STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Letter of Transmittal

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James Manahan, Geotechnical Bridge Program (Electronic Copy Only)
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Subject: Geotechnical Design Report, Maduskeag Bridge, Hodgdon, Maine

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Bridge #: 2492

Route: U.S. Route 1

WIN: 023625.00

Town: Hodgdon

Attached is one (1) pdf copy of Soils Report 2020-23, "GEOTECHNICAL DESIGN REPORT: For the Replacement of: MADUSKEAG BRIDGE, U.S. ROUTE 1 OVER MADUSKEAG STREAM, HODGDON, MAINE" dated: June 25, 2020.

This report is available in TEDOCS as Document #1884612 .

att: 1 of 2020-23

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MADUSKEAG BRIDGE
U.S. ROUTE 1 OVER MADUSKEAG STREAM
HODGDON, MAINE

Prepared by:

James Manahan
Assistant Transportation Engineer

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Aroostook County
WIN 23625.00

Soils Report 2020-23
Bridge No. 2492

June 25, 2020

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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 Maduskeag Bridge which carries U.S. Route 1 over Maduskeag Stream in Hodgdon, 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 Maduskeag Bridge was constructed in 1932. The structure consists of a single-span, concrete slab supported on mass concrete abutments bearing on soil. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the structure is in overall poor condition and is rated a 4. The superstructure is in poor condition with heavy efflorescence and cracking. There are substantial cracks in the wingwalls and abutments, and the concrete is crumbling in both abutments.

The proposed replacement structure is a 14-foot span and 8.5-foot rise, 134-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 3H:1V (horizontal:vertical) sideslopes. The box culvert invert will be embedded 2 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill. Bedrock was encountered in the borings within 5 feet of the proposed bottom of the box. Rock excavation may be required to provide at least 1-foot of clearance between the bottom of the box culvert and the top of bedrock.

The new box culvert will be located on nearly the same horizontal alignment as the existing bridge with an increased skew. Alternating one-way traffic will be maintained using a temporary special detour on the upstream side with traffic signals.

2.0 GEOLOGIC SETTING

Maduskeag Bridge carries U.S. Route 1 over Maduskeag Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Reconnaissance Surficial Geology Map of the Houlton Quadrangle, Open-File No. 81-9 (1981), indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is typically a loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock, deposited by glacial ice. Glacial till may include lenses of water-laid sand and gravel. Scattered boulders are also common.

The MGS Bedrock Geologic Map of Maine (1985) maps the bedrock at the project site as interbedded pelite and sandstone of the Smyrna Mills Formation.

3.0 SUBSURFACE INVESTIGATION

Three test borings and three probes explored subsurface conditions at the project location. Cased wash borings BB-HMS-102 and BB-HMS-102A were drilled behind the southeast corner of the existing bridge. Cased wash borings BB-HMS-103 was drilled behind the northwest corner of the bridge. Two of these borings terminated in bedrock cores. Probes BB-HMS-101, BB-HMS-101A, and BB-HMS-104 were advanced by solid stem augers. The probes were drilled south and north of the existing bridge. The test boring and probe locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings and probes were drilled in October 2018 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.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring 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 MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in April 2018. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 to the raw field N-values. This hammer efficiency factor (0.928) 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, and reviewed the field logs for accuracy. A MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions encountered in the borings and probes. The borings were located in the field using taped measurements at the completion of the drilling program and surveyed.

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 five standard grain size analyses with 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, Glacial Till and metamorphic Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A fill layer was encountered in the test borings. The thickness was approximately 12 to 13.4 feet. The unit generally consisted of brown, damp, sand, gravel or gravelly sand with little to some silt and occasional cobbles.

Corrected SPT N-values in the fill layer ranged from 11 to 15 blows per foot (bpf), indicating the fill unit is medium dense in consistency. One N-value exceeded 50 bpf and indicated the presence of a cobble. Four grain size analyses of the fill material resulted in the material being classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SM and GM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 10 to 18 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill soils in boring BB-HMS-103 and below the existing abutment concrete footing in BB-HMS-102A. The glacial till deposit encountered generally consisted of:

- Grey, wet, very dense, SAND, some gravel, some silt;
- Olive-brown, wet, very dense, SAND, some silt, some gravel.

Corrected SPT N-values of 76 and 102 bpf were recorded indicating the layer is very dense in consistency. Grain size analyses conducted on samples from the glacial till indicated the material is classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SM under the USCS. The natural water contents of the samples tested ranged from approximately 8 to 12 percent.

5.3 Bedrock

Bedrock was encountered and cored in two of the three borings. Table 1 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)
BB-HMS-102A	6+57.6	11.3 Rt	18.4	465.7	0, 29
BB-HMS-103	6+62.3	11.1 Lt	17.0	467.1	0, 0

Table 1 – Summary of Approximate Bedrock Depth, Approximate Bedrock Elevation, and RQD

The bedrock at the site is identified as Slate, Siltstone, and Quartzite. The RQD of the bedrock ranged from 0 to 29 percent corresponding to a rock quality of very poor to poor. Detailed bedrock descriptions and RQD are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Rock core photographs are included in Appendix B – Rock Core Photographs.

5.4 Groundwater

Groundwater was measured at depths ranging from 9.5 to 10 feet below the roadway surface upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured level may not represent stabilized groundwater elevation. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Given the span length of 14 feet, it was determined that a buried structure alternative would be the most suitable structure type to satisfy the purpose and need of this bridge replacement project. A precast concrete box culvert was chosen because of the structure’s durability, ease and speed of construction, and economic advantage compared to similar buried structures. Therefore, a 134-foot long precast concrete box culvert with a 14-foot span and 8.5-foot rise is the preferred replacement structure. The box will be embedded approximately 2 feet into the streambed and 2 feet of special fill will be placed inside to create a natural streambed.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 134-foot-long precast concrete box culvert with slope tapered inlet and outlet walls. The box culvert will have 1-foot tall precast headwalls. To prevent undermining, the box culvert will have 2-foot tall inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 2 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. 2-foot thick riprap aprons should be constructed at the inlet and outlet and should be embedded a minimum of 6 inches into the streambed. The riprap aprons will be covered with the engineered streambed material to provide continuity of the natural streambed.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2017 with 2018 interims (LRFD).

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

The box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill conforming to Standard Specification 703.19. The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The precast concrete box culvert shall be installed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification Section 534.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1-foot thick by 1-foot high concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert's outlet and inlet walls will be slope-tapered to match the sideslopes. 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 documentation.

7.1.3 Precast Concrete Inlet and Outlet Toe Walls

Toe walls shall extend below the bottom slab connecting the left and right walls at the inlet and outlet of the box culvert to prevent undermining per MaineDOT BDG Section 8.3.1. The inlet and outlet toe walls should extend a minimum of 1 foot below the maximum depth of scour.

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill with a bottom elevation ranging from approximately El. 470.0 to 468.7 feet. Rock excavation may be required to provide at least 1-foot of clearance between the bottom of the box culvert and the top of bedrock if bedrock is encountered.

For a precast concrete box culvert with a base width of 16 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 14 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 6 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. The service limit state bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$. See Appendix D – 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 94 pounds per cubic inch (pci).

7.2 Bedrock Excavation and Subgrade Preparation for Box Culvert

The bedrock surface is near the bottom of the proposed box culvert. To provide at least 1-foot of clearance between the bottom of the box culvert and bedrock, and to install the 12-inch-thick layer of granular borrow, the Contractor should be prepared to excavate and remove any bedrock that is higher than El. 468 feet. The top of bedrock subgrade shall be level and match the elevation of the surrounding soil subgrade.

The bedrock encountered in borings had RQD's ranging from 0 to 29 percent. Therefore, rock excavation can likely be completed using conventional excavation methods, but blasting may be required. The Contractor should maintain the excavation so that the box culvert is constructed in-the-dry. It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps.

The soils encountered in borings at the box subgrade elevation generally consisted of medium dense granular fills, very dense Glacial Till and concrete. Any unsuitable soils (i.e. low strength silts and clays and loose sands), and all concrete, that may be encountered at the subgrade elevation, should be excavated down to expose competent, firm material and replaced with compacted granular borrow.

7.3 Settlement

The fill unit encountered at the site is medium dense in consistency. The glacial till deposit is very dense in consistency. These coarse-grained materials are cohesionless and undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The proposed vertical alignment will remain nearly the same as the existing vertical alignment. As a result, little to no increase in vertical overburden pressure is expected. Any settlement is anticipated to be small and will occur relatively quickly.

Any loose or soft soils encountered at the subgrade elevation for the precast box culvert should be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

7.4 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, Hodgdon has a design freezing index (DFI) of approximately 2200 F-degree days. A water content of 15% was used for coarse-grained soils. These components correlate to a frost depth of 7.6 feet.

It is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 7.6 feet for frost protection. See Appendix D – Calculations for supporting calculations.

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.

7.7 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. Rock excavation will be required to provide at least 1-foot of clearance between the bottom of the box culvert and the top of bedrock.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. Any cobbles or boulders encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow – Material for Underwater Backfill.

Earthwork and excavations may result in the exposure of silt or other soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with compacted Granular Borrow – Material for Underwater Backfill.

The Contractor should be prepared to excavate and remove any bedrock exposed during the excavation for the box culvert and the granular borrow bedding layer. The top of the excavated bedrock shall be level and match the elevation of the surrounding soil subgrade. Excavation of bedrock may be done using conventional excavation methods but may require drilling and blasting techniques. Blasting should be conducted in accordance with MaineDOT Standard Specification 105.2.7. It is also recommended that the contractor conduct pre-blast and post-blast surveys, as well as blast vibration monitoring, in accordance with industry standards at the time of the blast.

Soils may become saturated and water seepage may be encountered during construction and in excavations. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater and surface water infiltration using temporary ditches, sump pumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Maduskeag Bridge in Hodgdon, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

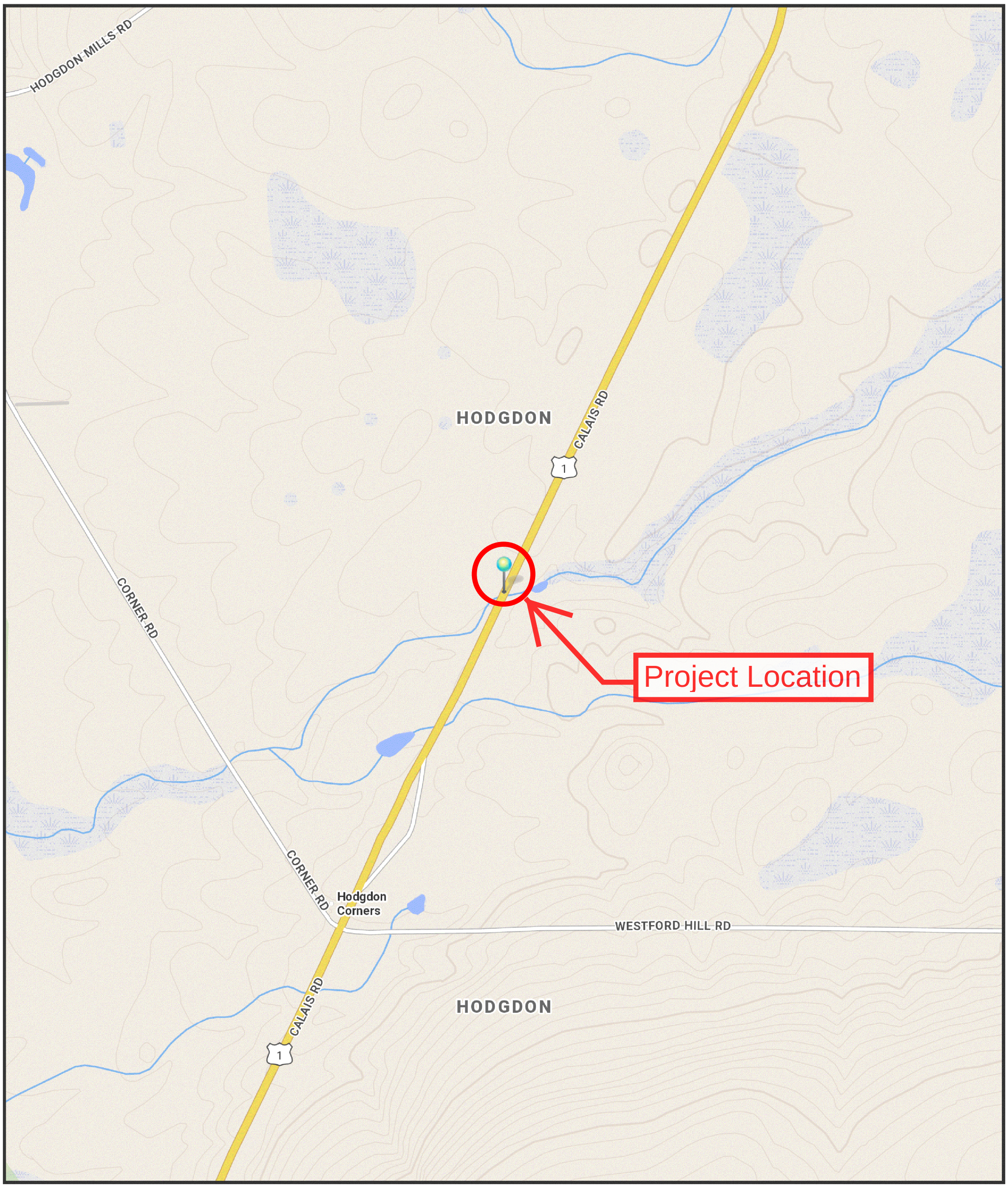
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

Username: Terry.White Division: GEOTECH Date:6/26/2020

HODGDON, MAINE



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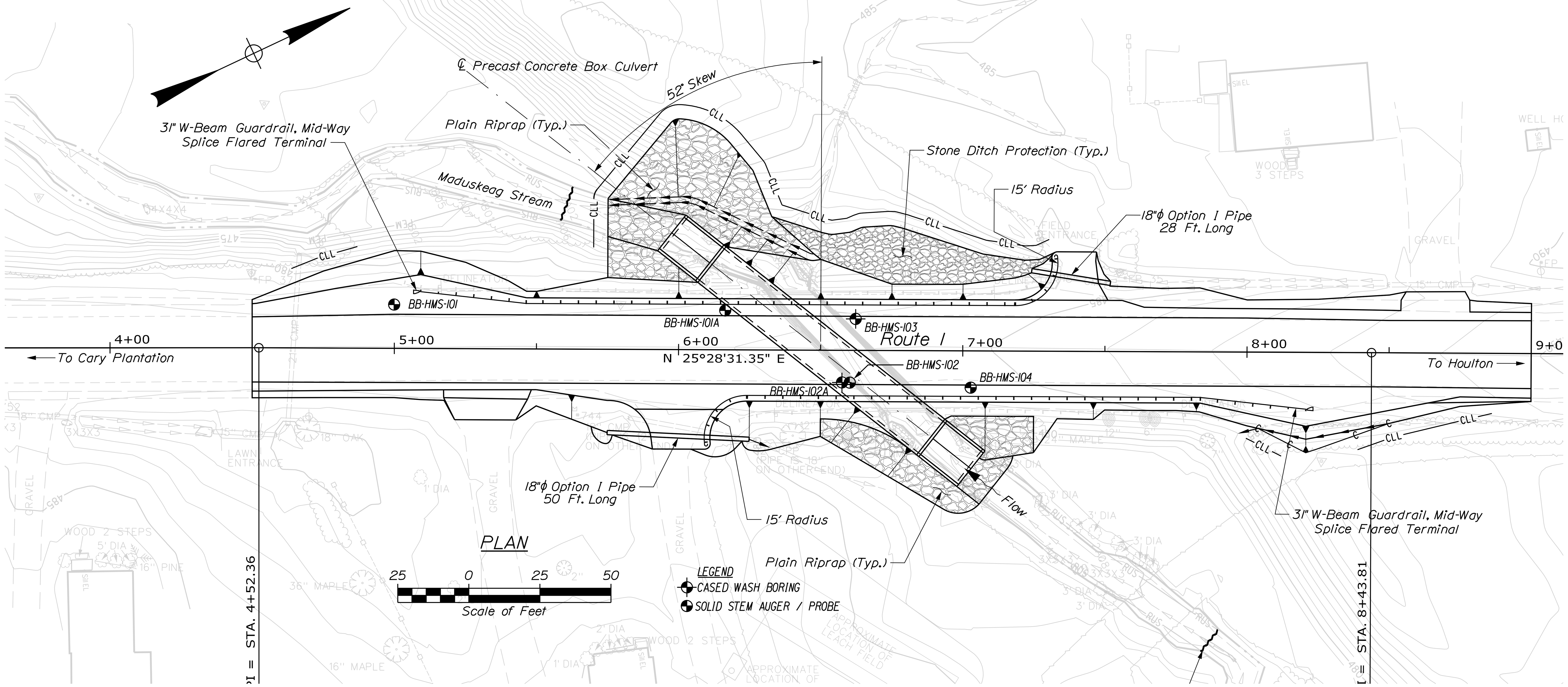
0.15 Miles

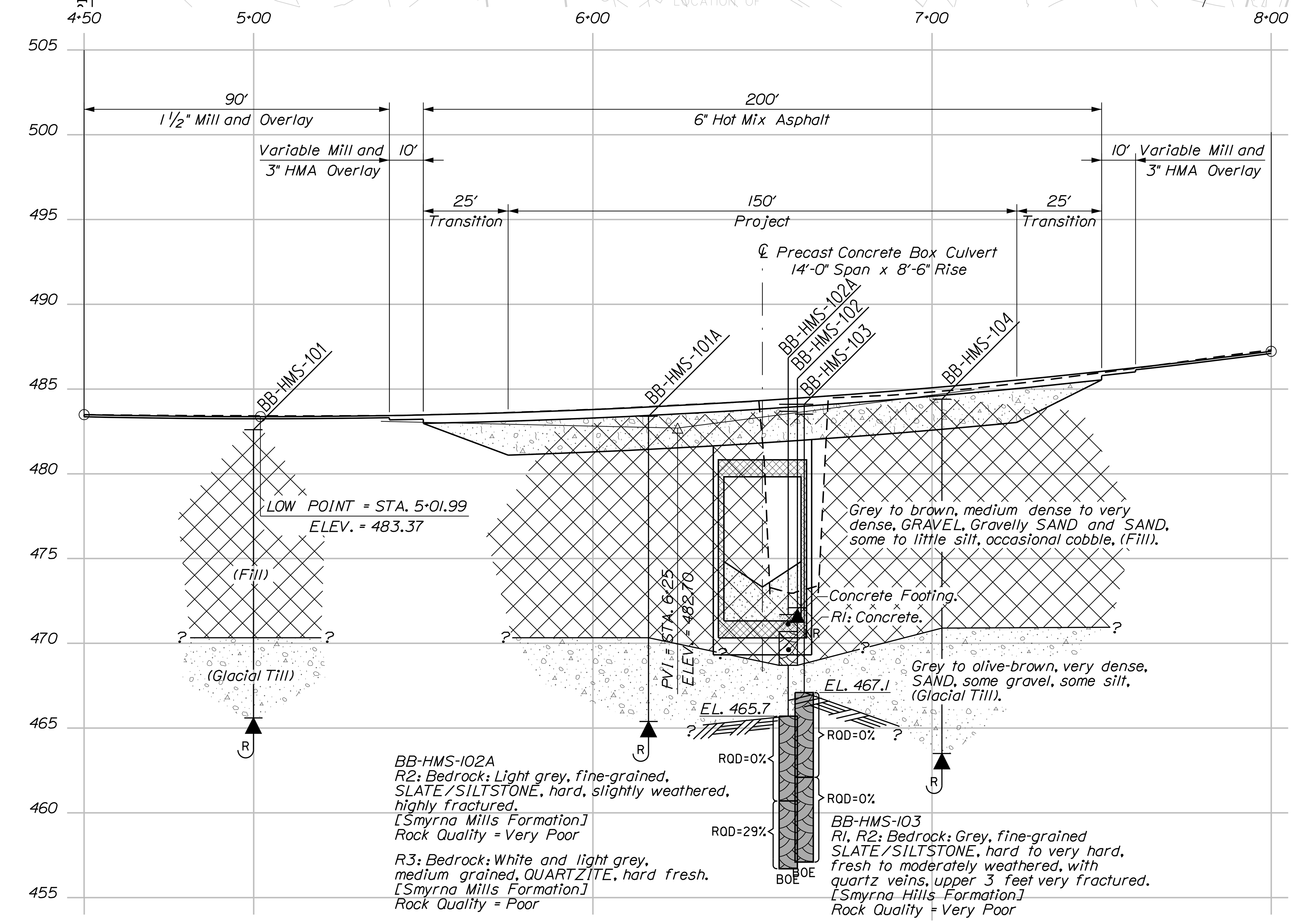
1 inch = 0.18 miles

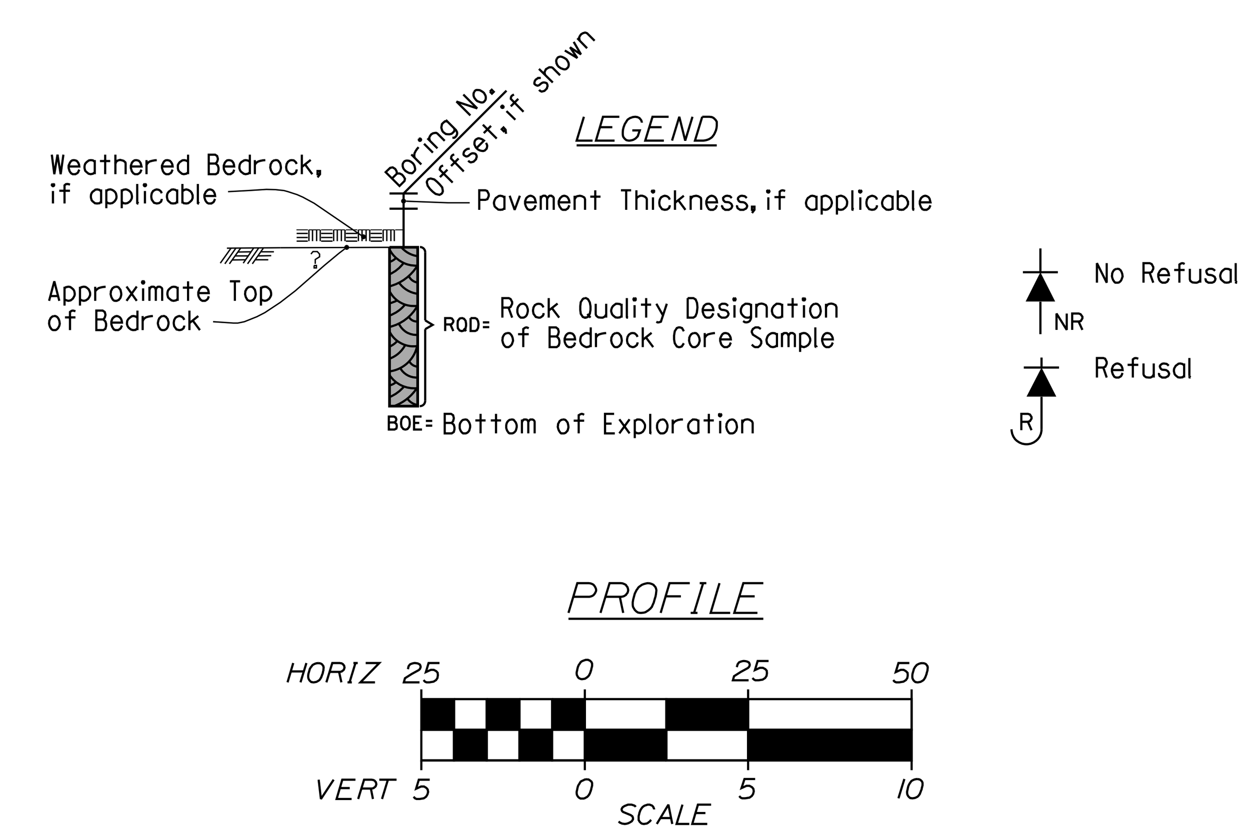
Date: 6/26/2020
Time: 10:15:15 AM

SHEET NUMBER	MADUSKEAG BRIDGE MADUSKEAG STREAM HODGDON AROOSTOOK CTY.	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1		023625.00
OF 3	LOCATION MAP	WIN BRIDGE NO. 2492 23625.00 BRIDGE PLANS

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: James.Manahan Date:6/25/2020







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

23625.00

WIN 23625.00

BRIDGE PLANS

BRIDGE NO. 2492

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	J.MANAHAN	T.WHITE	JUN 2020
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MADUSKEAG BRIDGE
MADUSKEAG STREAM
HODGDON AROOSTOOK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\MSTA\007_BORING LOG1.dgn Division: GEOTECH Username: Terry.White Date:7/9/2020

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-101
WIN: 23625.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	482.6	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/9/2018-10/9/2018	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	4+99.9, 15.5 ft Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; S = Sample off Auger Flights; B = Bucket Sample off Auger Flights; MD = Unsuccessful Split Spoon Sample Attempt; U = Thin Wall Tube Sample; MV = Unsuccessful Field Vane Shear Test Attempt; V = Field Vane Shear Test, PP= Pocket Penetrometer; MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value; T_v = Pocket Torvane Shear Strength (psf); WC = Water Content, percent; ? = Similar or Equal too; 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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-103.	
17.0							465.6		Bottom of Exploration at 17.0 feet below ground surface. REFUSAL	

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-HMS-101

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-101A
WIN: 23625.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	483.4	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/25/2018-10/25/2018	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	6+16.4, 13.9 ft Lt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; S = Sample off Auger Flights; B = Bucket Sample off Auger Flights; MD = Unsuccessful Split Spoon Sample Attempt; U = Thin Wall Tube Sample; MV = Unsuccessful Field Vane Shear Test Attempt; V = Field Vane Shear Test, PP= Pocket Penetrometer; MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value; T_v = Pocket Torvane Shear Strength (psf); WC = Water Content, percent; ? = Similar or Equal too; 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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-103.	
18.0							465.4		Bottom of Exploration at 18.0 feet below ground surface. REFUSAL	

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-HMS-101A

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-102A
WIN: 23625.00

Driller:	MaineDOT	Elevation (ft.)	484.1	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/23/2018; 08:00-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+57.6, 11.3 ft Rt.	Casing ID/OD:	NW-3"/HW-4"	Water Level*:	10.0 ft bgs.
Hammer Efficiency Factor: 0.928		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
0.4							SSA	483.7		5" HMA.	
										No soil samples retrieved. Observed soils similar to BB-HMS-102. Roller Coned ahead NW Casing, then drove HW Casing to 12.4 ft bgs., then Roller Coned ahead to 16.5 ft bgs.	
										Cobble from 9.0-9.5 ft bgs.	
12.4								471.7		Concrete Footing from 12.4-13.4 ft bgs.	
	R1	24/21	13.40 - 15.40				47			R1: Concrete. R1: Core Times (min:sec) 13.4-14.4 ft (4:00) 14.4-15.4 ft (3:15)	
							48				
15.4							99	468.7			
	1D	22.8/15	16.50 - 18.40	13/17/32/50(4.8")	49	76	45			Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).	G#336928 A-1-b, SM WC=7.7%
							26			950 blows for 0.4 ft.	
18.4	R2	60/58	18.40 - 23.40	RQD = 0%			950 NQ-2	465.7		Top of Bedrock at Elev. 465.7 ft. R2: Bedrock: Light Grey, fine-grained, SLATE/SILTSTONE, hard, very slightly weathered, highly fractured, joints steep, close, tight to open, with quartz veins. [Smyrna Mills Formation] Rock Quality = Very Poor. R2: Core Times (min:sec) 18.4-19.4 ft (5:08) 19.4-20.4 ft (4:08) 20.4-21.4 ft (3:09) 21.4-22.4 ft (4:49) 22.4-23.4 ft (7:06) 97% Recovery	
	R3	48/45	23.40 - 27.40	RQD = 29%						R3: Bedrock: White to light grey, medium-grained, QUARTZITE, hard, fresh, joints at low angles, close, tight. [Smyrna Mills Formation] Rock Quality = Poor. R3: Core Times (min:sec) 23.4-24.4 ft (5:00) 24.4-25.4 ft (6:01) 25.4-26.4 ft (6:06) 26.4-27.4 ft (7:00) 94% Recovery	
27.4								456.7		Bottom of Exploration at 27.4 feet below ground surface.	

Remarks:
10 ft of HW Casing left in bore hole.

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-HMS-102A

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-103
WIN: 23625.00

Driller:	MaineDOT	Elevation (ft.)	484.1	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/9/2018-10/10/2018	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+62.3, 11.1 ft Lt.	Casing ID/OD:	HW-4"	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.928		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
0.6							SSA	483.5		7" HMA.	
	1D	24/14	5.00 - 7.00	2/5/5/6	10	15				Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	G#336929 A-1-b, SM WC=5.4%
	2D	24/15	10.00 - 12.00	3/3/4/4	7	11	14			Grey-brown, wet, medium dense, SAND, some gravel, little silt, (Fill).	G#336930 A-1-b, SM WC=15.4%
							14				
							20				
14.0							46	470.1			
							52				
	3D	20/17	15.00 - 16.67	17/22/44/50(2")	66	102	28			Olive-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	G#336931 A-2-4, SM WC=11.5%
							169				
17.0	R1	60/59	17.00 - 22.00	RQD = 0%			NQ-2	467.1		Top of Bedrock at Elev. 467.1 ft. R1: Bedrock: Grey, fine-grained, SLATE/SILTSTONE, hard to very hard, fresh to moderately weathered, upper 3 feet very fractured with rust colored joints. Joints at low angles, close, tight to open, with quartz veins. [Smyrna Mills Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 17.0-18.0 ft (3:04) 18.0-19.0 ft (3:04) 19.0-20.0 ft (3:38) 20.0-21.0 ft (3:13) 21.0-22.0 ft (3:48) 98% Recovery	
	R2	60/60	22.00 - 27.00	RQD = 0%						R2: Bedrock: Grey, fine-grained, SLATE/SILTSTONE, very hard, fresh, joints at low angles, close, tight, with quartz veins. [Smyrna Mills Formation] Rock Quality=Very Poor R2: Core Times (min:sec) 22.0-23.0 ft (2:47) 23.0-24.0 ft (4:17) 24.0-25.0 ft (2:40) 25.0-26.0 ft (3:15) 26.0-27.0 ft (3:46) 100% Recovery	
27.0								457.1		Bottom of Exploration at 27.0 feet below ground surface.	

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-HMS-103

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-102
WIN: 23625.00

Driller:	MaineDOT	Elevation (ft.)	484.1	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/10/2018; 14:00-15:30	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	6+60.3, 11.4 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.928		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
0.4							SSA	483.7		5" HMA.	
	1D	24/12	5.00 - 7.00	2/3/5/7	8	12				Brown, damp, medium dense, GRAVEL, some sand, little silt, (Fill).	G#336926 A-1-b, GM WC=10.4%
										Cobble from 8.9-9.6 ft bgs.	
	2D	24/18	10.00 - 12.00	16/31/28/33	59	91				Brown, wet, very dense, SAND, some gravel, some silt, (Fill).	G#336927 A-2-4, SM WC=17.5%
12.0								472.1		Bottom of Exploration at 12.0 feet below ground surface. Spoon Tip broke off in Boring, moved to BB-HMS-102A.	

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-HMS-102

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

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag
Location: Hodgdon, Maine

Boring No.: BB-HMS-104
WIN: 23625.00

Drilling Contractor:	MaineDOT	Elevation (ft.)	484.4	Auger ID/OD:	5" Dia.
Operator:	Daggett/Niles	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/25/2018-10/25/2018	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	7+02.9, 13.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; S = Sample off Auger Flights; B = Bucket Sample off Auger Flights; MD = Unsuccessful Split Spoon Sample Attempt; U = Thin Wall Tube Sample; MV = Unsuccessful Field Vane Shear Test Attempt; V = Field Vane Shear Test, PP= Pocket Penetrometer; MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value; T_v = Pocket Torvane Shear Strength (psf); WC = Water Content, percent; ? = Similar or Equal too; 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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-102A; no concrete encountered.	
20.9							463.5		Bottom of Exploration at 20.9 feet below ground surface. REFUSAL	

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-HMS-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
23625.00
WIN 23625.00
BRIDGE PLANS
BRIDGE NO. 2492

	BY	DATE
PROJ. MANAGER	M. WIGHT	
DESIGN-DETAILED	J.MANAHAN	
CHECKED-REVIEWED	T.WHITE	JUL 2020
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

HODGDON
MADUSKEAG BRIDGE
MADUSKEAG STREAM
AROOSTOOK 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):

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream Location: Hodgdon, Maine	Boring No.: BB-HMS-101 WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	482.6	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/9/2018-10/9/2018	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	4+99.9, 15.5 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-103.	
5										
10										
15										
							465.6		17.0	
									Bottom of Exploration at 17.0 feet below ground surface. REFUSAL	
20										
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: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream
Location: Hodgdon, Maine

Boring No.: BB-HMS-101A
WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	483.4	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/25/2018-10/25/2018	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	6+16.4, 13.9 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-103.	
5										
10										
15										
							465.4		18.0	
									Bottom of Exploration at 18.0 feet below ground surface. REFUSAL	
20										
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.

Boring No.: BB-HMS-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream
Location: Hodgdon, Maine

Boring No.: BB-HMS-102
WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.1	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/10/2018; 14:00-15:30	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	6+60.3, 11.4 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	483.7		5" HMA. 0.4	
5	1D	24/12	5.00 - 7.00	2/3/5/7	8				Brown, damp, medium dense, GRAVEL, some sand, little silt, (Fill).	G#336926 A-1-b, GM WC=10.4%
									Cobble from 8.9-9.6 ft bgs.	
10	2D	24/18	10.00 - 12.00	16/31/28/33	59				Brown, wet, very dense, SAND, some gravel, some silt, (Fill).	G#336927 A-2-4, SM WC=17.5%
							472.1		12.0 **Bottom of Exploration at 12.0 feet below ground surface.** Spoon Tip broke off in Boring, moved to BB-HMS-102A.	
15										
20										
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.

Boring No.: BB-HMS-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream
Location: Hodgdon, Maine

Boring No.: BB-HMS-104

WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.4	**Auger ID/OD:**	5" Dia.
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/25/2018-10/25/2018	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	7+02.9, 13.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No soil samples retrieved. Observed soils similar to BB-HMS-102A; no concrete encountered.	
5										
10										
15										
20							463.5		20.9	
									Bottom of Exploration at 20.9 feet below ground surface. REFUSAL	
25										

Remarks:

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

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

Boring No.: BB-HMS-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream
Location: Hodgdon, Maine

Boring No.: BB-HMS-102A
WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/23/2018; 08:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+57.6, 11.3 ft Rt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	10.0 ft bgs.

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	483.7		5" HMA. 0.4	
									No soil samples retrieved. Observed soils similar to BB-HMS-102. Roller Coned ahead NW Casing, then drove HW Casing to 12.4 ft bgs., then Roller Coned ahead to 16.5 ft bgs.	
5										
									Cobble from 9.0-9.5 ft bgs.	
10										
							471.7		12.4	
									Concrete Footing from 12.4-13.4 ft bgs.	
	R1	24/21	13.40 - 15.40			47			R1: Concrete. R1: Core Times (min:sec) 13.4-14.4 ft (4:00) 14.4-15.4 ft (3:15)	
						48				
15						99	468.7		15.4	
	1D	22.8/15	16.50 - 18.40	13/17/32/50(4.8)	49	45			Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).	G#336928 A-1-b, SM WC=7.7%
						26				
	R2	60/58	18.40 - 23.40	RQD = 0%		a50 NQ-2	465.7		a50 blows for 0.4 ft. 18.4	
									Top of Bedrock at Elev. 465.7 ft R2: Bedrock: Light Grey, fine-grained, SLATE/SILTSTONE, hard, slightly weathered, highly fractured, joints steep, close, tight to open, with quartz veins. [Smyrna Mills Formation] Rock Quality = Very Poor. R2: Core Times (min:sec) 18.4-19.4 ft (5:08) 19.4-20.4 ft (4:08) 20.4-21.4 ft (3:09) 21.4-22.4 ft (4:49) 22.4-23.4 ft (7:06) 97% Recovery	
20										
	R3	48/45	23.40 - 27.40	RQD = 29%						
25										

Remarks:

10 ft of HW Casing left in bore hole.

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

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

Boring No.: BB-HMS-102A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream Location: Hodgdon, Maine	Boring No.: BB-HMS-102A WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/23/2018; 08:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+57.6, 11.3 ft Rt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	10.0 ft bgs.

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							456.7		R3: Bedrock: White to light grey, medium-grained, QUARTZITE, hard, fresh, joints at low angles, close, tight. [Smyrna Mills Formation] Rock Quality = Poor. R3: Core Times (min:sec) 23.4-24.4 ft (5:00) 24.4-25.4 ft (6:01) 25.4-26.4 ft (6:06) 26.4-27.4 ft (7:00) 94% Recovery 27.4	
30									**Bottom of Exploration at 27.4 feet below ground surface.**	
35										
40										
45										
50										

Remarks:

10 ft of HW Casing left in bore hole.

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: Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream
Location: Hodgdon, Maine

Boring No.: BB-HMS-103
WIN: 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/9/2018-10/10/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+62.3, 11.1 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	9.5 ft bgs.

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	483.5		7" HMA. 0.6	
5	1D	24/14	5.00 - 7.00	2/5/5/6	10				Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	G#336929 A-1-b, SM WC=13.4%
10	2D	24/15	10.00 - 12.00	3/3/4/4	7	14			Grey-brown, wet, medium dense, SAND, some gravel, little silt, (Fill).	G#336930 A-1-b, SM WC=15.4%
						14				
						20				
						46				
						52	470.1		14.0	
15	3D	20/17	15.00 - 16.67	17/22/44/50(2")	66	28			Olive-brown, wet, very dense, SAND, some silt, some gravel, (Glacial Till).	G#336931 A-2-4, SM WC=11.5%
						169	467.1		17.0	
	R1	60/59	17.00 - 22.00	RQD = 0%		NQ-2			Top of Bedrock at Elev. 467.1 ft. R1: Bedrock:Grey, fine-grained, SLATE/SILTSTONE, hard to very hard, fresh to moderately weathered, upper 3 feet very fractured with rust colored joints. Joints at low angles, close, tight to open, with quartz veins. [Smyrna Mills Formation] Rock Quality = Very Poor R1: Core Times (min:sec) 17.0-18.0 ft (3:04) 18.0-19.0 ft (3:04) 19.0-20.0 ft (3:38) 20.0-21.0 ft (3:13) 21.0-22.0 ft (3:48) 98% Recovery	
20										
	R2	60/60	22.00 - 27.00	RQD = 0%					R2: Bedrock:Grey, fine-grained, SLATE/SILTSTONE, very hard, fresh, joints at low angles, close, tight, with quartz veins.	
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:** Maduskeag Bridge #2492 carries U.S. Route 1 over Maduskeag Stream **Location:** Hodgdon, Maine	**Boring No.:** BB-HMS-103 **WIN:** 23625.00

Drilling Contractor:	MaineDOT	**Elevation (ft.)**	484.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/9/2018-10/10/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+62.3, 11.1 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	9.5 ft bgs.

Definitions: D = Spilt Spoon Sample
S = Sample off Auger Flights
B = Bucket Sample off Auger Flights
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MV = Unsuccessful Field Vane Shear Test Attempt
V = Field Vane Shear Test, PP= Pocket Penetrometer
MU = Unsuccessful Thin Wall Tube Sample 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 1 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-value = Raw Field SPT N-value
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent ≅ = Similar or Equal too
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							457.1		[Smyrna Mills Formation] Rock Quality=Very Poor R2: Core Times (min:sec) 22.0-23.0 ft (2:47) 23.0-24.0 ft (4:17) 24.0-25.0 ft (2:40) 25.0-26.0 ft (3:15) 260-27.0 ft (3:46) 100% Recovery 27.0	
									Bottom of Exploration at 27.0 feet below ground surface.	
30										
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.

Appendix B

Rock Core Photographs

MaineDOT

Maduskeag Bridge #2492 Carries US Route 1 Over Maduskeag Stream
Hodgdon, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Pentration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-HMS-102A	R2	18.4-23.4	60	58	0	0	SLATE/SILTSTONE	1
BB-HMS-102A	R3	23.4-27.4	48	45	14	29	QUARTZITE	2
BB-HMS-102A	R1	13.4-15.4	24	21	-	-	CONCRETE	3

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 3 = bottom.

MaineDOT

Maduskeag Bridge #2492 Carries US Route 1 Over Maduskeag Stream
Hodgdon, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Pentration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-HMS-103	R1	17.0-22.0	60	59	0	0	SLATE/SILTSTONE	1
BB-HMS-103	R2	22.0-27.0	60	60	0	0	SLATE/SILTSTONE	2

Notes: 1. “Box row” indicates the section of the box where the core run is contained: 1 = top, 2 = bottom.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Hodgdon   Work Number: 23625.00

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-HMS-102, 1D	6+60.3	11.4 Rt.	5.0-7.0	336926	1	10.4			GM	A-1-b	I
BB-HMS-102, 2D	6+60.3	11.4 Rt.	10.0-12.0	336927	1	17.5			SM	A-2-4	II
BB-HMS-102A, 1D	6+57.6	11.3 Rt.	16.5-18.4	336928	1	7.7			SM	A-1-b	II
BB-HMS-103, 1D	6+62.3	11.1 Lt.	5.0-7.0	336929	1	13.4			SM	A-1-b	II
BB-HMS-103, 2D	6+62.3	11.1 Lt.	10.0-12.0	336930	1	15.4			SM	A-1-b	II
BB-HMS-103, 3D	6+62.3	11.1 Lt.	15.0-16.7	336931	1	11.5			SM	A-2-4	II

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.

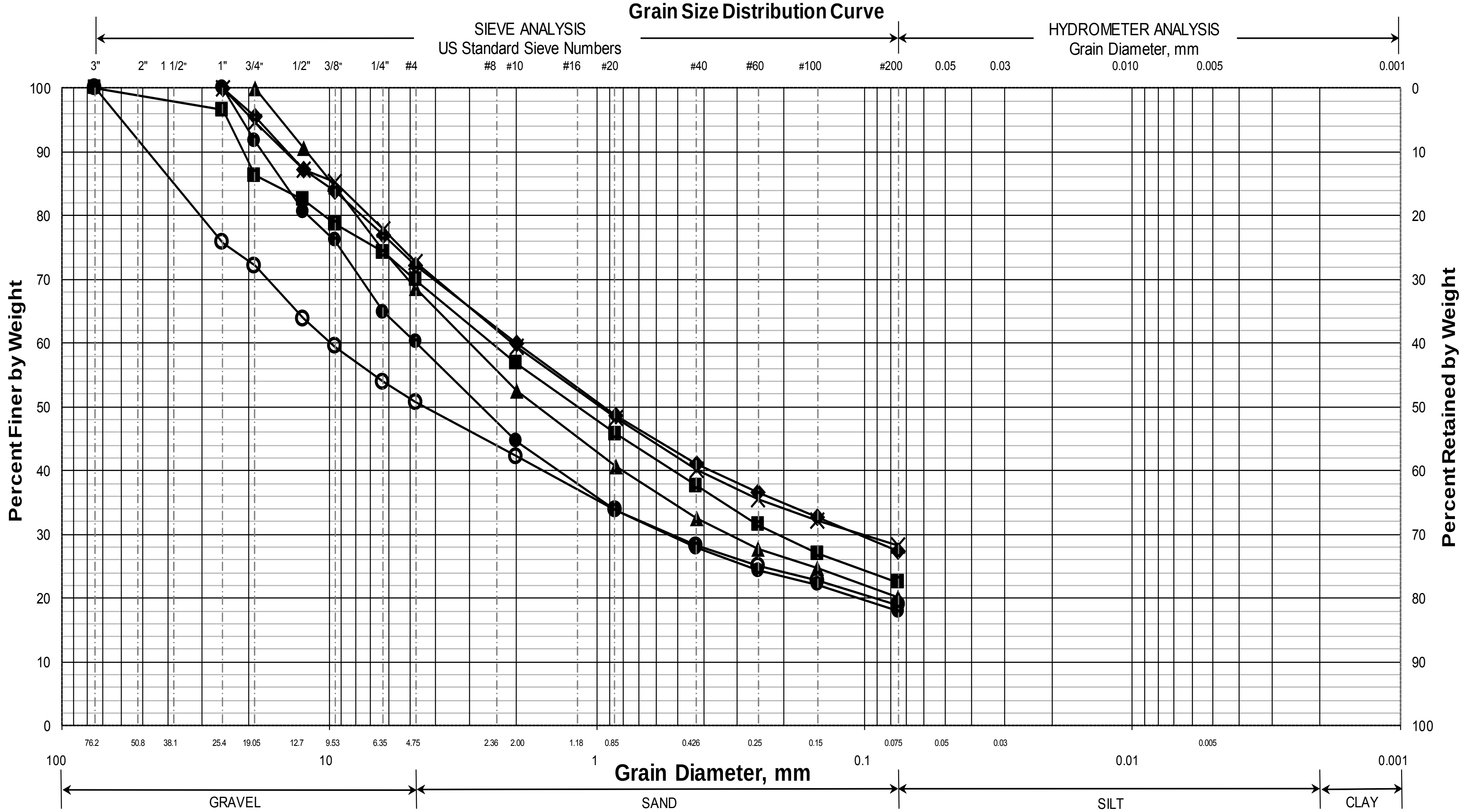
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-HMS-102/1D	6+60.3	11.4 RT	5.0-7.0	GRAVEL, some sand, little silt.	10.4			
◆	BB-HMS-102/2D	6+60.3	11.4 RT	10.0-12.0	SAND, some gravel, some silt.	17.5			
■	BB-HMS-102A/1D	6+57.6	11.3 RT	16.5-18.4	SAND, some gravel, some silt.	7.7			
●	BB-HMS-103/1D	6+62.3	11.1 LT	5.0-7.0	Gravelly SAND, little silt.	13.4			
▲	BB-HMS-103/2D	6+62.3	11.1 LT	10.0-12.0	SAND, some gravel, little silt.	15.4			
×	BB-HMS-103/3D	6+62.3	11.1 LT	15.0-16.7	SAND, some silt, some gravel.	11.5			

WIN	
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Town	
Hodgdon	
Reported by/Date	
WHITE, TERRY A	2/6/2019

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

[a] Unloading cycle.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

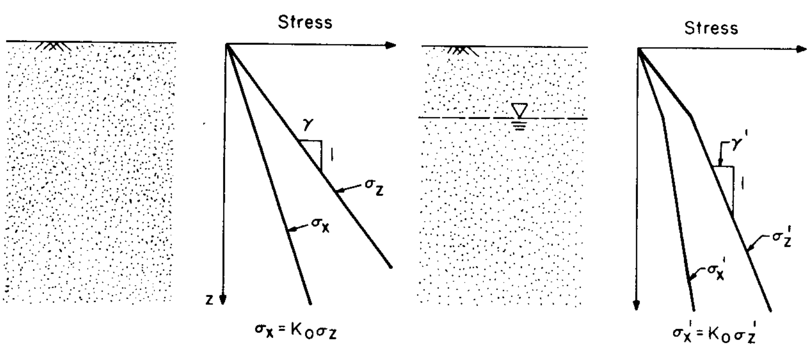


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

$\beta =$ 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:

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

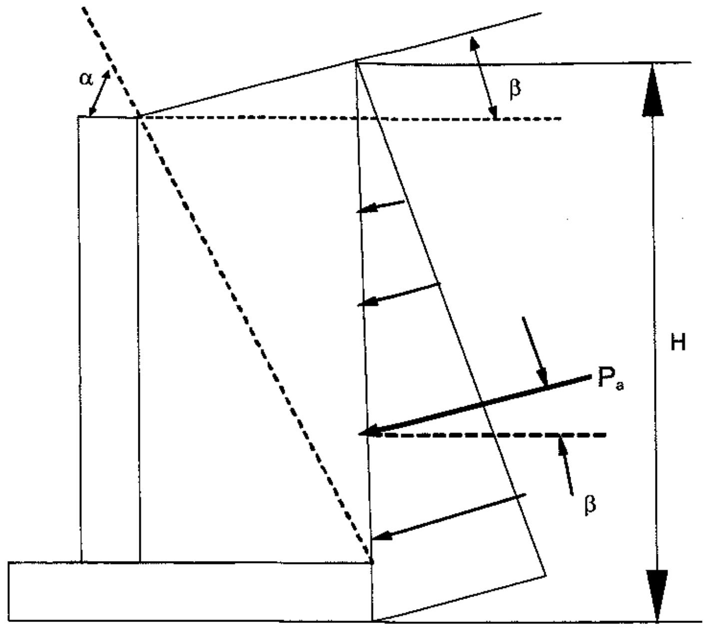


Figure 3-3 Rankine Theory

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:

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

where:

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

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

Bearing Resistance

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

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

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. The one foot thick layer of proposed Granular Borrow bedding material modeled as subgrade.
3. The proposed bearing elevation is approximately 470 feet.
4. Proposed finish roadway grade elevation is approximately 483.3 feet at the low point.
5. Proposed precast concrete box base is 16 feet wide.
6. 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 Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2*, *May 1983*, *Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Coarse to medium sand with little gravel (SW, SP)	Medium dense to dense	6-10	6

Recommend 6 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Foundation Width, Depth, and Water Surface

$$B := 16ft$$

$$D_f := 1.0 \cdot ft$$

$$D_w := 0 \cdot ft$$

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

Total unit weight of the soil above the base slab/soil envelope

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

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

Foundation soils:

Foundation soils based on BB-HMS-103 3D

$\gamma 1_d := 125 \cdot pcf$

$w_{sat} := 11.5\%$.

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

$\gamma 1_{sat} = 139.4 \cdot pcf$

$\phi := 36 \cdot deg$ Assumed friction angle for coarse-grained Glacial Till

$c := 0$

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

$L := 100 \cdot ft$

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

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

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

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

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

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

$C_{wq} := .5$ $\quad C_{w\gamma} := 0.5$

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

LRFD Table 10.6.3.1.2a-4 $\quad \frac{D_f}{B} = 0.06$

Therefore : $\quad d_q := 1.0$

Terms

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

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

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

$N_{cm} = 56.6$ $\quad N_{\gamma m} = 52.7$ $\quad N_{qm} = 42.2$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c \cdot N_{cm} + \gamma_{above\_sat} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$q_n = 32.2 \cdot ksf$

Factored Bearing Resistance

$\phi_b := 0.45$

$q_r := q_n \cdot \phi_b$

$\boxed{q_r = 14.5 \cdot ksf}$

Recommend a factored bearing resistance of 14 ksf for box bottom slabs 16 ft or greater on compacted granular borrow. Provide at least 1'-0" of clearance between bottom of the box culvert and top of bedrock.

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/ft^3	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Table C10.6.2.6.1-1—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

10.6.2.6.2—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 $0.3 f'_c$.

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

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

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

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

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

10.5.5.2.3—Driven Piles

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

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

C10.5.5.2.3

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

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

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:

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

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.

AASHTO LRFD 8th edition, 2017

where:

B = footing width (ft)
L = footing length (ft)
H = unfactored horizontal load (kips)
V = unfactored vertical load (kips)
θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

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

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

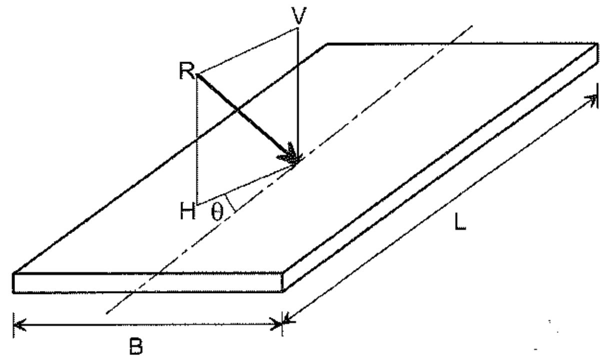


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

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

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

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

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

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

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, ϕ_f. Information beyond the range indicated is not available at this time.

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.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

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 base slab is approximately 471 feet.
2. Proposed finished roadway grade is approximately 469.5.
3. Proposed precast concrete box is 14 feet wide and approximately 134 feet long.
4. The subsurface conditions present at the proposed bearing elevation is glacial till.
5. The bottom of the box culvert will be submerged for the structure's design life.

Published values of subgrade modulus

Published values of subgrade modulus in medium sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction 236 to 472 pci
Subgrade of dense sand, average of upper and lower limit: k_s = 354 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Saturated coarse grained very dense sand K_{V1}, 260 pci / 2 = 130 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 dense sand, 478 - 553 pci: $k_{0.3}$ (k_1) = 515 pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:
Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand
Submerged dense sand: k_{s1} = 347 pci

Adjust Published values for dimensions of base slab

Select a subgrade modulus of 260 pci for saturated dense sand based on GEC No. 6. and divided by 2 per Note 2 (see references attached).

Value of k_{s1} = 130 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B - 16 ft, Length L = 100 ft).

Square to rectangle base adjustment:

$$k_{s1} := 130\text{pci} \quad B := 16\text{ft} \quad L := 100\text{ft}$$

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

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

$$k = 93.6 \cdot \text{pci}$$

Recommend a subgrade modulus of 94 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 equae 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 bearinged. 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 equahere 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 e if a load test was done. It might also be estimated from a triaxial ltimate" or at the maximum pressure from the stress-strain plot. $\epsilon_{\max}$ compute

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

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

TABLE 9-1
Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

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

Soil	k_s, kN/m³	ks, lb/in^3
Loose sand	4800–16 000	18 - 59
Medium dense sand	9600–80 000	35 - 295
Dense sand	64 000–128 000	236 - 472
Clayey medium dense sand	32 000–80 000	118 - 295
Silty medium dense sand	24 000–48 000	88 - 177
Clayey soil:		
$q_a \le 200$ 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

354 pci

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

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

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

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

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

For **center:**

$$H/B' = 5B/(B/2) = 10 \text{ (taking } H = 5B \text{ as recommended in Chap. 5)}$$
$$L/B = 1.83/1.22 = 1.5$$

From these we may write

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

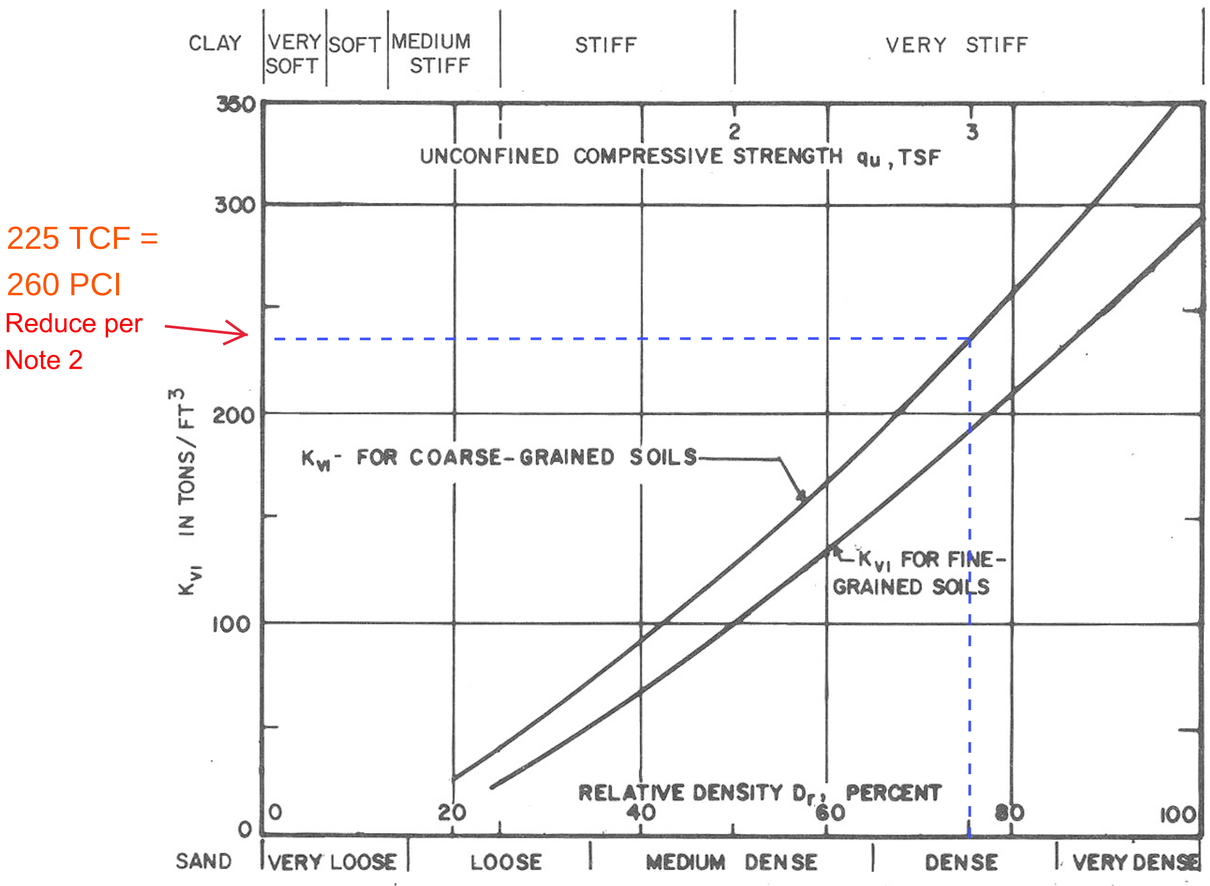
using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.
At $D/B = 0.61/1.22 = 0.5$, we obtain $I_F = 0.80$ from Fig. 5-7 (or when using FFACTOR for the I_s factors). Substitution into Eq. (9-7) with $B' = 1.22/2 = 0.61$, and $m = 4$ yields

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

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

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

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



DEFINITIONS

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

D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{VI} = MODULUS OF VERTICAL SUBGRADE REACTION

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

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

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

FOR $B \geq 40$ FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

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

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

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

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

 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

Terzaghi, Geotechnique, Vol. 5 No. 4

the bending moments in piles which are acted upon by horizontal forces above the ground surface (Cummings, 1937) and of those in core-walls of earth- and rock-fill dams (Löfquist, 1951).

Attempts have also been made to apply the theories to the solution of bulkhead problems (Rifaat, 1935). Baumann (1935) used them for estimating the stresses in an anchored bulkhead which had failed. Quite recently Blum (1951) proposed a procedure for the design of anchored bulkheads by means of the theory of horizontal subgrade reaction. All these investigations and design procedures were based on the tacit assumption that K'_0 in equation (15) is identical with the coefficient of active earth pressure K_a. The error due to this assumption may be quite important.

EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION

General procedure

The numerical values of the coefficients of subgrade reaction k_s and k_h required for the solution of engineering problems can either be estimated on the basis of published observational data or else they can be derived from the results of field tests to be performed on the subgrade of the proposed structure. For practical purposes, rough estimates of these values fully serve their purpose.

Vertical subgrade reaction

As a basis for estimating the coefficient of subgrade reaction k_s for beams and slabs, the value $\bar{k}_{s1}$ for a square plate with a width of 1 ft has been selected, because this value can, if necessary, be determined by averaging the results of several loading tests in the field, at the site of the structure.

If the subgrade consists of cohesionless or slightly cohesive sand, k_s can be estimated on the basis of the empirical values of $\bar{k}_{s1}$ given in Table 1. The density-category of the sand can be ascertained by means of a standard penetration test or other convenient means. The greatest error on the unsafe side results from using the proposed 130 value in the case of medium sand if its real value is equal to the lower limiting value of 60 tons/cu. ft.

Table 1.

Values of $\bar{k}_{s1}$ in tons/cu. ft for square plates, 1 ft × 1 ft, or beams 1 ft wide, resting on sand

Relative density of sand	Loose	Medium	Dense
Dry or moist sand, limiting values for $\bar{k}_{s1}$	20–60	60–300	300–1,000
Dry or moist sand, proposed values	40	130	500
Dense sand — Submerged sand, proposed values	25	80	$\boxed{300}$

In order to investigate the influence of such an error on the results of the computation of the bending moments in a beam, the maximum bending moment M_{max} in the beam shown in Fig. 1 was computed on the basis of both the assumed and the real value of $\bar{k}_{s1}$ for the supporting sand. The value of M_{max} for this beam is determined by equation (4). It was found that the moment computed by means of the proposed value exceeds the actual bending moment by not more than about 5%.

Once the value $\bar{k}_{s1}$ has been selected, the value of k_s to be used in the solution of a given

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

300 ton/ft^3 x 1.157 = 347 pci

EVAL

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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).

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

Method 1:

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

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: **Hodgdon, Maine**
DFI = 2200 degree-days.
Case 1 - coarse grained granular fill soils W=15% (assumed).

For DFI = 2200

at w=20% $d_1 := 82.6\text{in}$

at w=10% $d_2 := 100\text{in}$

Depth of Frost Penetration

$d := \dfrac{d_2 + d_1}{2}$ $d = 91.3 \cdot \text{in}$ $d = 7.6 \cdot \text{ft}$

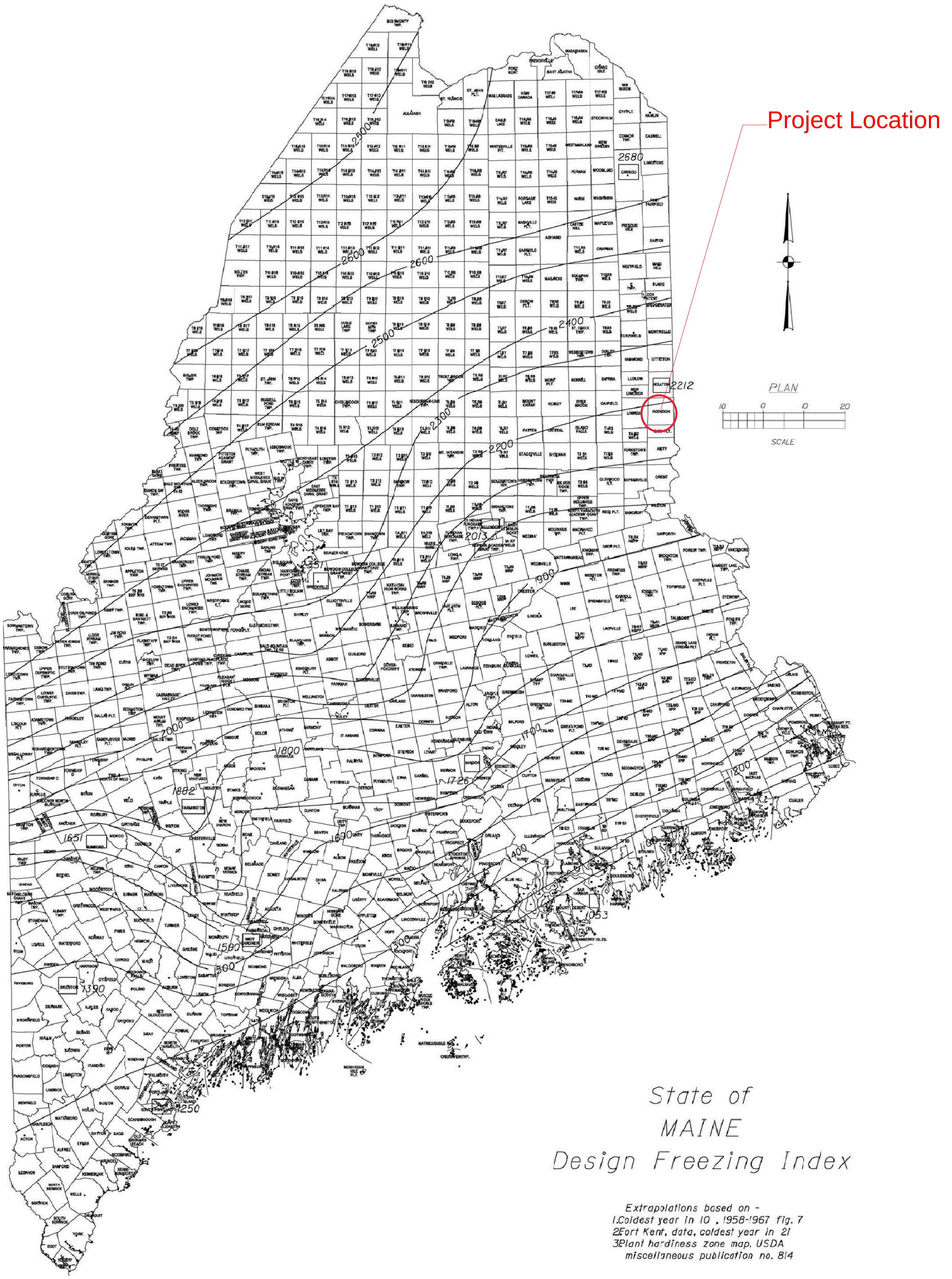

Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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