MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MULLIGAN STREAM BRIDGE
NOKOMIS ROAD OVER MULLIGAN STREAM
CORINNA, MAINE

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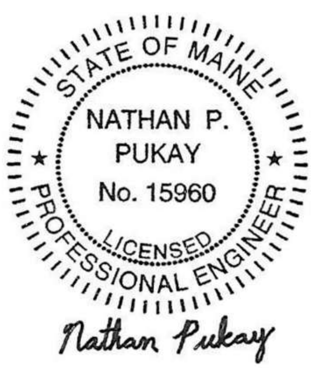

Penobscot County
WIN 26476.00

Soils Report 2026-33
Bridge No. 6160

Federal Project No. 02647600
April 24, 2026

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Appendices

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Mulligan Stream Bridge which carries Nokomis Road over Mulligan Stream in Corinna, 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 Mulligan Stream Bridge was constructed in 1968, replacing a wood-planked steel girder bridge founded on stone abutments on log grillage. The current structure consists of a single 16-foot 7-inch span by 10-foot 1-inch rise, steel plate pipe arch bearing on 1-foot of gravel borrow. According to the 2025 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge is structurally deficient. The culvert is rated a 4 (poor condition) due to holes, large rust nodules, and unzipping between plate sections.

The proposed replacement structure is a 18-foot span by 11-foot rise, 90-foot long, precast concrete box culvert. The box culvert shall have 1-foot-tall precast headwalls and 2-foot-deep toe walls at the inlet and outlet. The upstream and downstream ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) sideslopes. Riprap aprons will be installed at the inlet and outlet. The box shall be placed on a 2-foot-thick leveling layer of culvert bedding stone. Additional bedding stone may be required to replace soft or unsuitable soils encountered at the bearing subgrade.

The new box culvert will be located on nearly the same horizontal alignment with a roadway finish grade approximately matching the existing at the centerline of the structure. The existing bridge will be closed during construction and traffic detoured onto state-aid roads.

2.0 GEOLOGIC SETTING

Mulligan Stream Bridge carries Nokomis Road over Mulligan Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Reconnaissance Surficial Geology Map of the Pittsfield Quadrangle, Open-File No. 86-35 (1986), indicates the surficial soils in the vicinity of the bridge project consist of swamp and tidal-marsh deposits and glacial till. Swamp and tidal-marsh deposits consist of peat, silt, clay, sand. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones.

The MGS Reconnaissance Bedrock Geology of the Pittsfield Quadrangle, Open File No. 71-3 (1971) maps the bedrock in the vicinity of the project as interbedded phyllite and metasiltstone of the Waterville Formation.

3.0 SUBSURFACE INVESTIGATION

Three test borings explored subsurface conditions at the project location. Borings BB-CMS-101 and BB-CMS-101A were drilled west of the existing culvert, and boring BB-CMS-102 was drilled east of the existing culvert. The test boring locations are shown on Sheet 2 – Boring Location Plan.

The borings were drilled in January 2024 by S.W. Cole Explorations (now Seaboard Drilling LLC). Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

The borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically 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 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 2023. All N-values discussed in this report are corrected N-values computed by applying an average energy transfer of 1.066 to the raw field N-values. The hammer efficiency factor (1.066) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in the borings using NQ-2" core barrels and the Rock Quality Designation (RQD) of the cores were calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field-testing requirements, and logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program and then located by MaineDOT Survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of two standard grain size analyses with natural water content, one grain size analysis with hydrometer and natural water content, two Atterberg limits test, and one loss on ignition (organic content) test. The results of the soil tests are included as Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, Swamp Deposits and Glacial Till. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of fill was encountered in the test borings. The thickness of the fill unit was approximately 15 feet. The fill materials encountered consisted of:

- Brown to grey-brown, SAND, some silt, some gravel; and
- Brown, GRAVEL, some sand, some silt.

Corrected SPT N-values in the fill ranged from 9 to 28 blows per foot (bpf), indicating the fill is loose to medium dense in consistency.

One grain size analysis conducted on a sample of the fill indicated the material is classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 12 percent.

5.2 Swamp Deposit

A swamp deposit was encountered in BB-CMS-102 beneath the fill. The thickness encountered was approximately 3 feet. The deposit consisted of:

- Black, Silty PEAT; and
- Grey, Silty CLAY, trace sand.

One corrected SPT N-value within the swamp deposit was 2 bpf indicating the deposit is soft in consistency.

A grain size analysis could not be conducted due to the limited amount of recovered material.

One Atterberg limits test was conducted on a sample of the silty clay subunit of the swamp deposit, and is summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-CMS-102, 3D/B	Silty CLAY	38	37	23	14	1.05

The plasticity index of sample BB-CMS-102, 3D/B indicates that the silty clay subunit is medium in plasticity (Burmister, 1949). The natural water content of the same sample measured 38 percent. With a liquid limit of 37 and a plastic limit of 23, the resulting liquidity index exceeds 1.0.

Interpretation of these results indicates that the deposit has the potential to convert into a viscous fluid with the slightest disturbance. Soils with liquidity indices in excess of 1 have a high liquefaction or "quick" potential.

One loss on ignition test performed on a sample of the swamp deposit had an organic content of 50 percent. The natural water content of the two test samples recovered from the deposit were 38 and 292 percent.

5.3 Glacial Till

Glacial till was encountered in the test borings underlying the fill or swamp deposit. The thickness encountered was approximately 14 to 20 feet. The deposit consisted of:

- Grey, SILT, some sand, trace to little clay, trace gravel; and
- Grey, Sandy SILT, some gravel, trace clay.

Corrected SPT N-values in the glacial till ranged from 11 to greater than 50 bpf, indicating the deposit is stiff to hard in consistency.

One Atterberg limits test was conducted on a sample of the glacial till. The sample was determined to be non-plastic.

Two grain size analyses indicate the material is classified as A-4 under the AASHTO Soil Classification System and ML and SC-SM under the USCS. The natural water contents of the samples tested were 10 and 12 percent.

5.4 Bedrock

Bedrock was encountered and cored in the borings. The following table summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%), (R1, R2, R3)
BB-CMS-101A	8+44.6	7.6 Rt	35.7	177.2	0, 47, 20
BB-CMS-102	8+78.9	8.0 Lt	32.5	180.7	21, 33

The bedrock at the site is identified as grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh, high fissility, steep to vertical joints along foliation planes, closely spaced, with highly fractured zones, and some quartz or calcite infilling. The RQD of the bedrock ranged from 0 to 47 percent corresponding to a rock quality of very poor to poor. Detailed bedrock descriptions and RQD are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. Rock core photographs are included in Appendix B – Rock Core Photographs.

5.5 Groundwater

Groundwater was measured in the borings at a depth of 7 feet below the roadway surface upon completion of drilling. Note that water was introduced into the boreholes during drilling operations and the measured level may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Due to sufficient overburden at the bridge location, along with consultations with the MaineDOT environmental staff, a precast concrete box culvert was identified as the preferred bridge replacement alternative. 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 low cost.

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 should have 2-foot-tall inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 1-foot into the streambed. 2-foot-thick riprap aprons will be constructed at the inlet and outlet.

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 box culvert 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, 10^{th} Edition, 2024.

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the 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: $\phi = 32°$, $\gamma = 125$ pcf.

The box culvert will be bedded on a 2-foot-thick leveling layer of culvert bedding stone conforming to Special Provision 203 – Culvert Bedding Stone. The thickness of the bedding material shall be increased if over-excavation is required to remove soft or unsuitable soils. The bedding material will be placed on a stabilization/reinforcement geotextile lining the bottom and sides of the excavation and meeting the requirements of Standard Specification 722.01 – Stabilization/Reinforcement Geotextile. The excavation should be maintained so that the bedding layer and box culvert are constructed in-the-dry. 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 slope backfill and prevent material 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 and inlet walls will be slope-tapered at 2H:1V (maximum). The left and right outlet walls will share the same precast base slab. The sloped 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.

Inlet and 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 and free to rotate 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 D – Calculations for supporting calculations.

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. Culvert bedding material shall not be placed below the toe (seepage cutoff) walls.

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 2-foot-thick layer of culvert bedding stone with a slab bottom elevation of approximately El. 198.5.

For a precast concrete box culvert with a base width of 20 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 8.8 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 0.3*f'c*. See Appendix C – Calculations for supporting calculations.

7.1.5 Modulus of Subgrade Reaction

Large span precast box culverts can be viewed similarly to a mat foundation. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 75 pounds per cubic inch (pci).

7.2 Subgrade Preparation for Box Culvert

The soils encountered in the borings at the box subgrade elevation consisted primarily of stiff to hard glacial till and a soft swamp deposit. The swamp deposit, consisting of soft silt, clay, and peat is anticipated beneath portions of the culvert footprint and may extend up to 2 feet below the culvert bedding layer. These unsuitable soils shall be excavated to expose competent, firm material and replaced with compacted culvert bedding stone. Prior to placing and compacting the culvert bedding stone layer the subgrade shall be proof compacted and a stabilization/reinforcement geotextile deployed on the excavation subgrade. These recommendations should be included in the contract documents as a General Note.

7.3 Settlement

The proposed box culvert will bear on stiff to hard, cohesive, silty glacial till. Although the proposed reinforced concrete box culvert will be heavier than the existing steel plate pipe arch, the proposed span and rise are slightly larger than the existing culvert and the proposed vertical alignment will approximately match the existing. Therefore, any net increase in vertical stress imposed on the underlying soils is anticipated to be minimal. Any settlement is expected to be small and will occur quickly.

Considering the subgrade preparation recommendations in Section 7.2 of this report, all loose, soft or unsuitable soils encountered at the subgrade elevation will be excavated, replaced with compacted bedding stone and proof compacted. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

7.4 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, Corinna has a design freezing index (DFI) of approximately 1850 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 7.6 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 7.6 feet for frost protection.

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

7.5 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.

7.6 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

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 2-foot-thick layer of culvert bedding stone, conforming to Special Provision 203 – Culvert Bedding Stone. The bedding material will be placed on a stabilization/reinforcement geotextile lining the bottom and sides of the excavation and meeting the requirements of Standard Specification 722.01 – Stabilization/Reinforcement Geotextile. The culvert is expected to bear on the bedding stone underlain by stiff to hard glacial till. It is anticipated that a swamp deposit will be encountered below the culvert bearing elevation along a portion of the box culvert footprint and extend up to 2 feet below the bottom of the culvert bedding layer. Prior to placing the stabilization/reinforcement geotextile and culvert bedding material, all unsuitable clay, silt or organics from the swamp deposit shall be removed and replaced with culvert bedding stone.

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

Earthwork and excavations may result in the exposure of silty or other soft soils. These soils are 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. The contractor shall maintain the excavation so that the box culvert and culvert bedding layer are installed in-the-dry.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Mulligan Stream Bridge in Corinna, 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.

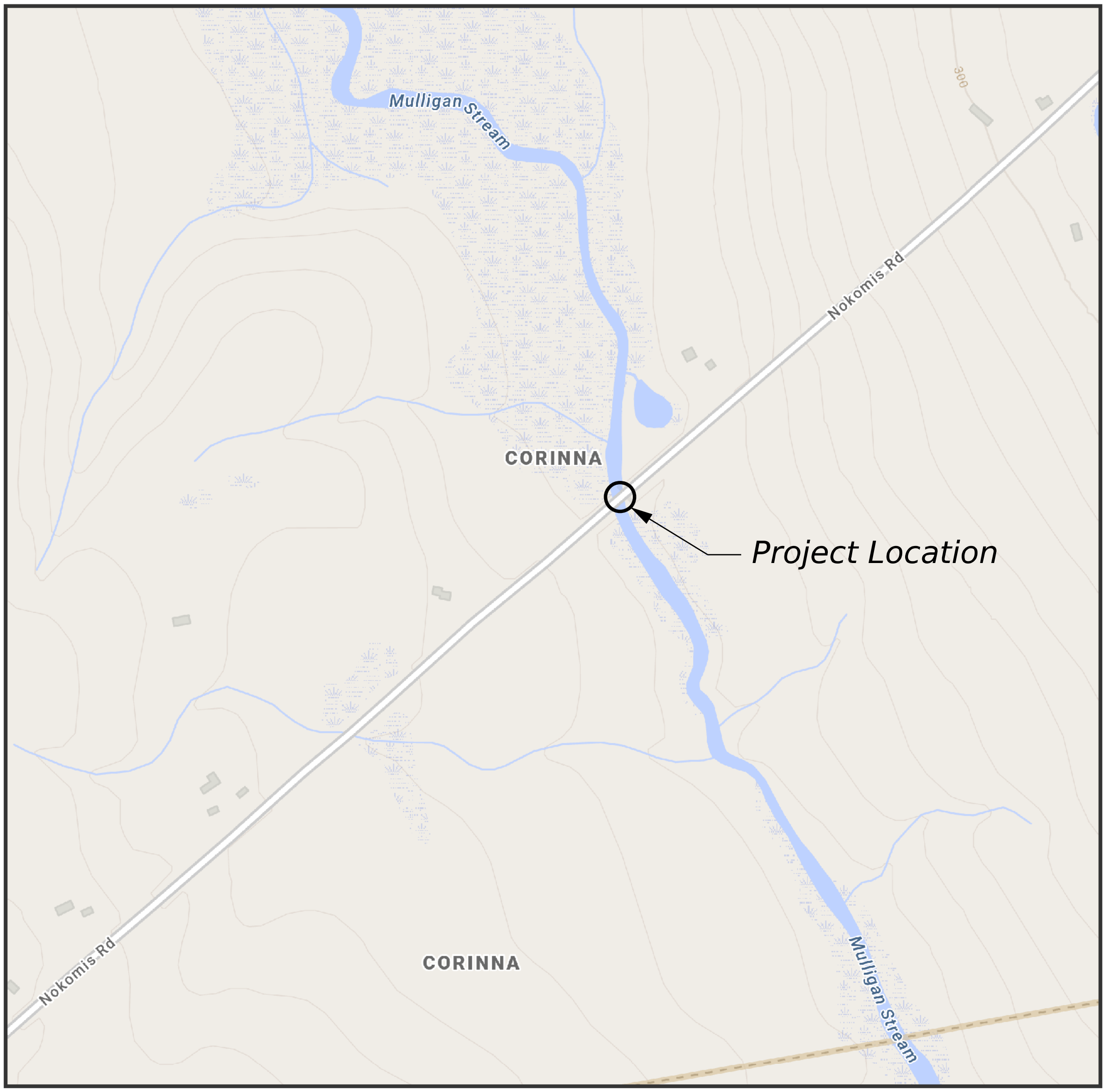
It is recommended that the geotechnical engineer be provided the opportunity for a review of the design and specifications so that the earthwork and foundation recommendations and construction considerations in the report are properly interpreted and implemented in the design and specifications.

Sheets

Date: 4/27/2026

Username: Terry.White

CORINNA, MAINE



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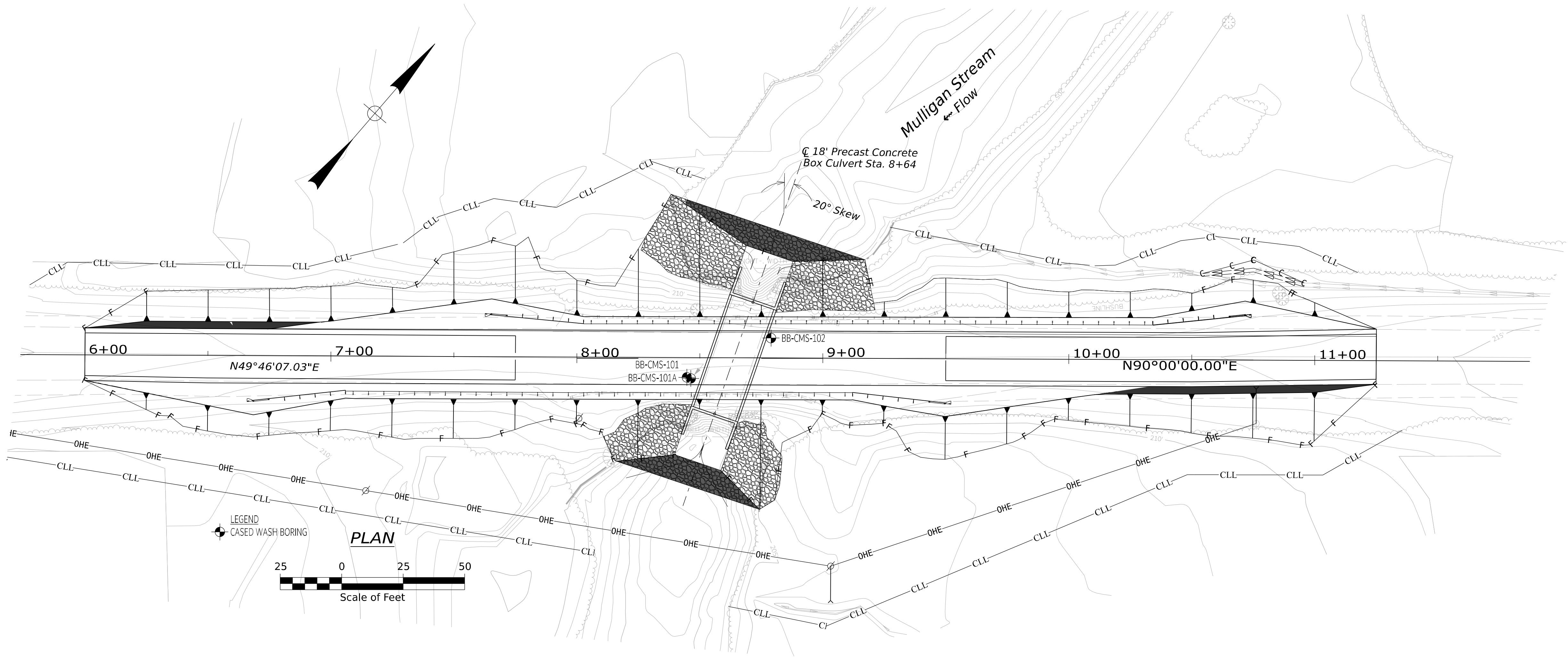
0.1 Miles

1 inch =0.14 miles

Date: 4/27/2026
Time: 9:32:00 AM

SHEET NUMBER 1 OF 4	MULLIGAN STREAM BRIDGE NO. 6160 CROSSING MULLIGAN STREAM CORINNA	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		Federal No. 2647600
	LOCATION MAP	WIN 26476.00 BRIDGE PLANS



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

Federal Project No. 2647600

WIN 26476.00

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED			
CHECKED-REVIEWED			
DESIGN2-DETAILED2	N. PUKAY	T. WHITE	APR 2026
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MULLIGAN STREAM BRIDGE NO. 6160
CROSSING MULLIGAN STREAM
CORINNA

BORING LOCATION PLAN

SHEET NUMBER

2

OF 4

Username: Nathan.P.Pukay Date: 4/28/2026

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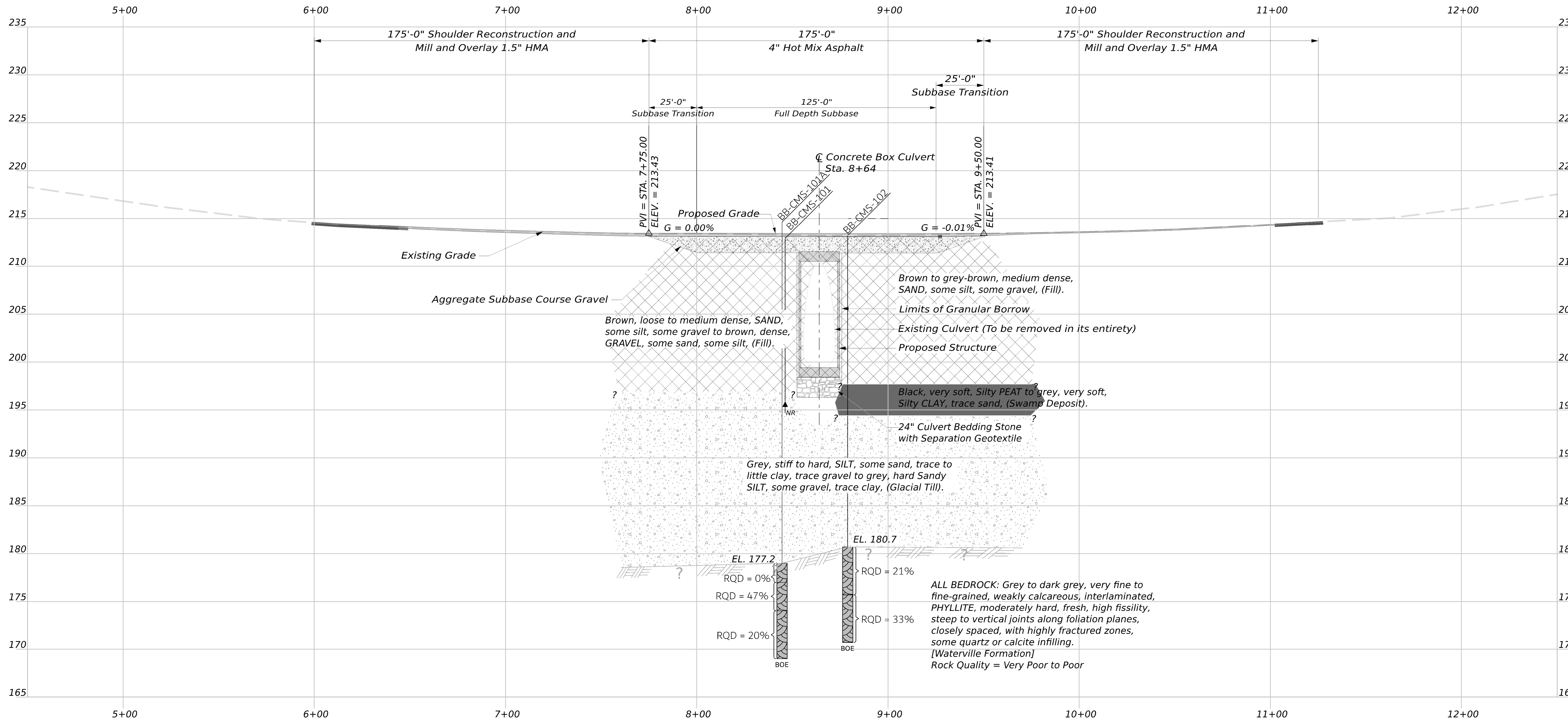
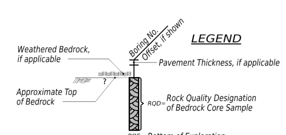
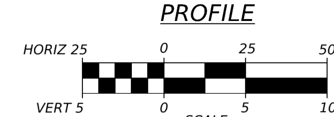




PROFILE



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
Federal Project No. 2647600
WIN 26476.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	D. TAYLOR	
DESIGN-DETAILED	W.CASEY	
CHECKED-REVIEWED	T.WHITE	MAR 2026
DESIGN2-DETAILED2	N.PUKAY	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MULLIGAN STREAM BRIDGE NO. 6160
CROSSING MULLIGAN STREAM
CORINNA

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Username: Nathan.P.Pukay Date: 6/1/2026

Filename: C:\Users\Nathan.P.Pukay\State of Maine\MaineDOT Work - 26476_00\GEOTECH\GEOProfile2.dgn

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

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over
Location: Corinna, Maine

Boring No.: BB-CMS-101
WIN: 26476.00

Driller:	S.W. Cole	Elevation (ft.)	212.9	Auger ID/OD:	5" Solid Stem
Operator:	Hanscom/Wall	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/12/2024; 09:00-09:50	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	8+46.2, 8.1 ft Rt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	7.0 ft bgs.
Hammer Efficiency Factor: 1.066		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.3	7" HMA. 0.6	
5	1D	24/15	5.00 - 7.00	6/8/8/6	16	28			Brown, moist, medium dense, SAND, some silt, some gravel, (Fill).	G#380941 A-1-b, SM WC=11.7%
10	2D	24/9	10.00 - 12.00	2/2/3/7	5	9	36		Brown, wet, loose, SAND, some silt, some gravel, (Fill).	
							40			
							74			
							110			
							83			
15	3D	24/1	15.00 - 17.00	4/2/18/25	20	36			Brown, wet, GRAVEL, some sand, some silt, (Fill).	
								196.9	16.0 Hole abandoned after 3D sample due to bent casing and casing disconnect. All casing was successfully retrieved.	
								195.9	17.0 Bottom of Exploration at 17.0 feet below ground surface.	
20										
25										

Remarks:

1) Auto Hammer #367

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

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

Page 1 of 1

Boring No.: BB-CMS-101

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

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over
Location: Corinna, Maine

Boring No.: BB-CMS-101A
WIN: 26476.00

Driller:	S.W. Cole	Elevation (ft.)	212.9	Auger ID/OD:	5" Solid Stem
Operator:	Hanscom/Wall	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/12/2024; 09:50-14:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	8+44.6, 7.6 ft Rt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	7.0 ft bgs.
Hammer Efficiency Factor: 1.066		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.3	7" HMA. 0.6 See BB-CMS-101 for material descriptions and casing blow counts up to 16.0 ft bgs.	
5										
10							DROVE HW			
15	1D	24/8	15.00 - 17.00	2/1/6/19	7	12	35	196.9	16.0 Grey, wet, stiff, SILT, some sand, little clay, trace gravel, (Glacial Till).	
							57			
							97			
							82			
							92			
20	2D	24/12	20.00 - 22.00	9/11/9/11	20	36	OPEN HOLE		Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	G#380942 A-4, ML WC=11.6%
25	3D	24/9	25.00 - 27.00	5/12/11/11	23	41			Grey, wet, hard, Sandy SILT, some gravel, trace clay, (Glacial Till).	G#380945 A-4, SC-SM WC=10.4% Non-Plastic
30	4D	24/7	30.00 - 32.00	16/13/15/20	28	50			Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	
	R1	24/15	33.90 - 35.90	RQD = 0%			NQ-2		R1: Grey cobbles and sand (Glacial Till)	
35	R2	36/33	35.90 - 38.90	RQD = 47%				177.2	35.7 Top of Bedrock at Elev. 177.2 ft. R1 Continued: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh. [Waterville Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 33.9-34.9 ft (2:42) 34.9-35.9 ft (3:40) 62% Recovery	
	R3	60/56	38.90 - 43.90	RQD = 20%					R2: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLITTE, moderately hard, fresh, steeply dipping joints along foliation planes, closely spaced, with some quartz or calcite infilling, fracture zone with very high fissility, readily splitting along foliation planes. [Waterville Formation] Rock Quality = Poor R2: Core Times (min:sec) 35.9-36.9 ft (3:50) 36.9-37.9 ft (2:50) 37.9-38.9 ft (3:45) 92% Recovery	
40										
								169.0	R3: Bedrock: Similar to R2, but with multiple fracture zones. [Waterville Formation] Rock Quality = Very Poor R3: Core Times (min:sec) 38.9-39.9 ft (4:02) 39.9-40.9 ft (4:08) 40.9-41.9 ft (3:45) 41.9-42.9 ft (6:35) 42.9-43.9 ft (6:02) 93% Recovery 43.9 Bottom of Exploration at 43.9 feet below ground surface.	
45										
50										
55										

Remarks:

1) Auto Hammer #367

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Page 1 of 1

Boring No.: BB-CMS-101A

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

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over
Location: Corinna, Maine

Boring No.: BB-CMS-102
WIN: 26476.00

Driller:	S.W. Cole	Elevation (ft.)	213.2	Auger ID/OD:	5" Solid Stem
Operator:	Hanscom/Wall	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/11/2024; 09:00-13:15	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	8+78.9, 8.0 ft Lt.	Casing ID/OD:	HW(4.0"/4.5")	Water Level*:	7.0 ft bgs.
Hammer Efficiency Factor: 1.066		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.6	7" HMA. 0.6	
5	1D	24/16	5.00 - 7.00	5/5/5/6	10	18			Brown, moist, medium dense, SAND, some silt, some gravel, (Fill).	
10	2D	24/8	10.00 - 12.00	1/4/6/5	10	18	22		Similar to above, except wet.	
							27			
							36			
							18			
							9			
15	3D/AB	24/16	15.00 - 17.00	1/1/WOH/1	1	2	HP	197.7	3D (Top 7") Grey-brown, wet, SAND, some silt, some gravel, (Fill). 15.5 3D/A (Middle 7") Black, wet, very soft, Silty PEAT, (Swamp Deposit). 3D/B (Bottom 2") Grey, wet, very soft, Silty CLAY, trace sand, (Swamp Deposit).	#380943 WC=292.3% Organic Content 50%
							HP			
							HP			
	MV		18.00 - 18.80	Could Not Push			8	194.4	Failed 55x110 mm vane attempt. Could not push due to Cobble at 18.8 ft bgs. 18.8	#380944 WC=37.7% LL=37 PL=23 PI=14
							78			
20	4D	24/4	20.00 - 22.00	11/2/4/7	6	11	63		Grey, wet, stiff, SILT, some sand, trace clay, trace gravel, (Glacial Till).	
							68			
							60			
							61			
							105			
25	5D	24/9	25.00 - 27.00	9/9/19/31	28	50	94		Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	
							150			
							99			
							73			
							112			
30	6D	24/6	30.00 - 32.00	48/26/18/16	44	78	122		Similar to 5D.	
							a198		a198 blows for 0.4 ft. Roller cone ahead to 32.5.	
	R1	60/60	32.50 - 37.50	RQD = 21%			NQ-2	180.7	32.5 Top of Bedrock at Elev. 180.7 ft. R1: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh, high fissility, vertical joints along foliation planes, closely spaced, with some quartz or calcite infilling. [Waterville Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 32.5-33.5 ft (5:36) 33.5-34.5 ft (4:26) 34.5-35.5 ft (4:25) 35.5-36.5 ft (5:10) 36.5-37.5 ft (5:07) 100% Recovery	
35										
	R2	60/60	37.50 - 42.50	RQD = 33%					R2: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh, high fissility, vertical joints along foliation planes, closely spaced, with some quartz or calcite infilling, highly fractured at the top of the run and then becomes more competent. [Waterville Formation] Rock Quality = Poor R2: Core Times (min:sec) 37.5-38.5 ft (4:54) 38.5-39.5 ft (5:18) 39.5-40.5 ft (4:10) 40.5-41.5 ft (3:55) 41.5-42.5 ft (3:47) 100% Recovery	
40										
								170.7	42.5 Bottom of Exploration at 42.5 feet below ground surface.	
45										
50										

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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Page 1 of 1

Boring No.: BB-CMS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
Federal Project No. 2647600
WIN 26476.00

PROJ MANAGER
DESIGN-DETAILED N. PUKAY
CHECKED-REVIEWED T. WHITE
DESIGN2-DETAILED2
DESIGN3-DETAILED3
REVISIONS 1
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES
BY
DATE APR 2026
SIGNATURE
P.E. NUMBER
DATE

MULLIGAN STREAM BRIDGE NO. 6160
CROSSING MULLIGAN STREAM
CORINNA

BORING LOGS

SHEET NUMBER
4
OF 4

Username: Nathan.P.Pukay Date: 4/28/2026

Filename: C:\Users\Terry.White\State of Maine\MaineDOT Work - 26476_00\GEOTECH\Boring_Log_Plan1.dgn

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream
Location: Corinna, Maine

Boring No.: BB-CMS-101
WIN: 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	212.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/12/2024; 09:00-09:50	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	8+46.2, 8.1 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.3		7" HMA. 0.6	
5	1D	24/15	5.00 - 7.00	6/8/8/6	16	28				Brown, moist, medium dense, SAND, some silt, some gravel, (Fill).	G#380941 A-1-b, SM WC=11.7%
10	2D	24/9	10.00 - 12.00	2/2/3/7	5	9	36			Brown, wet, loose, SAND, some silt, some gravel, (Fill).	
							40				
							74				
							110				
							83				
15	3D	24/1	15.00 - 17.00	4/2/18/25	20	36				Brown, wet, GRAVEL, some sand, some silt, (Fill).	
								196.9		16.0 Hole abandoned after 3D sample due to bent casing and casing disconnect. All casing was successfully retrieved.	
								195.9		17.0 **Bottom of Exploration at 17.0 feet below ground surface.**	
20											
25											

Remarks:

1) Auto Hammer #367

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream
Location: Corinna, Maine

Boring No.: BB-CMS-101A
WIN: 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	212.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/12/2024; 09:50-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44.6, 7.6 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.3		7" HMA. 0.6	
										See BB-CMS-101 for material descriptions and casing blow counts up to 16.0 ft bgs.	
5											
10							DROVE HW				
15	1D	24/8	15.00 - 17.00	2/1/6/19	7	12	35				
							57	196.9		16.0 Grey, wet, stiff, SILT, some sand, little clay, trace gravel, (Glacial Till).	
							97				
							82				
							92				
20	2D	24/12	20.00 - 22.00	9/11/9/11	20	36	OPEN HOLE			Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	G#380942 A-4, ML WC=11.6%
25											

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream **Location:** Corinna, Maine	**Boring No.:** BB-CMS-101A **WIN:** 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	212.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/12/2024; 09:50-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44.6, 7.6 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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RC = Roller Cone
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WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	3D	24/9	25.00 - 27.00	5/12/11/11	23	41				Grey, wet, hard, Sandy SILT, some gravel, trace clay, (Glacial Till).	G#380945 A-4, SC-SM WC=10.4% Non-Plastic
30	4D	24/7	30.00 - 32.00	16/13/15/20	28	50				Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	
	R1	24/15	33.90 - 35.90	RQD = 0%			NQ-2			R1: Grey cobbles and sand (Glacial Till)	
35	R2	36/33	35.90 - 38.90	RQD = 47%				177.2		35.7 Top of Bedrock at Elev. 177.2 ft. R1 Continued: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh. [Waterville Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 33.9-34.9 ft (2:42) 34.9-35.9 ft (3:40) 62% Recovery	
	R3	60/56	38.90 - 43.90	RQD = 20%						R2: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLITTE, moderately hard, fresh, steeply dipping joints along foliation planes, closely spaced, with some quartz or calcite infilling, fracture zone with very high fissility, readily splitting along foliation planes. [Waterville Formation] Rock Quality = Poor R2: Core Times (min:sec) 35.9-36.9 ft (3:50) 36.9-37.9 ft (2:50) 37.9-38.9 ft (3:45) 92% Recovery	
40								169.0		R3: Bedrock: Similar to R2, but with multiple fracture zones. [Waterville Formation] Rock Quality = Very Poor R3: Core Times (min:sec) 38.9-39.9 ft (4:02) 39.9-40.9 ft (4:08)	
45											
50											

Remarks:

1) Auto Hammer #367

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream
Location: Corinna, Maine

Boring No.: BB-CMS-101A
WIN: 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	212.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/12/2024; 09:50-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+44.6, 7.6 ft Rt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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HSA = Hollow Stem Auger
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										40.9-41.9 ft (3:45) 41.9-42.9 ft (6:35) 42.9-43.9 ft (6:02) 93% Recovery	
										43.9	
										Bottom of Exploration at 43.9 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

1) Auto Hammer #367

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream Location: Corinna, Maine	Boring No.: BB-CMS-102 WIN: 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	213.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2024; 09:00-13:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+78.9, 8.0 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	212.6		7" HMA. 0.6	
5	1D	24/16	5.00 - 7.00	5/5/5/6	10	18				Brown, moist, medium dense, SAND, some silt, some gravel, (Fill).	
10	2D	24/8	10.00 - 12.00	1/4/6/5	10	18	22			Similar to above, except wet.	
							27				
							36				
							18				
							9				
15	3D/AB	24/16	15.00 - 17.00	1/1/WOH/1	1	2	HP	197.7		3D (Top 7") Grey-brown, wet, SAND, some silt, some gravel, (Fill). 15.5	
							HP			3D/A (Middle 7") Black, wet, very soft, Silty PEAT, (Swamp Deposit).	#380943 WC=292.3% Organic Content 50%
							HP			3D/B (Bottom 2") Grey, wet, very soft, Silty CLAY, trace sand, (Swamp Deposit).	
	MV		18.00 - 18.80	Could Not Push			8	194.4		Failed 55x110 mm vane attempt. Could not push due to Cobble at 18.8 ft bgs. 18.8	#380944 WC=37.7% LL=37 PL=23 PI=14
							78				
20	4D	24/4	20.00 - 22.00	11/2/4/7	6	11	63			Grey, wet, stiff, SILT, some sand, trace clay, trace gravel, (Glacial Till).	
							68				
							60				
							61				
25							105				

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mulligan Stream Bridge #6160 carries Nokomis Road over Mulligan Stream
Location: Corinna, Maine

Boring No.: BB-CMS-102
WIN: 26476.00

Driller:	S.W. Cole	**Elevation (ft.)**	213.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Hanscom/Wall	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2024; 09:00-13:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	8+78.9, 8.0 ft Lt.	**Casing ID/OD:**	HW(4.0"/4.5")	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	1.066	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/9	25.00 - 27.00	9/9/19/31	28	50	94			Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	
							150				
							99				
							73				
							112				
30	6D	24/6	30.00 - 32.00	48/26/18/16	44	78	122			Similar to 5D.	
							a198			a198 blows for 0.4 ft. Roller cone ahead to 32.5.	
	R1	60/60	32.50 - 37.50	RQD = 21%			NQ-2	180.7		32.5 Top of Bedrock at Elev. 180.7 ft. R1: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh, high fissility, vertical joints along foliation planes, closely spaced, with some quartz or calcite infilling. [Waterville Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 32.5-33.5 ft (5:36) 33.5-34.5 ft (4:26) 34.5-35.5 ft (4:25) 35.5-36.5 ft (5:10) 36.5-37.5 ft (5:07) 100% Recovery	
35											
	R2	60/60	37.50 - 42.50	RQD = 33%						R2: Bedrock: Grey to dark grey, very fine to fine-grained, weakly calcareous, interlaminated, PHYLLITE, moderately hard, fresh, high fissility, vertical joints along foliation planes, closely spaced, with some quartz or calcite infilling, highly fractured at the top of the run and then becomes more competent. [Waterville Formation] Rock Quality = Poor R2: Core Times (min:sec) 37.5-38.5 ft (4:54) 38.5-39.5 ft (5:18) 39.5-40.5 ft (4:10) 40.5-41.5 ft (3:55) 41.5-42.5 ft (3:47) 100% Recovery	
40											
								170.7		42.5 **Bottom of Exploration at 42.5 feet below ground surface.**	
45											
50											

Remarks:

1) Auto Hammer #367
2) HP = Hydraulic Push

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

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

Boring No.: BB-CMS-102

<u>Appendix B</u>

Rock Core Photographs

MaineDOT

Mulligan Stream Bridge #6160 Carries Nokomis Road Over Mulligan Stream

Corinna, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CMS-102	R1	32.50-37.50	60	60	13	21	PHYLLITE	1
BB-CMS-102	R2	37.50-42.50	60	60	20	33	PHYLLITE	2
BB-CMS-101A	R1	33.90-35.90	24	15	0	0	PHYLLITE	3
BB-CMS-101A	R2	35.90-38.90	36	33	17	47	PHYLLITE	3
BB-CMS-101A	R3	38.90-43.90	60	56	12	20	PHYLLITE	4

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

2. Top of each core run is on the left and increases with depth to the right.

3. Transition between core runs is marked by wooden blocks.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Corinna Work Number: 26476.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-CMS-101, 1D	8+46.2	8.1 Rt.	5.0-7.0	380941	1	11.7			SM	A-1-b	II
BB-CMS-101A, 2D	8+44.6	7.6 Rt.	20.0-22.0	380942	1	11.6			ML	A-4	IV
BB-CMS-101A, 3D	8+44.6	7.6 Rt.	25.0-27.0	380945	1	10.4	-N	P-	SC-SM	A-4	III
BB-CMS-102, 3D/A	8+78.9	8.0 Lt.	15.5-16.0	380943	---	292.3	Loss on Ignition (T 267) 50%				
BB-CMS-102 3D/B	8+78.9	8.0 Lt.	16.0-16.5	380944	---	37.7	37	14	CL	A-6	III

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

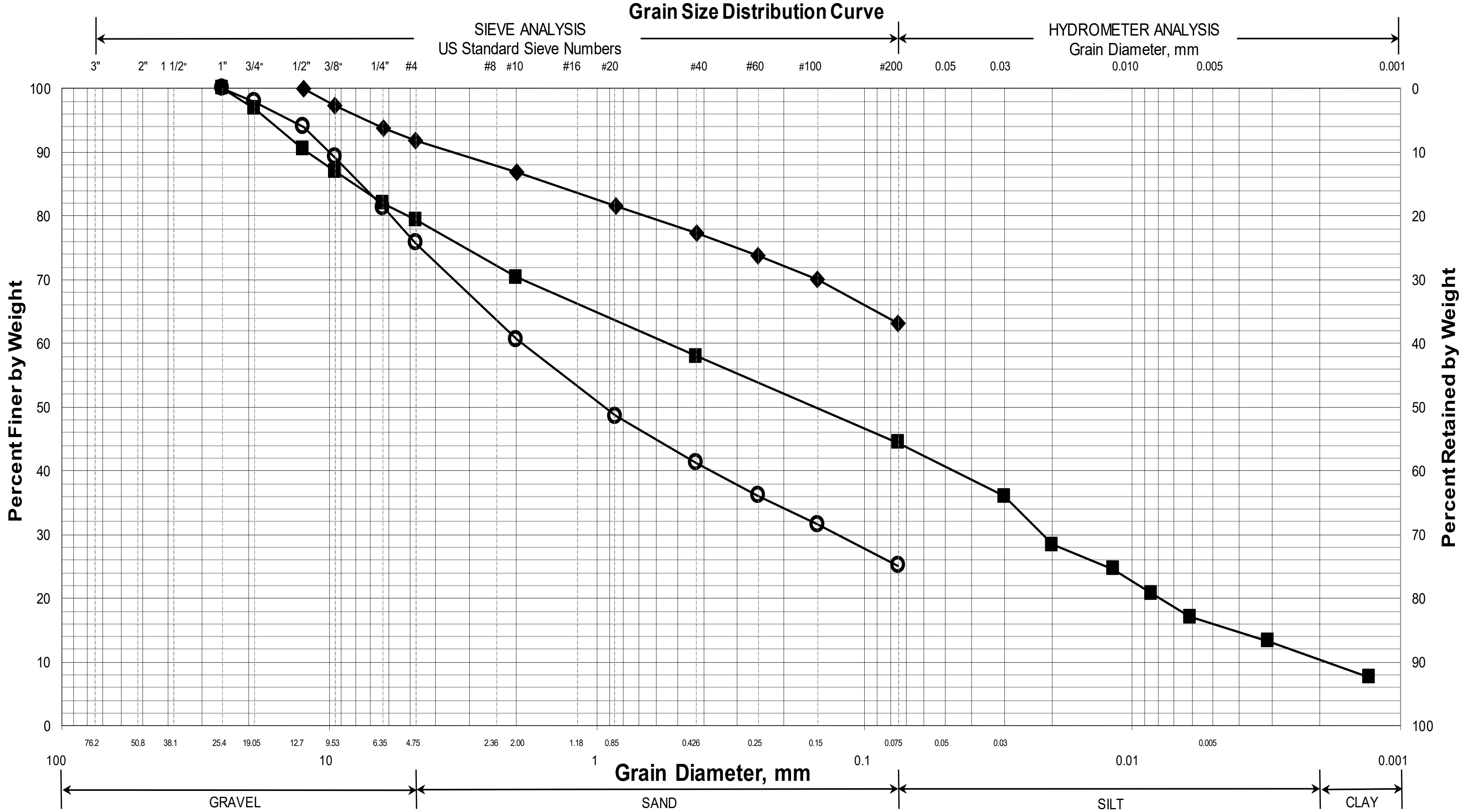
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98 NP = Non Plastic

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

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-CMS-101/1D	8+46.2	8.1 RT	5.0-7.0	SAND, some silt, some gravel.	11.7			
◆	BB-CMS-101A/2D	8+44.6	7.6 RT	20.0-22.0	SILT, some sand, trace garvel.	11.6			
■	BB-CMS-101A/3D	8+44.6	7.6 RT	25.0-27.0	Sandy SILT, some gravel, trace clay.	10.4			NP
●									
▲									
×									

WIN	
026476.00	
Town	
Corinna	
Reported by/Date	
WHITE, TERRY A	4/22/2026

SHEET 1

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380945	**BB-CMS-101A/3D**	**GEOTECHNICAL (DISTURBED)**	**1/12/2024**	**2/2/2024**

Sample Type: **GEOTECHNICAL** Location: Station: **8+44.6** Offset, ft: **7.6** **RT** Dbfg, ft: **25.0-27.0**

WIN/Town **026476.00 - CORINNA** Sampler: **NATHAN PUKAY**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**96.8**
½ in. [12.5 mm]	**90.5**
⅜ in. [9.5 mm]	**87.0**
¼ in. [6.3 mm]	**82.0**
No. 4 [4.75 mm]	**79.4**
No. 10 [2.00 mm]	**70.4**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**58.0**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**44.4**
[0.0301 mm]	**36.0**
[0.0199 mm]	**28.4**
[0.0118 mm]	**24.6**
[0.0085 mm]	**20.8**
[0.0061 mm]	**17.1**
[0.0031 mm]	**13.3**
[0.0013 mm]	**7.6**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	**NP**
Specific Gravity, Corrected to 20°C (T 100)	**2.70**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**10.4**

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/12/2024**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380943	BB-CMS-102/3DA	GEOTECHNICAL (DISTURBED)	1/11/2024	1/19/2024

Sample Type: **GEOTECHNICAL** Location: Station: **8+78.9** Offset, ft: **8.0 LT** Dbfg, ft: **15.0-17.0**

WIN/Town **026476.00 - CORINNA** Sampler: **NATHAN PUKAY**

TEST RESULTS

Sieve Analysis (T 27)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	

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)	50
Water Content (T 265), %	292.3

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **1/24/2024**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
380944	BB-CMS-102/3DB	GEOTECHNICAL (DISTURBED)	1/11/2024	1/19/2024

Sample Type: **GEOTECHNICAL** Location: Station: **8+78.9** Offset, ft: **8.0** **LT** Dbfg, ft: **15.0-17.0**

WIN/Town **026476.00 - CORINNA** Sampler: **NATHAN PUKAY**

TEST RESULTS

Sieve Analysis (T 27)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	
[mm]	
[mm]	
[mm]	
[mm]	
[mm]	
[mm]	
[mm]	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	37
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	37.7

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

Comments:

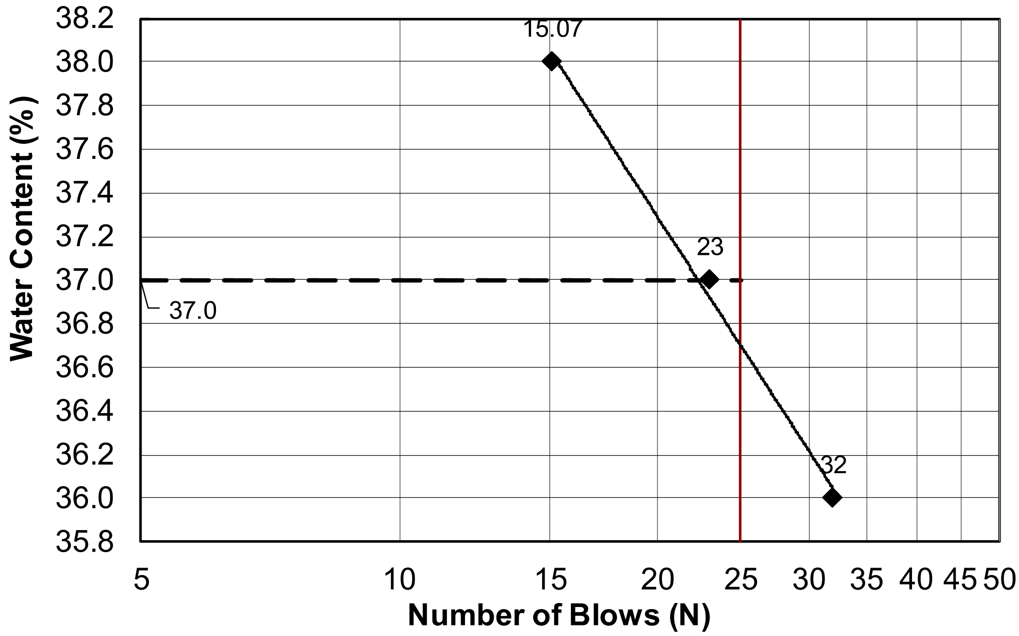
Hydro cancelled due to insufficient material.

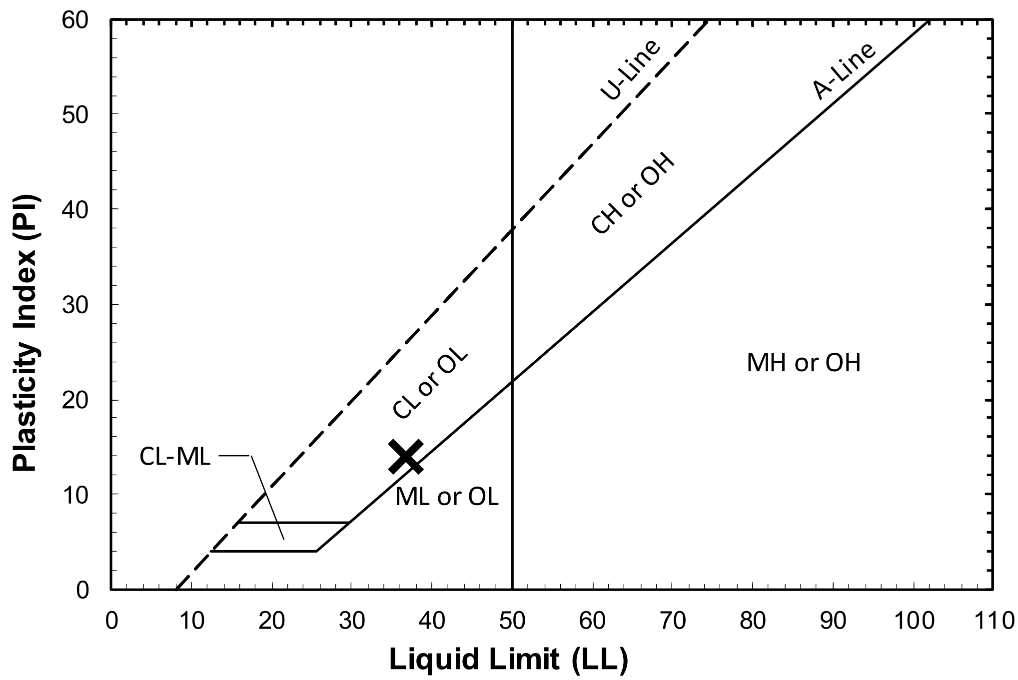
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **1/25/2024**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Corinna	Reference No.	380944
WIN	026476.00	Water Content, %	37.7
Sampled	1/11/2024	Liquid Limit @ 25 blows (T 89), %	37
Boring No./Sample No.	BB-CMS-102/3DB	Plastic Limit (T 90), %	23
Station	8+78.9	Plasticity Index (T 90), %	14
Depth	15.0-17.0	Tested By	BBURR





Appendix D

Calculations

Liquidity Index

Liquidity Index

$$LI := \frac{WC - PL}{LL - PL}$$

Das, Principles of Engineering, 7th Edition, Equation 4.16

BB-CMS-102, 3D/B

$WC := 37.7$

$LL := 37$

$PL := 23$

$$LI := \frac{WC - PL}{LL - PL} = 1.05$$

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

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

$\boxed{K_o = 0.47}$

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:

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

$\boxed{K_{ar} = 0.31}$

For a sloped 2H:1V backfill

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

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

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

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

6.1 AT-REST LATERAL PRESSURES

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

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

$$\sigma'_x = K_0(\sigma'_z) \tag{6.1}$$

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:

$$\boxed{K_0 = 1 - \sin\phi'} \tag{6.2}$$

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

[a] Unloading cycle.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

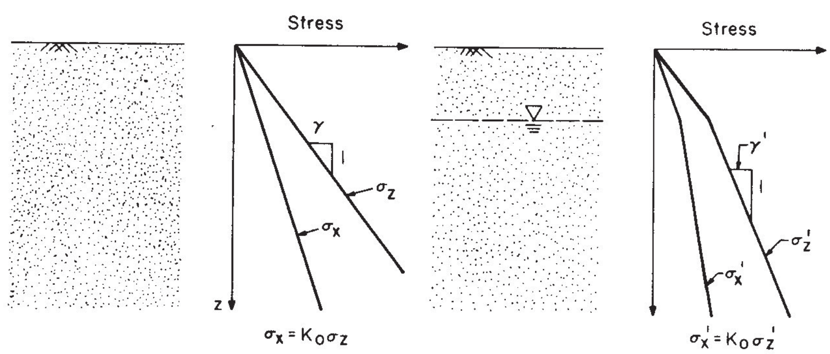


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

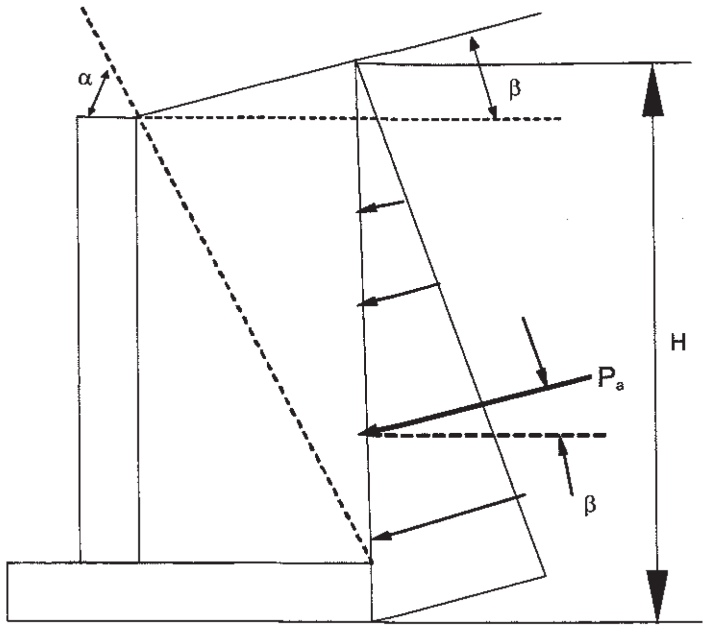


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing Resistance

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

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

Assumptions:
1. The box culvert's embedment is 1' into the streambed.
2. The design box culvert invert is El. 199.59 ft. With an estimated bottom slab thickness of 1 foot, the proposed bearing elevation is approximately El. 198.5 feet. The box culvert will be founded on 2 feet of culvert bedding stone. Any soft silty clay or organic material beneath the bedding stone will be removed to expose glacial till subgrade and replaced with compacted culvert bedding stone.
3. Proposed finish roadway grade elevation is approximately 213 feet at the low point.
4. Proposed precast concrete box base is 20 feet wide. The box culvert span is 18 feet with the walls assumed to be 1 foot thick based on previous culverts of similar size.
5. The bottom of the box culvert will be submerged for the structure's design life.

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 10th Edition Table C10.6.2.5.1-1 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	AASHTO Recommended Value of Use	MaineDOT Recommended Value
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Medium stiff to stiff	2-6	3	6

Recommend 6 ksf to limit settlement on stiff to hard glacial till soils to 1.0 inch for Service Limit State Loads

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

Foundation Width, Depth, and Water Surface

$$B := 20ft$$

$$D_f := 1 \cdot ft$$

$$D_w := 0 \cdot ft$$

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

Total unit weight of the soil above the bearing depth of the base slab

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

$\gamma_{above\_sat} := 135pcf$

MaineDOT Bridge Design Guide p. 3-3
Soil Type 4 (granular borrow)

Foundation soils:

Foundation soils properties based on BB-CMS-101A (2D) and BB-CMS-102 (4D)

$\gamma_{below\_dry} := 121 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2, Dense angular-grained silty sand

$w_{sat} := 11.6\%$

From BB-CMS-101A (2D)

$\gamma_{below\_sat} := \gamma_{below\_dry} \cdot (1 + w_{sat})$

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

$\gamma_{below\_sat} = 135 \cdot pcf$

$\gamma_{below\_moist} := 130pcf$

Moist unit weight estimated between dry and saturated unit weights

$\phi := 32 \cdot deg$

$c := 0psf$

Friction angle of subgrade soils encountered in borings, using correlations: (1) Table 11.3 (HOUGH) and (2) Table 2-6, (BOWLES 5th Ed.).
BB-CMS-101A (2D) Dense Nonplastic Silt (34 deg)
BB-CMS-102 (4D) Medium Dense Nonplastic Silt (32 deg)

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)

$N_c := 35.5$

$N_q := 23.2$

$N_\gamma := 30.2$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$L := 90 \cdot ft$

Length of base slab

$$s_c := 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right)$$

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

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

$s_c = 1.1$ $\quad s_\gamma = 0.9$ $\quad s_q = 1.1$

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.

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

Load Inclination factors

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

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

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. Competent fill soils above.

$$d_q := 1 + 2 \cdot \tan(\phi) \cdot (1 - \sin(\phi))^2 \cdot \tan\left(\frac{D_f}{B}\right)^{-1}$$ LRFD 10.6.3.1.2a-10

Per LRFD 10.6.3.1.2a, d_q shall not exceed 1.4. Additionally, per LRFD C10.6.3.1.2a, the above equation has been verified to cover a range of friction angle, ϕ, of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8.

$$\frac{D_f}{B} = 0.1$$

Therefore $\quad d_q := 1$

Terms

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

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

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

$N_{cm} = 40.7$ $\quad N_{\gamma m} = 27.5$ $\quad N_{qm} = 26.4$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 19.5 \cdot ksf$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$\boxed{q_r = 8.8 \cdot ksf}$$

Recommend a factored bearing resistance of 8.8 ksf for the proposed box culvert with a bottom slab width of 20 ft 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 δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

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

3.4 Various Unit-Weight Relationships

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

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

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

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

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft³	kN/m³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

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SUBJECT ____________ DATE ____________

COMPUTED BY ____________

CHECKED BY ____________

Table 11.3 Summary of Friction Angle Data for Use in Preliminary Design

	Slope Angle of Repose		At Ultimate Strength		At Peak Strength: Medium Dense		At Peak Strength: Dense	
Classification	i(°)	Slope (vert. to hor.)	ϕ_{cv}(°)	tan ϕ_{cv}	ϕ(°)	tan ϕ	ϕ(°)	tan ϕ
Silt (nonplastic)	26 to 30	1 on 2 / 1 on 1.75	26 to 30	0.488 / 0.577	28 to 32	0.532 / 0.625	30 to 34	0.577 / 0.675
Uniform fine to medium sand	26 to 30	1 on 2 / 1 on 1.75	26 to 30	0.488 / 0.577	30 to 34	0.577 / 0.675	32 to 36	0.675 / 0.726
Well-graded sand	30 to 34	1 on 1.75 / 1 on 1.50	30 to 34	0.577 / 0.675	34 to 40	0.675 / 0.839	38 to 46 GLACIAL TILL	0.839 / 1.030
Sand and gravel	32 to 36	1 on 1.60 / 1 on 1.40	32 to 36	0.625 / 0.726	36 to 42	0.726 / 0.900	40 to 48	0.900 / 1.110

USE 28 ≤ φ ≤ 30
SEE FIG. 11.14 LAMBE/WHIT.

31°

35°

From B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.
Note. Within each range, assign lower values if particles are well rounded or if there is significant soft shale or mica content, higher values for hard, angular particles. Use lower values for high normal pressures than for moderate normal pressure.

Table 1.4 Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

med. DENSE SAND
$\gamma = \frac{118+130}{2} = 124$

Description	Porosity (n)	Void Ratio (e)	Water Content (w)[a]	Unit Weight g/cu cm γ_d[b]	Unit Weight g/cu cm γ_{sat}[c]	Unit Weight lb/cu ft γ_d	Unit Weight lb/cu ft γ_{sat}
1. Uniform sand, loose	0.46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	0.34	0.51	19	1.75	2.09	109	130
3. Mixed-grained sand, loose	0.40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	0.30	0.43	16	1.86	2.16	116	135
5. Windblown silt (loess)	0.50	0.99	21	1.36	1.86	85	116
6. Glacial till, very mixed-grained	0.20	0.25	9	2.12	2.32	132	145
7. Soft glacial clay	0.55	1.2	45	1.22	1.77	76	110
8. Stiff glacial clay	0.37	0.6	22	1.70	2.07	106	129
9. Soft slightly organic clay	0.66	1.9	70	0.93	1.58	58	98
10. Soft very organic clay	0.75	3.0	110	0.68	1.43	43	89
11. Soft montmorillonitic clay (calcium bentonite)	0.84	5.2	194	0.43	1.27	27	80

$\gamma \cong 122$
Fine to medium sand

$\frac{124+135}{2}$

mixed fine sand well graded
$\gamma = 130$ PCF

med. Glacial clay
$\gamma = 120$

[a] w = water content when saturated, in per cent of dry weight.
[b] γ_d = dry unit weight.
[c] γ_{sat} = saturated unit weight.

PEAT 90-100 PCF

FOUNDATION ANALYSIS AND DESIGN, BOWLES, 5TH EDITION

TABLE 2-6
Representative values for angle of internal friction ϕ

Soil	Unconsolidated-undrained, U	Consolidated-undrained, CU	Consolidated-drained, CD
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28–34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		$\boxed{30–35°}$
Clay	0° if saturated	3–20°	20–42°

*See a laboratory manual on soil testing for a complete description of these tests, e.g., Bowles (1992).

Notes:

1. Use larger values as γ increases.
2. Use larger values for more angular particles.
3. Use larger values for well-graded sand and gravel mixtures (GW, SW).
4. Average values for gravels, 35–38°; sands, 32–34°.

Figure 2-35 Correlation between ϕ' and plasticity index I_p for normally consolidated (including marine) clays. Approximately 80 percent of data falls within one standard deviation. Only a few extreme scatter values are shown *[Data from several sources: Ladd et al. (1977), Bjerrum and Simons (1960), Kanja and Wolle (1977), Olsen et al. (1986).]*

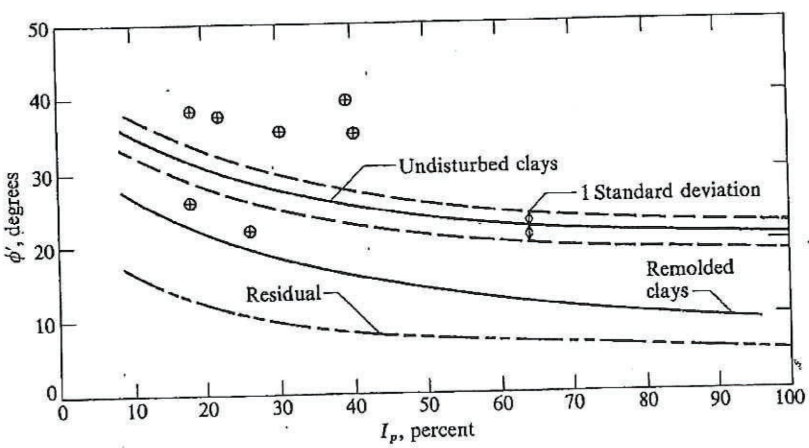


Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) 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 Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 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	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.

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

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:

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

in which:

$$N_{cm} = N_c s_c i_c \tag{10.6.3.1.2a-2}$$

$$N_{qm} = N_q s_q d_q i_q \tag{10.6.3.1.2a-3}$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \tag{10.6.3.1.2a-4}$$

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)

C10.6.3.1.2a

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.

N_γ = unit weight (footing width) term (drained loading) bearing capacity factor, as specified in Table 10.6.3.1.2a-1 (dim)
γ_q = total (moist) unit weight of soil above the bearing depth of the footing (kcf)
γ_f = total (moist) unit weight of soil below the bearing depth of the footing (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table, as specified in Table 10.6.3.1.2a-2 (dim)
s_c, s_γ, s_q = footing shape correction factors, as specified in Table 10.6.3.1.2a-3 (dim)
d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation determined from Eq. 10.6.3.1.2a-10 (dim)
i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For $\phi_f = 0$:

$$i_c = 1 - (nH/cBLN_c) \qquad (10.6.3.1.2a\text{-}5)$$

For $\phi_f > 0$:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \qquad (10.6.3.1.2a\text{-}6)$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \qquad (10.6.3.1.2a\text{-}7)$$

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

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

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)

Most geotechnical engineers nationwide have not used the load inclination factors. This is due, in part, to the lack of knowledge of the vertical and horizontal loads at the time of geotechnical explorations and preparation of bearing resistance recommendations.

Furthermore, the basis of the load inclination factors computed by Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8 is a combination of bearing resistance theory and small scale load tests on 1.0 in. wide plates on London Clay and Ham River Sand (Meyerhof, 1953). Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of $D_f/B = 1$, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by Brinch-Hansen (1970), with additional modification by Vesic' (1973) into the form provided in Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8.

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.

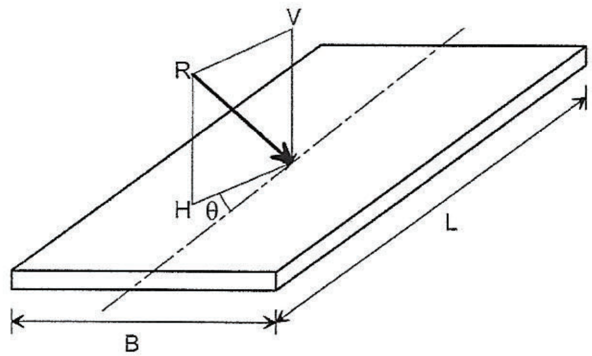


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
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
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

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

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.

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.

10.6.3.1.2b—Considerations for Punching Shear

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:

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

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

where:

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

C10.6.3.1.2b

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.

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.

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, 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 design box culvert invert is El. 199.59 ft. With an estimated bottom slab thickness of 1 foot, the proposed bearing elevation is approximately El. 198.5 feet. The box culvert will be founded on 2 feet of culvert bedding stone. Any soft silty clay or organic material beneath the bedding stone will be removed to expose glacial till subgrade and replaced with compacted culvert bedding stone.
2. Proposed finished roadway grade is approximately 213 feet.
3. Proposed precast concrete box is 20 feet wide. The box culvert span is 18 feet with the walls assumed to be 1 foot thick based on previous culverts of similar size.
4. Proposed precast box culvert is 90 feet long.
5. Any soft silty clay or organic material will be removed to expose glacial till subgrade. The glacial till at the bearing elevation is comprised of stiff to hard SILT, some sand, trace clay, trace gravel (BB-CMS-101A and BB-CMS-102). (Modeled as a Silty SAND)
6. 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 sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction 88 to 177 pci
Silty medium dense sand, use the lower limit: k_s = 88 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Saturated, medium dense, coarse grained glacial till K_{v1}, 150 pci/2 = 75 pci

Recommend a subgrade modulus of **75 pci** for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \qquad (9\text{-}10)$$

r either horizontal or vertical members
for depth variation
terest below ground
give k_s the best fit (if load test or other data are available)

ation 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 $B_s \cong 0$. used with the proper interpretation of the bearing-capacity equa- e d_i factors dropped) to give

$$q_{\text{ult}} = cN_c s_c + \gamma Z N_q s_q + 0.5\gamma B N_\gamma s_\gamma) \qquad (9\text{-}10a)$$

$$s_c + 0.5\gamma B N_\gamma s_\gamma) \qquad \text{and} \qquad B_s Z^1 = C(\gamma N_q s_q)Z^1$$

to estimate k_s. In these equations the Terzaghi or Hansen bearing- ed. The C factor is 40 for SI units and 12 for Fps, using the same at a 0.0254-m and 1-in. settlement but with no SF, since this equa- here there is concern that k_s does not increase without bound with e $B_s Z$ 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
h of interest
timate of the exponent

to estimate a value of k_s to determine the correct order of magnitude btained using one of the approximations given here. Obviously if a three times larger than the table range indicates, the computations possible gross error. Note, however, if you use a reduced value of r 12 mm) instead of 0.0254 m you may well exceed the table range. putational error (or a poor assumption) is found then use judgment table values are intended as guides. The reader should not use, say, ven as a "good" estimate.
d in Fig. 9-9c (and used in your diskette program FADBEMLP as stimated at some small value of, say, 6 to 25 mm, or from inspection if a load test was done. It might also be estimated from a triaxial ltimate" or at the maximum pressure from the stress-strain plot. $\epsilon_{\max}$ compute

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

Bowles , Foundation Analysis and Design, 5th ed. p. 505

TABLE 9-1
Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

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

Soil	k_s, kN/m³	ks, 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

133 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:

$$B = 1.22 \text{ m} \qquad L = 1.83 \text{ m} \qquad D = 0.610 \text{ m}$$
$$q_a = 200 \text{ kPa (clayey sand approximately 10 m deep)}$$
$$E_s = 11.72 \text{ MPa (average in depth } 5B \text{ below base)}$$

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

$$E_s' = \frac{1-\mu^2}{E_s} = \frac{1-0.3^2}{11.72} = 0.077\,65 \text{ m}^2/\text{MN}$$

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

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

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

$$k_s = \frac{1}{0.61(0.077\,65)(4 \times 0.597)(0.8)} = \mathbf{11.05} \text{ MN/m}^3$$

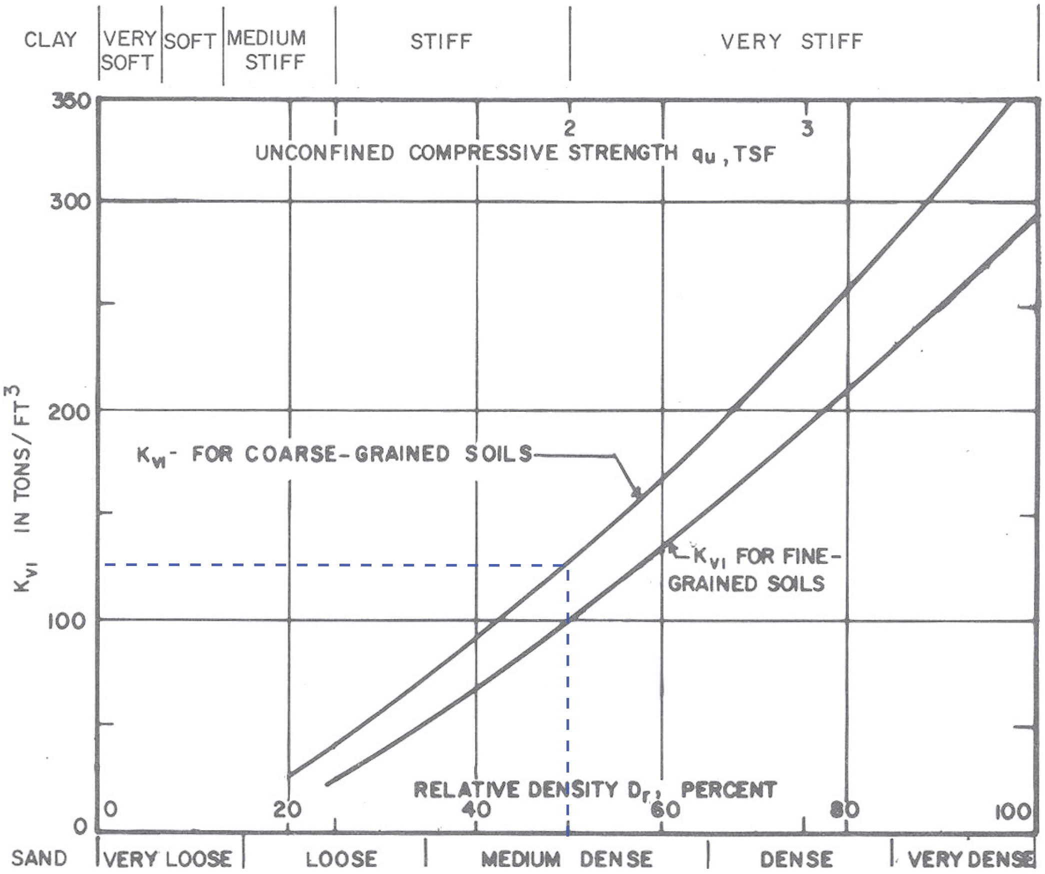
You should note that k_s does not depend on the contact pressure of the base q_o.
For **corner:**

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

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

130 TCF = 150 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

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

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

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

FOR $B \geq 40$ 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 $B \leq 20$ 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)

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: Corinna, Maine
DFI = 1850 degree-days.
Coarse-Grained Fill w=10% (BB-CMS-101 1D)

Coarse-Grained Fill

For DFI = 1800, Coarse-Grained Soil, w=10%

$DFI_1 := 1800 \quad d_1 := 90.1in$

d=Depth of Frost Penetration

For DFI = 1900, Coarse-Grained Soil, w=10%

$DFI_2 := 1900 \quad d_2 := 92.6in$

Interpolate for DFI = 1850, Coarse-Grained Soil, w=10%

$DFI_3 := 1850$

$$d_{coarse} := d_1 + \left(DFI_3 - DFI_1\right) \cdot \frac{\left(d_2 - d_1\right)}{\left(DFI_2 - DFI_1\right)}$$

$\boxed{d_{coarse} = 91.4 \cdot in}$ $\boxed{d_{coarse} = 7.6 \cdot ft}$

Recommend any foundation bearing on soil be embedded **7.6 feet** for frost protection.

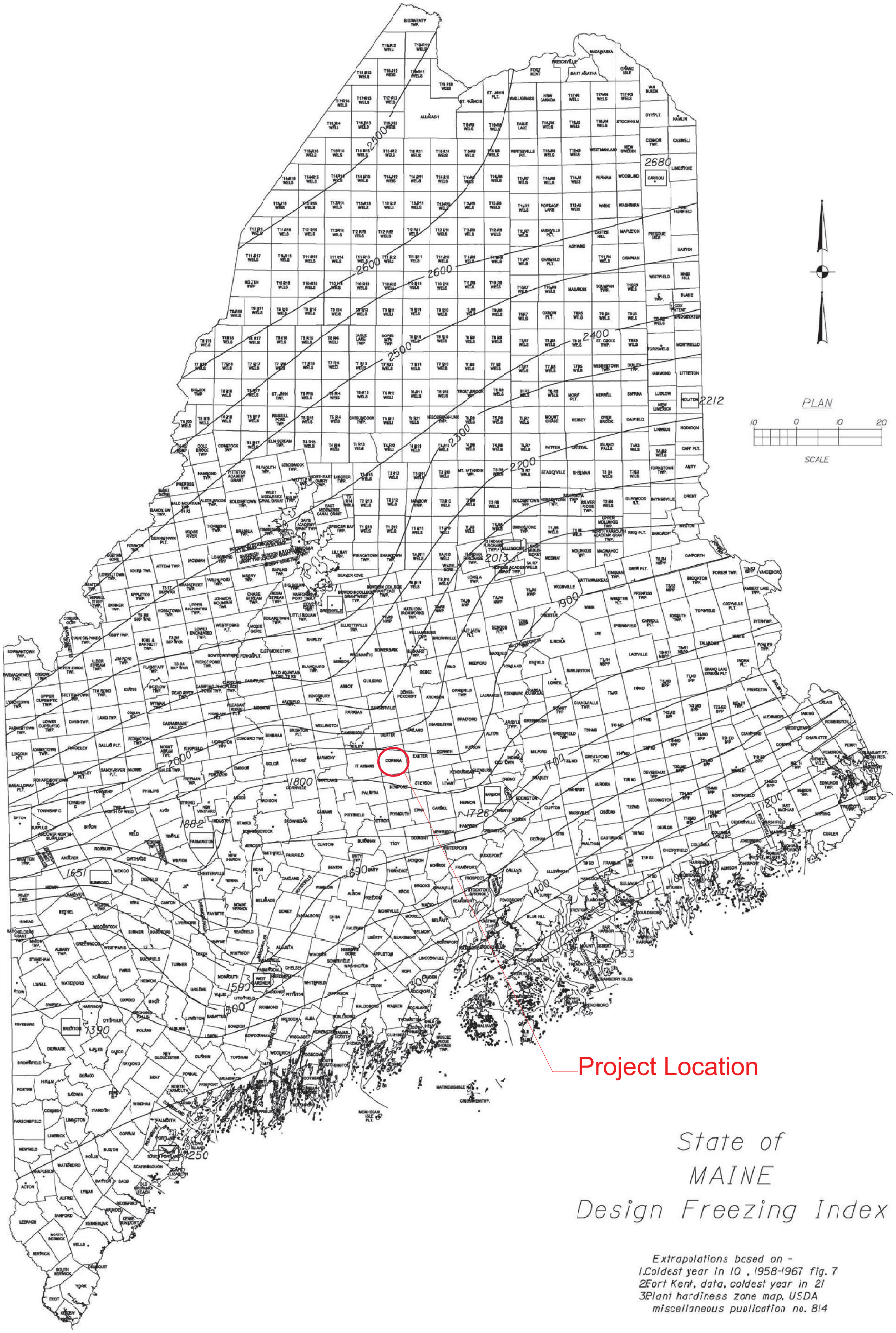


Figure 5-1 Maine Design Freezing Index Map

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