimate the modulus of subgrade reaction k_s for the following design parameters:

$$B = 1.22 \text{ m} \quad L = 1.83 \text{ m} \quad D = 0.610 \text{ m}$$

$$q_a = 200 \text{ kPa (clayey sand approximately 10 m deep)}$$

$$E_s = 11.72 \text{ MPa (average in depth } 5B \text{ below base)}$$

Solution. Estimate Poisson's ratio $\mu = 0.30$ so that

$$E'_s = \frac{1 - \mu^2}{E_s} = \frac{1 - 0.3^2}{11.72} = 0.07765 \text{ m}^2/\text{MN}$$

For center:

$$H/B' = 5B/(B/2) = 10 \text{ (taking } H = 5B \text{ as recommended in Chap. 5)}$$

$$L/B = 1.83/1.22 = 1.5$$

From these we may write

$$I_s = 0.584 + \frac{1 - 2(0.3)}{1 - 0.3} (0.023) = 0.597$$

using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.

At $D/B = 0.61/1.22 = 0.5$, we obtain $I_F = 0.80$ from Fig. 5-7 (or when using FFACTOR for the I_s factors). Substitution into Eq. (9-7) with $B' = 1.22/2 = 0.61$, and $m = 4$ yields

$$k_s = \frac{1}{0.61(0.07765)(4 \times 0.597)(0.8)} = 11.05 \text{ MN/m}^3$$

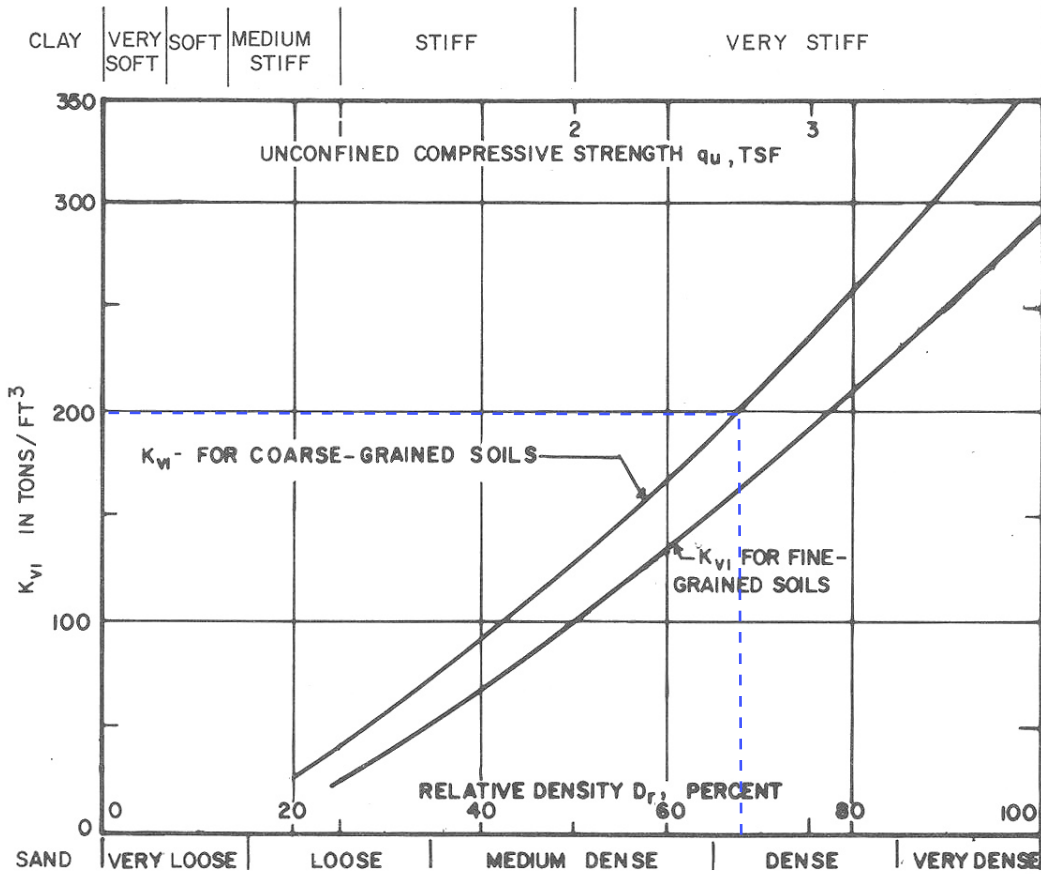
You should note that k_s does not depend on the contact pressure of the base q_o .

For corner:

$$H/B' = 5B/B = 5(1.22)/1.22 = 5$$

[from Table 5-2 with $L/B = 1.5$ obtained for Eq. (5-16)]

200 TCF =
259 PCI
Reduce per
Note 2



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
 q = FOOTING UNIT LOAD IN tsf
 B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE

K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

$$\frac{\text{ton}}{\text{ft}^3} \rightarrow \frac{\text{lb}}{\text{in}^3} = \frac{2000 \text{ lb}}{1 \text{ ton}} * \frac{1 \text{ ft}^3}{1728 \text{ in}^3} = 1.157 \frac{\text{ton}}{\text{ft}^3} \rightarrow 1 \frac{\text{lb}}{\text{in}^3}$$

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)
 SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20 \text{ FT}$:

$$\Delta H_i = \frac{4 q B^2}{K_{vi} (B+1)^2}$$

FOR $B \geq 40 \text{ FT}$:

$$\Delta H_i = \frac{2 q B^2}{K_{vi} (B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20 \text{ FT}$:

$$\Delta H_i = \frac{2 q B^2}{K_{vi} (B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.

2. VALUES OF K_{vi} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST $1.5B$ BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{vi}/2$ IN COMPUTING SETTLEMENT

Figure 8-3: Modulus of Subgrade Reaction (NAVFAC, 1986a)

Equation (6.44) indicates that the value of k for a very long foundation with a width B is approximately $0.67k_{(B \times B)}$.

The modulus of elasticity of granular soils increases with depth. Because the settlement of a foundation depends on the modulus of elasticity, the value of k increases with the depth of the foundation.

Table 6.2 provides typical ranges of values for the coefficient of subgrade reaction, $k_{0.3}(k_1)$, for sandy and clayey soils.

For long beams, Vesic (1961) proposed an equation for estimating subgrade reaction, namely,

$$k' = Bk = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{1 - \mu_s^2}$$

or

$$k = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{B(1 - \mu_s^2)} \tag{6.45}$$

where

- E_s = modulus of elasticity of soil
- B = foundation width
- E_F = modulus of elasticity of foundation material
- I_F = moment of inertia of the cross section of the foundation
- μ_s = Poisson's ratio of soil

$$\frac{MN}{m^3} \rightarrow \frac{lb}{in^3} : \frac{224809 lb}{1 MN} * \frac{1 m^3}{61024 in^3} \rightarrow 3.684 \frac{lb}{in^3} = \frac{1 MN}{M^3}$$

Table 6.2 Typical Subgrade Reaction Values, $k_{0.3}(k_1)$

Soil type	$k_{0.3}(k_1)$ MN/m ³	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

The unit of k is kN/m^3 . The value of the coefficient of subgrade reaction is not a constant for a given soil, but rather depends on several factors, such as the length L and width B of the foundation and also the depth of embedment of the foundation. A comprehensive study by Terzaghi (1955) of the parameters affecting the coefficient of subgrade reaction indicated that the value of the coefficient decreases with the width of the foundation. In the field, load tests can be carried out by means of square plates measuring $0.3 \text{ m} \times 0.3 \text{ m}$, and values of k can be calculated. The value of k can be related to large foundations measuring $B \times B$ in the following ways:

Foundations on Sandy Soils

For foundations on sandy soils,

$$k = k_{0.3} \left(\frac{B + 0.3}{2B} \right)^2 \quad (6.42)$$

where $k_{0.3}$ and k = coefficients of subgrade reaction of foundations measuring $0.3 \text{ m} \times 0.3 \text{ m}$ and $B \text{ (m)} \times B \text{ (m)}$, respectively (unit is kN/m^3).

Foundations on Clays

For foundations on clays,

$$k (\text{kN/m}^3) = k_{0.3} (\text{kN/m}^3) \left[\frac{0.3 \text{ (m)}}{B \text{ (m)}} \right] \quad (6.43)$$

The definitions of k and $k_{0.3}$ in Eq. (6.43) are the same as in Eq. (6.42).

For rectangular foundations having dimensions of $B \times L$ (for similar soil and q),

$$k = \frac{k_{(B \times B)} \left(1 + 0.5 \frac{B}{L} \right)}{1.5} \quad (6.44)$$

Method 1:

where

k = coefficient of subgrade modulus of the rectangular foundation ($L \times B$)
 $k_{(B \times B)}$ = coefficient of subgrade modulus of a square foundation having dimension of $B \times B$

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: **Orono, Maine**

DFI = 1750 degree-days.

Case 1 - coarse grained soils W=7.2% (HB-ORN-101 1D).

For DFI = 1700
at w=10%

$$d_1 := 87.5 \text{ in}$$

For DFI = 1800
at w=10%

$$d_2 := 90.1 \text{ in}$$

Depth of Frost Penetration

$$d := \frac{d_1 + d_2}{2} = 7.4 \text{ ft}$$

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Figure 5-1 Maine Design Freezing Index Map

