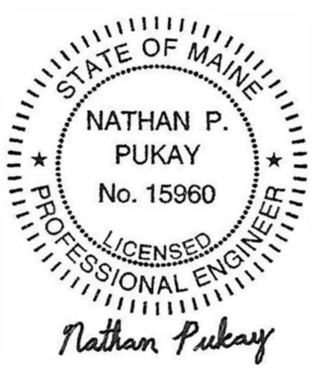
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CLIFFORD BRIDGE
STATE ROUTE 86 OVER CLIFFORD STREAM
MARION TOWNSHIP, MAINE



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Washington County — Soils Report 2023-23
WIN 25211.00 — Bridge No. 5223

Federal Project No. 2521100
October 23, 2023

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Clifford Bridge which carries State Route 86 over Clifford Stream in Marion Township, 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 Clifford Bridge was installed in 1991, and it replaced a steel arch on a timber mat foundation constructed circa 1950. The structure consists of a single 12-foot 10-inch span by 8-foot 4-inch rise, steel plate pipe arch bearing on 1-foot of gravel borrow. According to the 2021 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge is structurally deficient. The culvert is rated a 3 (serious condition) due to unzipping of the culvert ends and reduced wall strength from corrosion.

The proposed replacement structure is a 26-foot span by 9-foot rise, 80-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 culvert invert will be embedded approximately 3 feet into the streambed, and 2 feet of special fill will be placed inside the bottom of the culvert and across the riprap aprons to create a natural streambed. Stream channel rock and streambed rock features will be installed on the special fill to facilitate fish passage. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill.

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. Construction will be staged to accommodate alternating one lane traffic through the project site.

2.0 GEOLOGIC SETTING

Clifford Bridge carries State Route 86 over Clifford Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Eastport Quadrangle, Open-File No. 87-10 (1987), indicates the surficial soils in the vicinity of the bridge project consist of glaciomarine deposits and glacial till. Glaciomarine deposits consist of silt, clay, sand, and minor amounts of gravel. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones.

The MGS Reconnaissance Bedrock Geology of the Gardner Lake Quadrangle, Open File No. 78-3 (1978) maps the bedrock in the vicinity of the project as an unnamed Devonian, intrusive diorite.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the project location. Boring BB-MTCS-101 was drilled west of the existing culvert, and boring BB-MTCS-102 was drilled east of the existing culvert. Both borings terminated in bedrock cores. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled in June 2022 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were performed by using a combination of solid stem auger and cased wash boring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. 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 September 2021. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.974 to the raw field N-values. This hammer efficiency factor (0.974) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, reviewed the field logs for accuracy, 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 three standard grain size analyses with natural water content and two grain size analyses with hydrometer and natural water content. The results of 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 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill and Glacial Till. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of Fill was encountered in the test borings. The thickness encountered was approximately 12 to 13 feet. The fill materials encountered consisted of:

- Brown, Sandy GRAVEL, little silt;
- Brown-grey, Gravelly SAND, trace silt;
- Brown, SAND, some gravel, trace silt;
- Grey, GRAVEL, little sand, little silt.

Corrected SPT N-values in the fill layer ranged from 26 to greater than 50 blows per foot (bpf), indicating the fill is medium dense to very dense in consistency. Two grain size analyses conducted on samples of the fill indicated the material is classified as A-1-a under the AASHTO Soil Classification System and SW-SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 4 to 10 percent.

5.2 Glacial Till

Glacial Till was encountered in the test borings beneath the fill soils. The encountered thickness was approximately 8 feet. The deposit consisted of:

- Grey, GRAVEL, some sand, little silt, trace clay;
- Grey, SAND, some gravel, some silt, trace clay;
- Brown, Sandy SILT, trace gravel.

Corrected SPT N-values in the coarse-grained Glacial Till ranged from 26 to greater than 50 bpf, indicating the deposit is medium dense to very dense in consistency.

One corrected SPT N-value in the fine-grained Glacial Till was greater than 50 bpf, indicating the deposit is hard in consistency.

Three grain size analyses indicate the material is classified as A-2-4 and A-4 under the AASHTO Soil Classification System and GC-GM, SC-SM, and CL under the USCS. The natural water contents of the samples tested ranged from approximately 7 to 12 percent.

5.3 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-MTCS-101	9+88.9	8.5 Rt	20.9	93.3	28, 79
BB-MTCS-102	10+10.7	7.7 Lt	21.3	92.4	19, 22, 0

The bedrock at the site is identified as grey to dark grey, fine to medium-grained, DIORITE, hard, fresh to moderately weathered, with competent and fractured zones, joints at low angles to steeply dipping, spaced very close to moderately close, some joint healing, with frequent oxidation staining and some rock flour on the joint faces. The RQD of the bedrock ranged from 0 to 79 percent corresponding to a Rock Quality of Very Poor to Good. Detailed bedrock descriptions and RQD are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Rock core photographs are included in Appendix B – Rock Core Photographs.

5.4 Groundwater

Groundwater was measured in one boring at a depth of 8 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

The previous steel culverts installed at the project site had shorter than anticipated service lives, therefore a steel replacement structure was not considered. Due to sufficient overburden at the bridge location, along with consultations with the MaineDOT environmental staff and Maine Department of Inland Fisheries and Wildlife, 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 economic advantages.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 80-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 3 feet into the streambed, and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. 3-foot thick riprap aprons will be constructed at the inlet and outlet. The riprap aprons will be covered with the engineered streambed material to provide continuity of the natural streambed. Stream channel rock and streambed rock features will be installed on the special fill to facilitate fish passage.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials Load Resistance and Factor Design Bridge Design Specifications, 9th Edition, 2020.

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to calculate earth loads and earth pressures from the soil envelope. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

The box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill conforming to Standard Specification 703.19. The 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.

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill with a slab bottom elevation of approximately El. 100.

For a precast concrete box culvert with a base width of 28 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 23 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 8 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 99 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 medium dense to very dense sand and gravel (glacial till). Any unsuitable soils (i.e. low strength silts and clays) that may be encountered at subgrade elevation should be excavated down to expose competent, firm material and replaced with compacted granular borrow. Prior to placing and compacting the granular borrow bedding layer the subgrade shall be proof compacted. These recommendations should be included in the contract documents as a General Note.

7.3 Settlement

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

Considering the subgrade preparation recommendations in Section 7.2 of this report, all loose or soft soils encountered at the subgrade elevation will be excavated, replaced with compacted granular borrow 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, Marion Township has a design freezing index (DFI) of approximately 1800 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 7.5 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 7.5 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 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill, conforming to Standard Specification 703.19. Medium dense to very dense glacial till was encountered in the test borings at the proposed culvert subgrade. Any unsuitable soils that are encountered at the subgrade shall be excavated down to expose competent, firm material and replaced with compacted granular borrow.

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.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Clifford Bridge in Marion Township, 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.

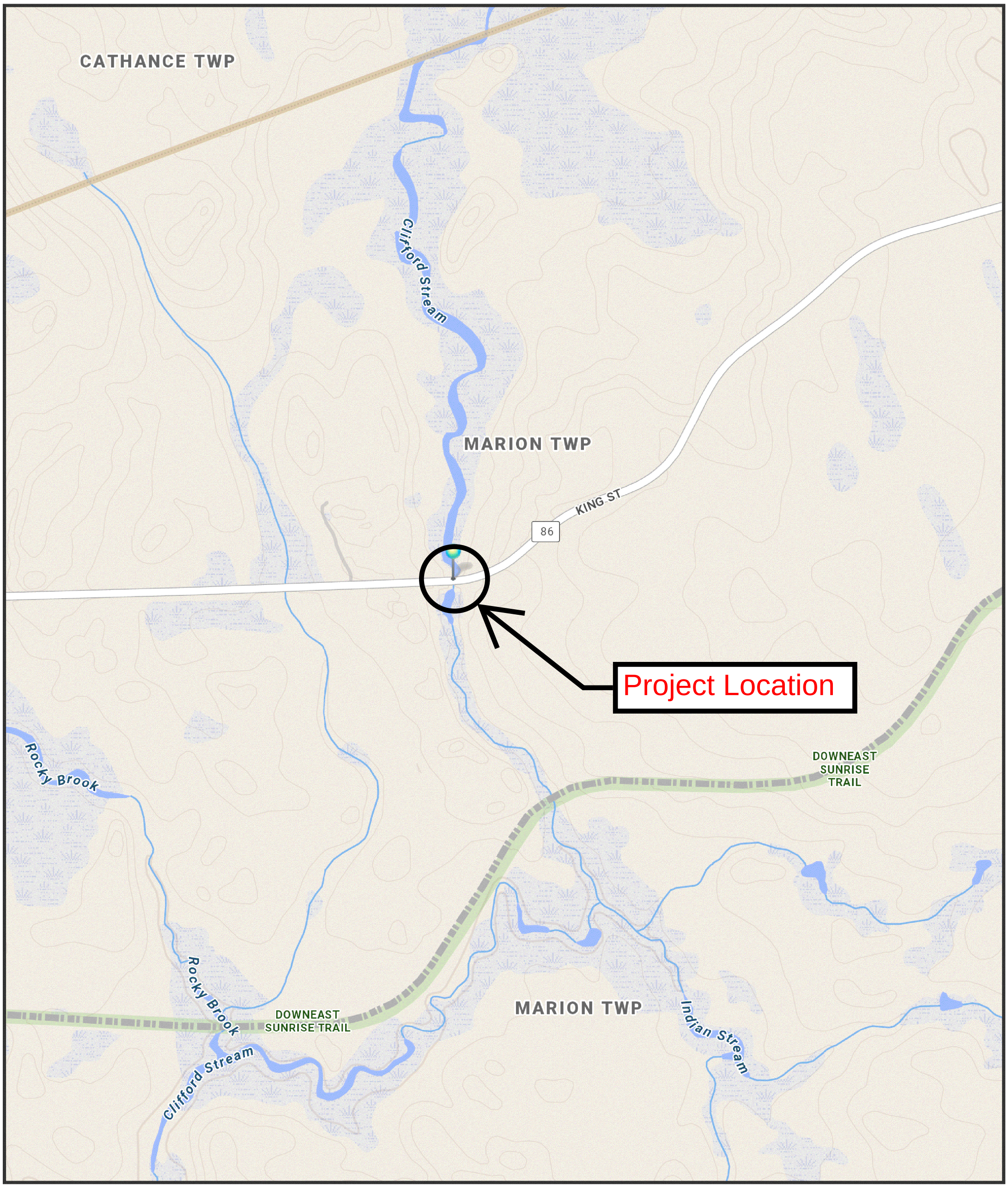
Sheets

Date:7/20/2023

Division: GEOTECH

Username: terry.white

MARION TWP., MAINE



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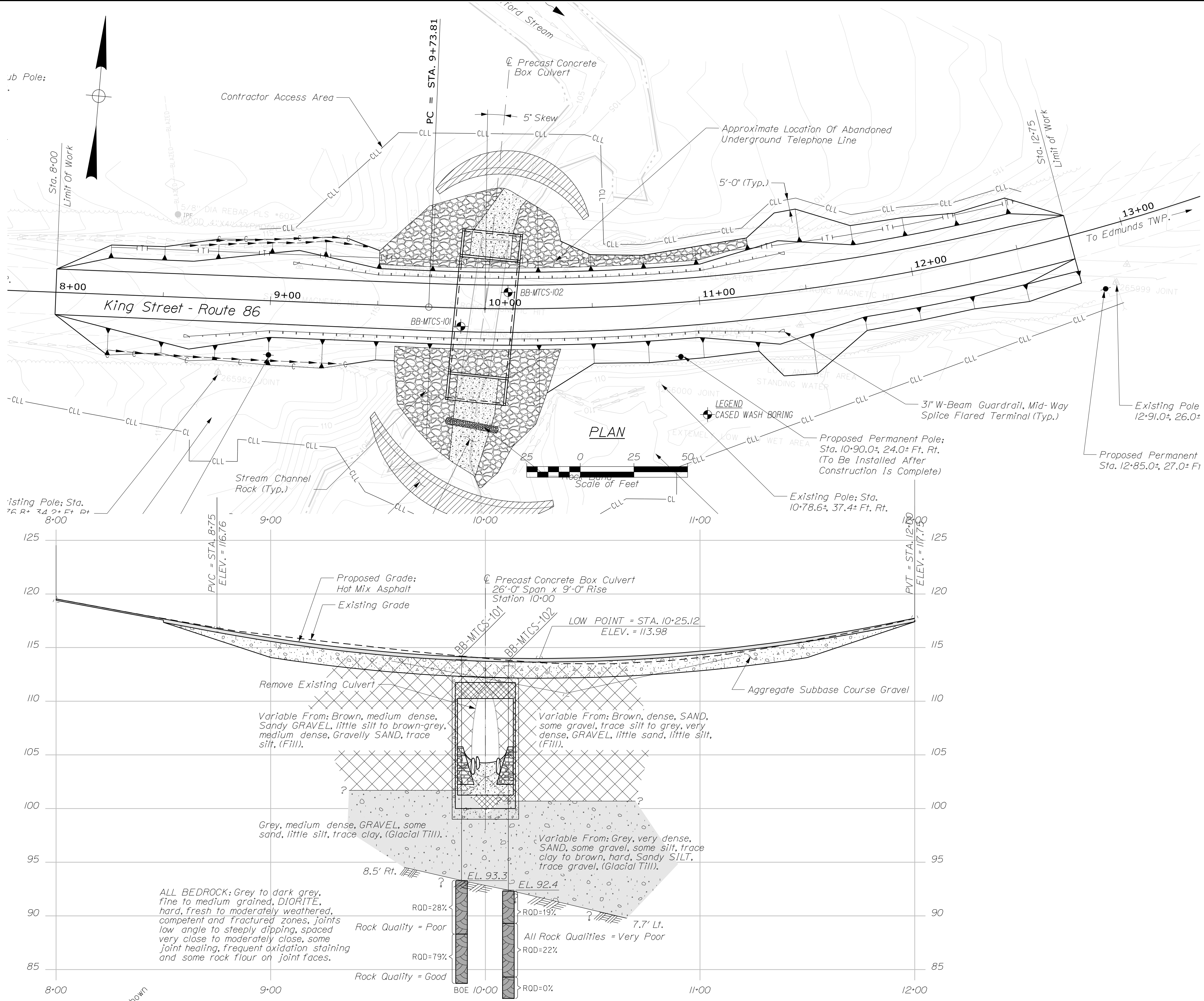
0.25 Miles

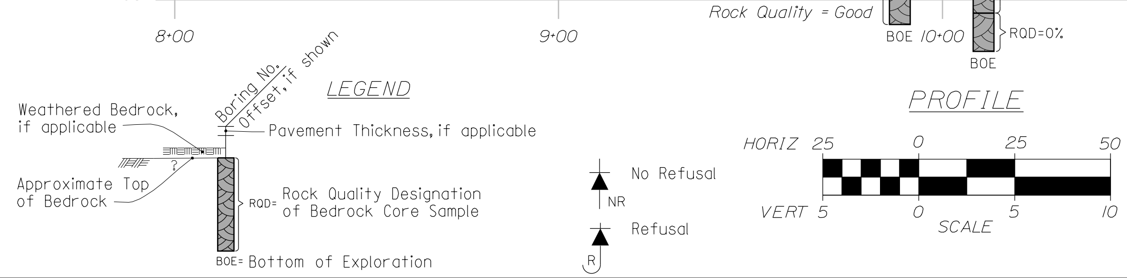
1 inch = 0.28 miles

Date: 7/20/2023
Time: 5:52:48 AM

SHEET NUMBER	CLIFFORD BRIDGE CLIFFORD STREAM MARION TWP. WASHINGTON COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1		2521100
OF 3	LOCATION MAP	WIN BRIDGE NO. 5223 25211.00 BRIDGE PLANS

Filename: ...\GEOTECH\MSTA\005_BLP&ISP1.dgn   Division: GEOTECH   Username: Nathan.P.Pukay   Date:10/23/2023





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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2521100

WIN 25211.00

BRIDGE PLANS

BRIDGE NO. 5223

	BY	DATE	
PROJ. MANAGER	M. PARLIN		
DESIGN-DETAILED	M. GRAY	R. MAYER	
CHECKED-REVIEWED	N. PUKAY	T. WHITE	OCT 2023
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

CLIFFORD BRIDGE
CLIFFORD STREAM
MARION TWP.   WASHINGTON COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\MSTA\006_BORING LOGS1.dgn   Division: GEOTECH   Username: Nathan.P.Pukay   Date:10/23/2023

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-101
WIN: 25211.00

Driller:	MaineDOT	Elevation (ft.)	114.2	Auger ID/OD:	5" Solid Stem
Operator:	Travis/Dustin	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/28/2022; 08:00-10:15	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	9+88.9, 8.5 ft Rt.	Casing ID/OD:	NW(3.0"/3.5")	Water Level*:	8.0 ft bgs. after drilling
Hammer Efficiency Factor: 0.974		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	113.9	4" HMA. —0.3	
5	1D	24/7	5.00 - 7.00	13/7/10/10	17	28	36		Brown, dry, medium dense, Sandy GRAVEL, little silt, (Fill).	
							32			
							31			
							34			
							34			
10	2D	24/9	10.00 - 12.00	6/7/9/22	16	26	20		Brown-grey, wet, medium dense, Gravelly SAND, trace silt, (Fill).	G#241518 A-1-a, SW-SM WC=9.5%
							43			
							128	101.7	—12.5	
							120			
							63			
15	3D	24/10	15.00 - 17.00	13/11/5/8	16	26	29		Grey, wet, medium dense, GRAVEL, some sand, little silt, trace clay, (Glacial Till).	G#241519 A-2-4, GC-GM WC=7.2%
							29			
							36			
							54			
							77			
20	4D R1	10.8/4 60/60	20.00 - 20.90 20.90 - 25.90	10/70(4.8") RQD = 28%	---		a84 NQ-2	93.3	a84 blows for 0.9 ft. GRAVEL (Bedrock Chips), trace silt. Roller Coned to 20.9 ft bgs. —20.9 Top of Bedrock at Elev. 93.3 ft. R1: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, fresh, competent, steep joints, slight oxidation staining on joint faces, then Grey, fine to medium-grained, DIORITE, hard, slightly weathered, highly fractured, steep joints, very closely spaced, with increased oxidation staining on joint faces. Rock Quality = Poor. R1: Core Times (min:sc) 20.9-21.9 ft (2:09) 21.9-22.9 ft (1:36) 22.9-23.9 ft (1:25) 23.9-24.9 ft (1:17) 24.9-25.9 ft (1:16) 100% Recovery	
25	R2	55.6/ 55.6	25.90 - 30.53	RQD = 79%					R2: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, fresh, competent, low angle joints, spaced moderately close, then Grey, fine to medium-grained, DIORITE, hard, brittle, slightly weathered, low angle joints, closely spaced, with some joint healing. Rock Quality = Good. R2: Core Times (min:sec) 25.9-26.9 ft (1:53) 26.9-27.9 ft (1:31) 27.9-28.9 ft (1:49) 28.9-29.9 ft (1:50) 29.9-30.5 ft (1:38) 100% Recovery	
30								83.7	—30.5 Bottom of Exploration at 30.5 feet below ground surface.	
35										
40										
45										
50										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-MTCS-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-102
WIN: 25211.00

Driller:	MaineDOT	Elevation (ft.)	113.7	Auger ID/OD:	5" Solid Stem
Operator:	Travis/Dustin	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	N. Pukay	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/28/2022; 10:30-13:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+10.7, 7.7 ft Lt.	Casing ID/OD:	NW(3.0"/3.5")	Water Level*:	None Observed
Hammer Efficiency Factor: 0.974		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	113.3	5" HMA. —0.4	
5	1D	24/18	5.00 - 7.00	10/13/9/12	22	36	45		Brown, dry, dense, SAND, some gravel, trace silt, (Fill).	G#241520 A-1-a, SW-SM WC=4.2%
							63			
							43			
							46			
							23			
10	2D	24/3	10.00 - 12.00	6/11/27/17	38	62	9		Grey, wet, very dense, GRAVEL, little sand, little silt, (Fill).	
							22			
							53	100.7	—13.0	
							52			
							75			
15	3D	24/13	15.00 - 17.00	14/24/18/12	42	68	25		Grey, wet, very dense, SAND, some gravel, some silt, trace clay, (Glacial Till).	G#241524 A-2-4, SC-SM WC=8.4%
							47			
							64			
							101			
							217			
20	4D	10.8/8	20.00 - 20.90	14/50(4.8")	---		230		Brown, damp, hard, Sandy SILT, trace gravel, (Glacial Till). Bedrock in tip of spoon. —21.3	G#241521 A-4, CL WC=11.5%
	R1	36/34	21.40 - 24.40	RQD = 19%			RC NQ-2	92.4 92.3	Drove NW Casing to 21.3 ft bgs. Top of Bedrock at Elev. 92.4 ft. Roller Coned to 21.4 ft bgs. —21.4 R1: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, moderately weathered, multiple fracture zones, steep joints, spaced very close to close, joint faces moderately weathered with oxidation staining, several annealed fractures. Rock Quality = Very Poor. R1: Core Times (min:sec) 21.4-22.4 ft (2:22) 22.4-23.4 ft (1:49) 23.4-24.4 ft (2:03) Core Blocked 94% Recovery	
25	R2	60/53	24.40 - 29.40	RQD = 22%					R2: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, slightly weathered with a moderately weathered fractured zone at the bottom of run, steep joints with oxidation staining, closely spaced, frequent joint healing, fresh rock flour along several joint faces. Rock Quality = Very Poor. R2: Core Times (min:sec) 24.4-25.4 ft (1:40) 25.4-26.4 ft (1:24) 26.4-27.4 ft (1:30) 27.4-28.4 ft (1:49) 28.4-29.4 ft (1:37) 88% Recovery	
30	R3	24/21	29.40 - 31.40	RQD = 0%				82.3	R3: Bedrock: Grey, fine to medium-grained, DIORITE, hard, moderately weathered, highly fractured, steep joints, spaced very close, frequent joint healing, oxidation staining and fresh rock flour on several joint faces. Rock Quality = Very Poor. R3: Core Times (min:sec) 29.4-30.4 ft (2:22) 30.4-31.4 ft (3:31) 88% Recovery —31.4 Bottom of Exploration at 31.4 feet below ground surface.	
35										
40										
45										
50										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-MTCS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2521100

WIN
25211.00

BRIDGE PLANS

BRIDGE NO. 5223

PROJ. MANAGER	M. PARLIN	BY	DATE
DESIGN-DETAILED	M. GRAY	R. MAYER	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	N.PUKAY	T.WHITE	OCT 2023
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

CLIFFORD BRIDGE
CLIFFORD STREAM
MARION TWP. WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER

3

OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

*Minimum NQ rock core (1.88 in. OD of core)

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

WIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-101

WIN: 25211.00

Driller:	MaineDOT	**Elevation (ft.)**	114.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/Dustin	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/28/2022; 08:00-10:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+88.9, 8.5 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	8.0 ft bgs. after drilling
Hammer Efficiency Factor:	0.974	**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	113.9		4" HMA. 0.3	
5	1D	24/7	5.00 - 7.00	13/7/10/10	17	28	36			Brown, dry, medium dense, Sandy GRAVEL, little silt, (Fill).	
							32				
							31				
							34				
							34				
10	2D	24/9	10.00 - 12.00	6/7/9/22	16	26	20			Brown-grey, wet, medium dense, Gravelly SAND, trace silt, (Fill).	G#241518 A-1-a, SW-SM WC=9.5%
							43				
							128	101.7		12.5	
							120				
							63				
15	3D	24/10	15.00 - 17.00	13/11/5/8	16	26	29			Grey, wet, medium dense, GRAVEL, some sand, little silt, trace clay, (Glacial Till).	G#241519 A-2-4, GC-GM WC=7.2%
							29				
							36				
							54				
							77				
20	4D	10.8/4	20.00 - 20.90	10/70(4.8")	---		a84			a84 blows for 0.9 ft. GRAVEL (Bedrock Chips), trace silt.	
	R1	60/60	20.90 - 25.90	RQD = 28%			NQ-2	93.3		Roller Coned to 20.9 ft bgs. 20.9	
										Top of Bedrock at Elev. 93.3 ft. R1: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, fresh, competent, steep joints, slight oxidation staining on joint faces, then Grey, fine to medium-grained, DIORITE, hard, slightly weathered, highly fractured, steep joints, very closely spaced, with increased oxidation staining on joint faces. Rock Quality = Poor.	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-101
WIN: 25211.00

Driller:	MaineDOT	**Elevation (ft.)**	114.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/Dustin	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/28/2022; 08:00-10:15	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	9+88.9, 8.5 ft Rt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	8.0 ft bgs. after drilling

Hammer Efficiency Factor: 0.974 **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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	55.6/55.6	25.90 - 30.53	RQD = 79%						R1: Core Times (min:sec) 20.9-21.9 ft (2:09) 21.9-22.9 ft (1:36) 22.9-23.9 ft (1:25) 23.9-24.9 ft (1:17) 24.9-25.9 ft (1:16) 100% Recovery	
										R2: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, fresh, competent, low angle joints, spaced moderately close, then Grey, fine to medium-grained, DIORITE, hard, brittle, slightly weathered, low angle joints, closely spaced, with some joint healing. Rock Quality = Good.	
30								83.7		R2: Core Times (min:sec) 25.9-26.9 ft (1:53) 26.9-27.9 ft (1:31) 27.9-28.9 ft (1:49) 28.9-29.9 ft (1:50) 29.9-30.5 ft (1:38) 100% Recovery	
										30.5	
										Bottom of Exploration at 30.5 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-102
WIN: 25211.00

Driller:	MaineDOT	**Elevation (ft.)**	113.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/Dustin	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/28/2022; 10:30-13;30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+10.7, 7.7 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.974	**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
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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
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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	113.3		5" HMA. 0.4	
5	1D	24/18	5.00 - 7.00	10/13/9/12	22	36	45			Brown, dry, dense, SAND, some gravel, trace silt, (Fill).	G#241520 A-1-a, SW-SM WC=4.2%
							63				
							43				
							46				
							23				
10	2D	24/3	10.00 - 12.00	6/11/27/17	38	62	9			Grey, wet, very dense, GRAVEL, little sand, little silt, (Fill).	
							22				
							53	100.7		13.0	
							52				
							75				
15	3D	24/13	15.00 - 17.00	14/24/18/12	42	68	25			Grey, wet, very dense, SAND, some gravel, some silt, trace clay, (Glacial Till).	G#241524 A-2-4, SC-SM WC=8.4%
							47				
							64				
							101				
							217				
20	4D	10.8/8	20.00 - 20.90	14/50(4.8")	---		230			Brown, damp, hard, Sandy SILT, trace gravel, (Glacial Till). Bedrock in tip of spoon.	G#241521 A-4, CL WC=11.5%
	R1	36/34	21.40 - 24.40	RQD = 19%			RC NQ-2	92.4 92.3		21.3 Drove NW Casing to 21.3 ft bgs. Top of Bedrock at Elev. 92.4 ft. Roller Coned to 21.4 ft bgs. 21.4	
										R1: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, moderately weathered, multiple fracture zones, steep joints, spaced very close to close, joint faces moderately weathered with	
25	R2	60/53	24.40 - 29.40	RQD = 22%							

Remarks:

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Clifford Bridge #5223 carries Route 86 over Clifford Stream
Location: Marion Township

Boring No.: BB-MTCS-102
WIN: 25211.00

Driller:	MaineDOT	**Elevation (ft.)**	113.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Travis/Dustin	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	N. Pukay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/28/2022; 10:30-13;30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+10.7, 7.7 ft Lt.	**Casing ID/OD:**	NW(3.0"/3.5")	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.974	**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										oxidation staining, several annealed fractures. Rock Quality = Very Poor. R1: Core Times (min:sec) 21.4-22.4 ft (2:22) 22.4-23.4 ft (1:49) 23.4-24.4 ft (2:03) Core Blocked 94% Recovery	
										R2: Bedrock: Grey to dark grey, fine to medium-grained, DIORITE, hard, slightly weathered with a moderately weathered fractured zone at the bottom of run, steep joints with oxidation staining, closely spaced, frequent joint healing, fresh rock flour along several joint faces. Rock Quality = Very Poor. R2: Core Times (min:sec) 24.4-25.4 ft (1:40) 25.4-26.4 ft (1:24) 26.4-27.4 ft (1:30) 27.4-28.4 ft (1:49) 28.4-29.4 ft (1:37) 88% Recovery	
	R3	24/21	29.40 - 31.40	RQD = 0%							
30											
								82.3		R3: Bedrock: Grey, fine to medium-grained, DIORITE, hard, moderately weathered, highly fractured, steep joints, spaced very close, frequent joint healing, oxidation staining and fresh rock flour on several joint faces. Rock Quality = Very Poor. R3: Core Times (min:sec) 29.4-30.4 ft (2:22) 30.4-31.4 ft (3:31) 88% Recovery 31.4	
35											
										Bottom of Exploration at 31.4 feet below ground surface.	
40											
45											
50											

Remarks:

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

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

<u>Appendix B</u>

Rock Core Photographs

MaineDOT

Clifford Bridge #5223 Carries Route 86 Over Clifford Stream

Marion Township, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-MTCS-101	R1	20.9-25.9	60	60	17	28	DIORITE	1
BB-MTCS-101	R2	25.9-30.5	55.6	55.6	44	79	DIORITE	2
BB-MTCS-102	R1	21.4-24.4	36	34	7	19	DIORITE	3
BB-MTCS-102	R2	24.4-29.4	60	53	13	22	DIORITE	3+4
BB-MTCS-102	R3	29.4-31.4	24	21	0	0	DIORITE	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.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Marion Township Work Number: 25211.00

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-MTCS-101, 2D	9+88.9	8.5 Rt.	10.0-12.0	241518	1	9.5			SW-SM	A-1-a	0
BB-MTCS-101, 3D	9+88.9	8.5 Rt.	15.0-17.0	241519	1	7.2			GC-GM	A-2-4	II
BB-MTCS-102, 1D	10+10.7	7.7 Lt.	5.0-7.0	241520	1	4.2			SW-SM	A-1-a	0
BB-MTCS-102, 3D	10+10.7	7.7 Lt.	15.0-17.0	241524	1	8.4			SC-SM	A-2-4	III
BB-MTCS-102, 4D	10+10.7	7.7 Lt.	20.0-20.9	241521	1	11.5			CL	A-4	IV

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.

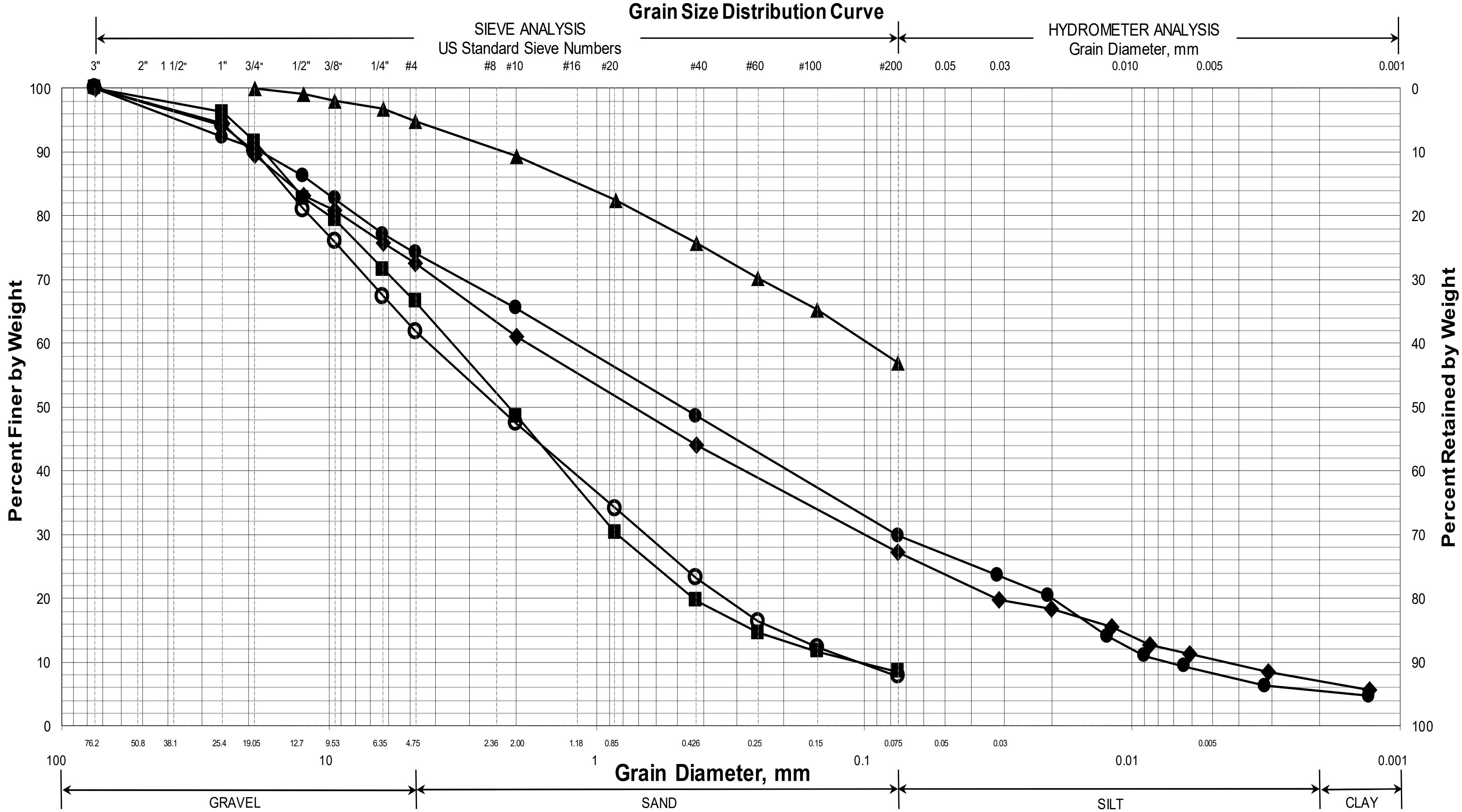
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-MTCS-101/2D	9+88.9	8.5 RT	10.0-12.0	Gravelly SAND, trace silt.	9.5			
◆	BB-MTCS-101/3D	9+88.9	8.5 RT	15.0-17.0	GRAVEL, some sand, little silt, trace clay.	7.2			
■	BB-MTCS-102/1D	10+10.7	7.7 LT	5.0-7.0	SAND, some gravel, trace silt.	4.2			
●	BB-MTCS-102/3D	10+10.7	7.7 LT	15.0-17.0	SAND, some gravel, some silt, trace clay.	8.4			
▲	BB-MTCS-102/4D	10+10.7	7.7 LT	20.0-20.9	Sandy SILT, trace gravel.	11.5			
×									

WIN	
025211.00	
Town	
Marion Twp	
Reported by/Date	
WHITE, TERRY A	8/15/2022

SHEET 1

Appendix D

Calculations

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

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

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

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

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

Outlet Walls Fixed to Box
At-Rest Earth Pressure - Rankine Theory

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

$$\boxed{K_o = 0.47}$$

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

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

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

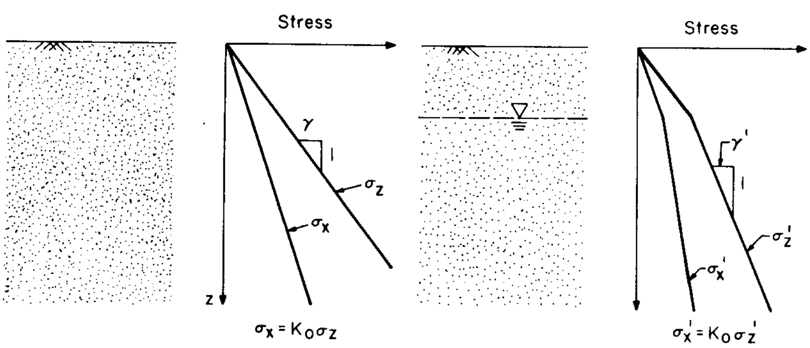


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

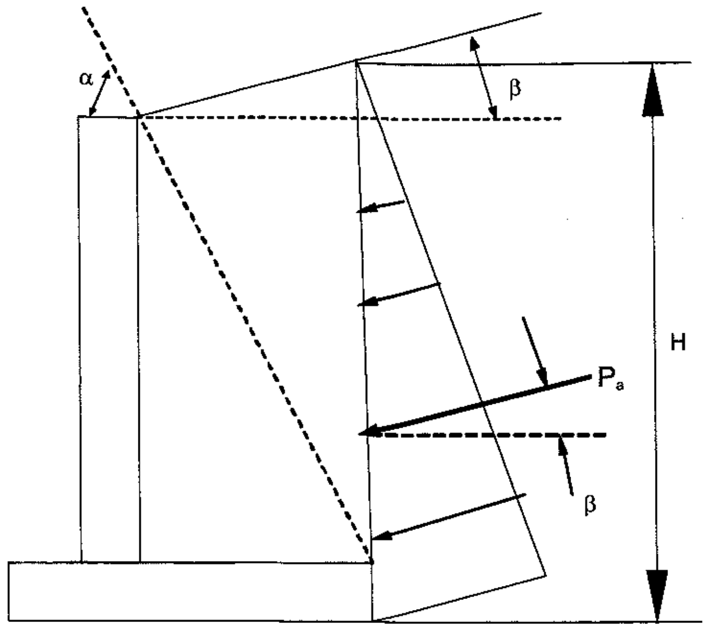


Figure 3-3 Rankine Theory

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 2' into the streambed, accounting for the possible scouring away of 1 foot of special fill.
2. The proposed bearing elevation is approximately 100 feet.
3. Proposed finish roadway grade elevation is approximately 114 feet at the low point.
4. Proposed precast concrete box base is 28 feet wide. The box culvert span is 26 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 9th Edition Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2*, *May 1983*, *Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)		
		Ordinary Range	AASHTO Recommended Value of Use	MaineDOT Recommended Value
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Medium dense to dense	4-8	5	8

Recommend 8 ksf to limit settlement on medium dense to dense 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

$B := 28ft$

$D_f := 2 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

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

$\gamma_{above\_moist} := 125 \cdot pcf$ MaineDOT Bridge Design Guide p. 3-3
Soil Type 4 (granular borrow)

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

Foundation soils:

Foundation soils properties based on BB-MTCS-101 (3D) and -102 (3D)

$\gamma_{below\_dry} := 134 \cdot pcf$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2, Glacial Till

$w_{sat} := 7.2\%$ From BB-MTCS-101 (3D)

$\gamma_{below\_sat} := \gamma_{below\_dry} \cdot (1 + w_{sat})$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit weight relationships

$\gamma_{below\_sat} = 143.6 \cdot pcf$

$\gamma_{below\_moist} := 139pcf$ Moist unit weight estimated between dry and saturated unit weights

$\phi := 36 \cdot deg$ Friction angle of subgrade soils encountered in borings, using correlations: (1) Table 11.3 (HOUGH) and (2) Table 2-6, (BOWLES 5th Ed.).

$c := 0psf$ BB-MTCS-101 (3D) Medium Dense Sand and Gravel (36 deg)
BB-MTCS-102 (3D) Dense Sand and Gravel (40 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)

$N_c := 50.6$

$N_q := 37.8$

$N_\gamma := 56.3$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$L := 80 \cdot ft$ Length of base slab

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

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

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

$s_c = 1.3$ $\quad$ $s_\gamma = 0.9$ $\quad$ $s_q = 1.3$

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

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

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

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

$N_{cm} = 63.8$ $\quad$ $N_{\gamma m} = 48.4$ $\quad$ $N_{qm} = 47.4$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 53 \cdot ksf$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$\boxed{q_r = 23.9 \cdot ksf}$$

Recommend a factored bearing resistance of 23 ksf for box bottom slabs 28 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	Dry unit weight, γ_d kN/m³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

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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.)	$\phi_{cv}(°)$	$\tan \phi_{cv}$	$\phi(°)$	$\tan \phi$	$\phi(°)$	$\tan \phi$
Silt (nonplastic)	26	1 on 2	26	0.488	28	0.532	30	0.577
	to		to		to		to	
	30	1 on 1.75	30	0.577	32	0.625	34	0.675
Uniform fine to medium sand	26	1 on 2	26	0.488	30	0.577	32	0.675
	to		to		to		to	
	30	1 on 1.75	30	0.577	34	0.675	36	0.726
Well-graded sand	30	1 on 1.75	$\boxed{30}$	0.577	34	0.675	38	0.839
	to		to		to		to	
	34	1 on 1.50	34	0.675	40	0.839	46 GLACIAL TILL	1.030
Sand and gravel	32	1 on 1.60	$\boxed{32}$	0.625	36	0.726	40	0.900
	to		to		to		to	
	36	1 on 1.40	36	0.726	42	0.900	48	1.110

LOOSE $28 \le \phi \le 30$
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]	g/cu cm γ_{sat}[c]	lb/cu ft γ_d	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

$\gamma \cong 122$

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

$\gamma = 130$ 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

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 Geomechanic Rock Mass Rating System, 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 $0.3f'_c$.

10.5.5.2.2—Spread Footings

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

C10.5.5.2.2

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

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

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

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

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

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:

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

in which:

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

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

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

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

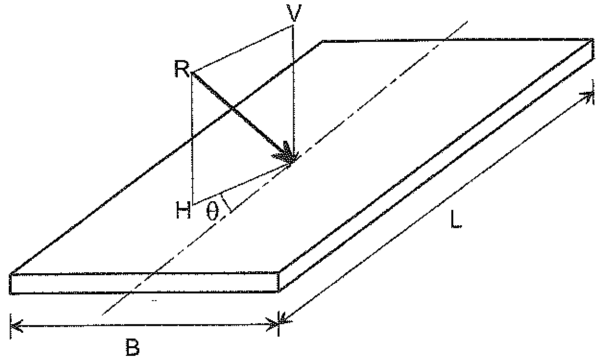


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ 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
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

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

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:

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

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

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.

Modulus of Subgrade Reaction

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

Given:
1. Limited lab data and SPT N-values.

Assumptions:
1. The proposed bearing elevation of the base slab is approximately 100 feet.
2. Proposed finished roadway grade is approximately 114 feet.
3. Proposed precast concrete box is 28 feet wide. The box culvert span is 26 feet with the walls assumed to be 1 foot thick based on previous culverts of similar size.
4. Proposed precast box culvert is 80 feet long.
5. The subsurface conditions present at the proposed bearing elevation is glacial till comprised of medium dense, GRAVEL, some sand little silt, trace clay (BB-MTCS-101) and very dense, SAND, some gravel, some silt, trace clay (BB-MTCS-102). (Modeled as a medium dense to dense 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
Medium dense sand, average of upper and lower limit: k_s = 165 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Saturated, medium dense, coarse grained glacial till K_{v1}, 254 pci/2 = 127 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:
Typical subgrade reaction values for 0.3 m x 0.3 m plate
Saturated sand, medium dense, 129-147 pci, use upper limit: $k_{0.3}$ (k_1) = 147 pci

Adjust Published values for dimensions of base slab

Published range for medium dense sand subgrade is 99-165 pci

Assume a subgrade modulus of 127 pci using FHWA GEC No. 6, Figure 8-3

Value of k_{s1} = 127 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B - 28 ft, Length L = 80 ft).

Square to rectangle base adjustment:

$$k_{s1} := 127\text{pci} \quad B := 28\text{ft} \quad L := 80\text{ft}$$

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

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

$$k = 99 \cdot \text{pci}$$

Recommend a subgrade modulus of 99 pci

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \tag{9-10}$$

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

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

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

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:

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

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

lepth 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

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

TABLE 9-1
Range of modulus of subgrade reaction k_s

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

Use values as guide and for comparison when using approximate equations

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

Soil	k_s, kN/m³	ks, lb/in^3	
Loose sand	4800–16 000	18 - 59	
Medium dense sand	9600–80 000	35 - 295	165 pci
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:			
$q_a \le 200$ kPa	12 000–24 000	44 - 88	
$200 < q_a \le 800$ kPa	24 000–48 000	88 - 177	
$q_a > 800$ kPa	> 48 000	> 177	

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

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

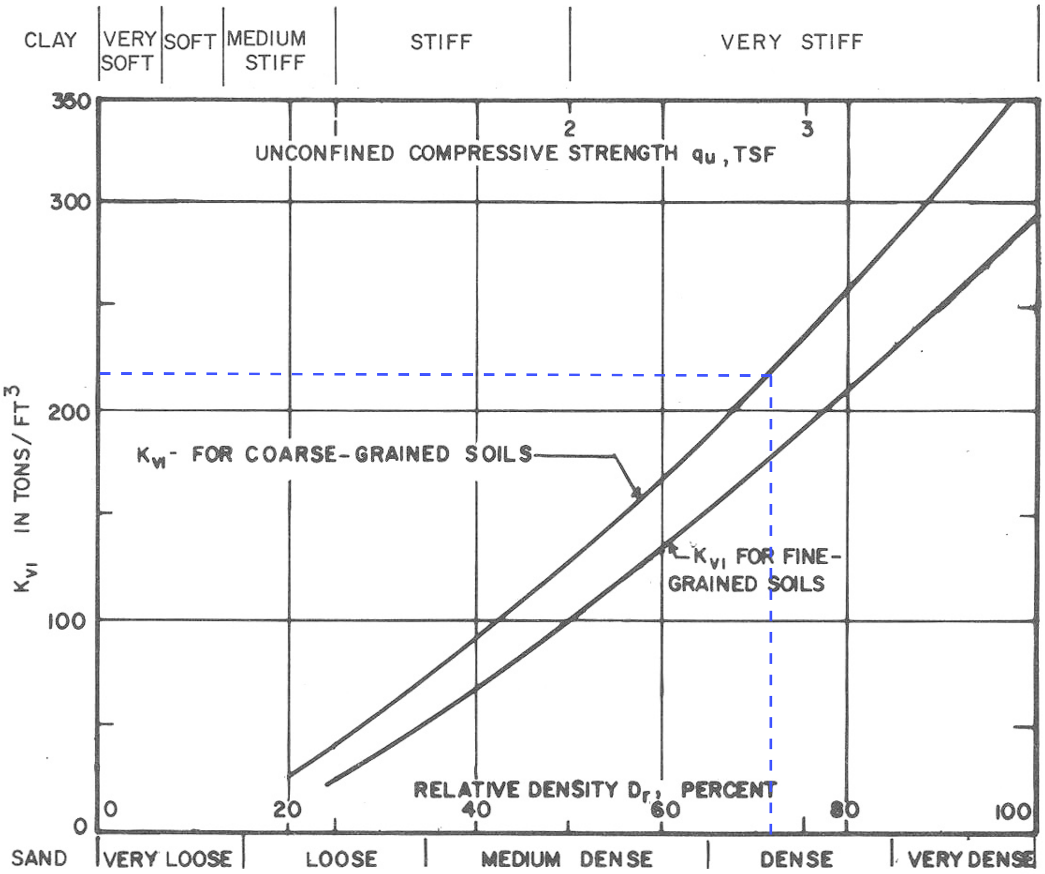
You should note that k_s does not depend on the contact pressure of the base q_o.
For **corner:**

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

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

220 TCF =
254 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 ≦ B

FOR B ≦ 20 FT:

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

FOR B ≧ 40 FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION D ≧ 5B

FOR B ≦ 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)

 Das, Priciples of Foundation Engineering 7th Ed.

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

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

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

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

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

or

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

where

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

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

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

Soil type	$k_{0.3}(k_1)$ MN/m³	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

147 pci

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

Foundations on Sandy Soils

For foundations on sandy soils,

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

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

Foundations on Clays

For foundations on clays,

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

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

Method 1:

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

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

where

k = coefficient of subgrade modulus of the rectangular foundation $(L \times B)$

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

Frost

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

From Design Freezing Index Map: Marion Township, Maine
DFI = 1800 degree-days.
Case 1 - Coarse Grained Soils W=10% (BB-MTCS-101 2D).

For DFI = 1800

at w=10% $d_1 := 90.1\text{in}$ $d_1 := 7.5\text{ft}$ Depth of Frost Penetration

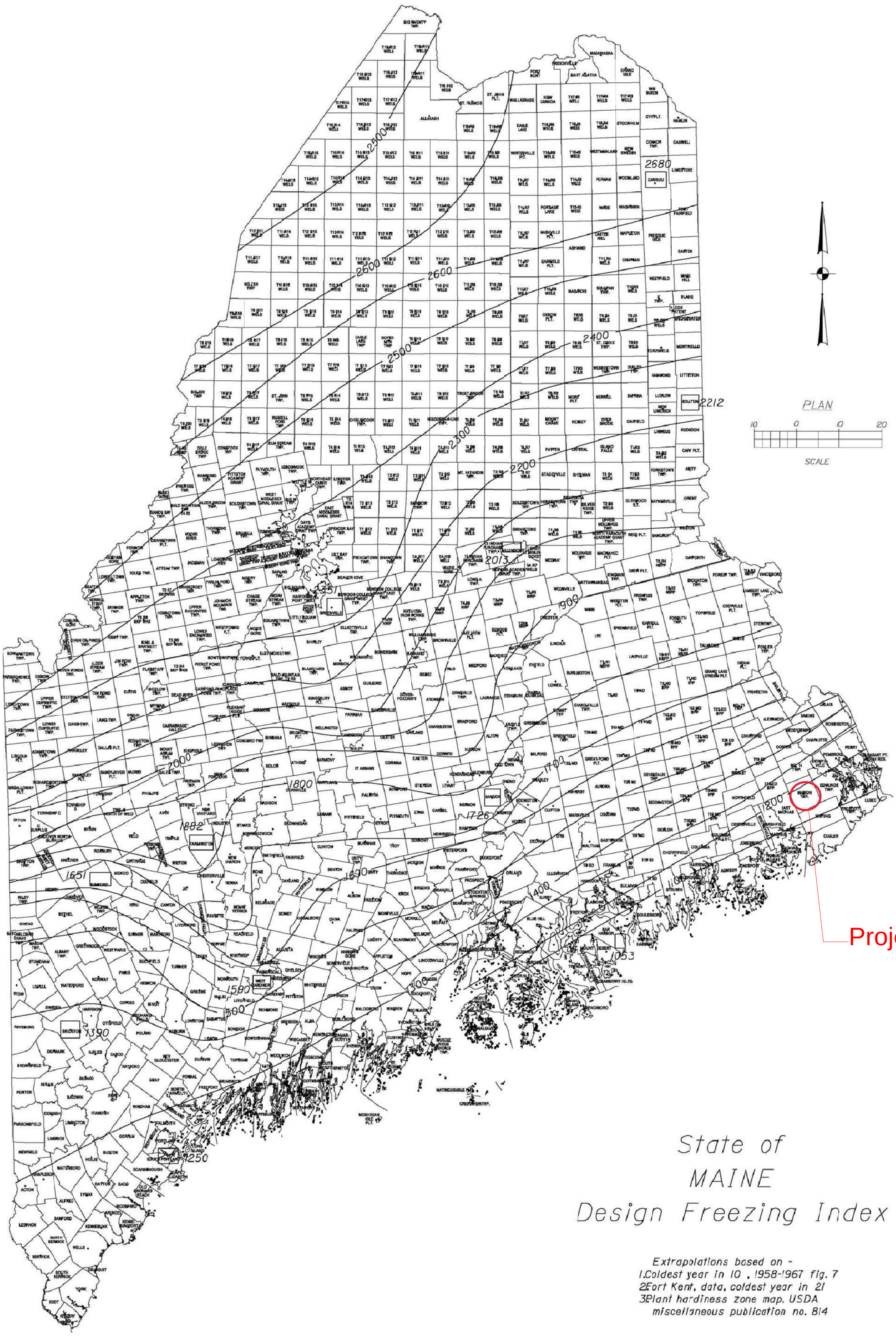


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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