
Addendum #1

To: File
cc: TEDOCS
From: Andrew Van Buskirk
Date: June 8, 2019
Re: Soils Report No. 2019-03 Geotechnical
Design Report for the Replacement of
Goose River Bridge over Goose River
Belfast, Maine
WIN: 21874.00

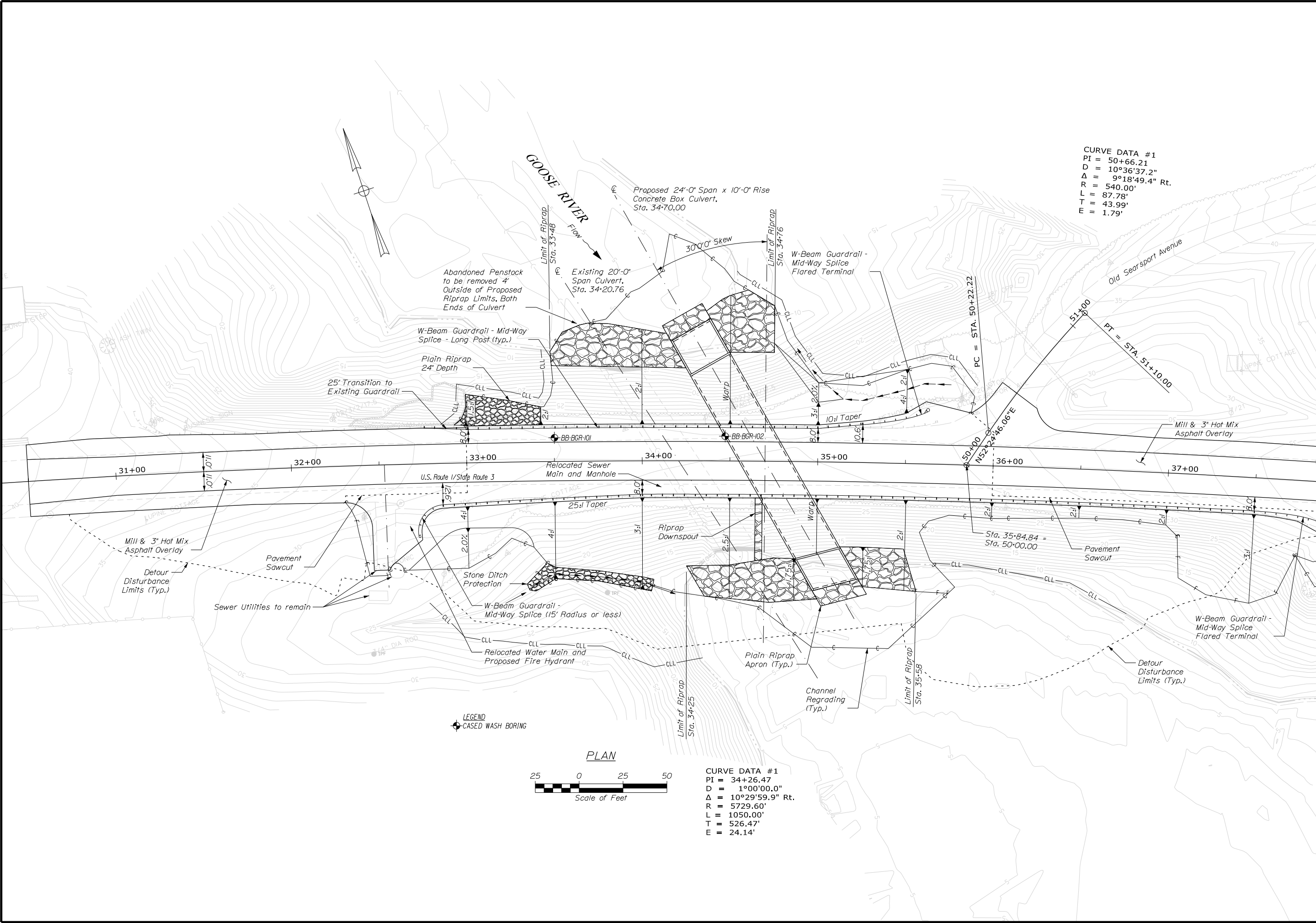
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The following changes are made to the Geotechnical Design Report for the Replacement of Goose River Bridge over Goose River, Belfast, Maine, Soils Report No. 2019-03:

1. Throughout the document, remove references to 2 feet of special fill inside the proposed concrete box culvert. The references to the 2 feet of special fill are found on pages 1 and 5.
2. The invert elevation of the proposed box culvert has been raised to El. 4.0 (inlet) and El. 3.5 (outlet). Throughout the document, revise the bottom elevation of the proposed culvert bedding layer to El. 2.0 (inlet) and El. 1.5 (outlet). References to the bottom elevation of the culvert bedding material are found on pages 6 and 8.
3. Throughout the document, revise the proposed box culvert rise from 12 feet to 10 feet. References to the culvert rise are found on pages 1 and 5.
4. The box culvert outlet changes from being skewed to being normal to the culvert. Reference to the skew of the culvert outlet is found on page 5.
5. Replace Sheet 2 – Boring Location Plan with the attached Sheet 2 – Boring Location Plan, which has been updated with the correct culvert outlet orientation.
6. Replace Sheet 3 – Interpretive Subsurface Profile with the attached Sheet 3 – Interpretive Subsurface Profile, which has been updated with the current proposed culvert invert elevation.

Attachments:

Sheet 2 – Boring Location Plan
Sheet 3 – Interpretive Subsurface Profile



CURVE DATA #1
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 Δ = 9°18'49.4" Rt.
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 E = 1.79'

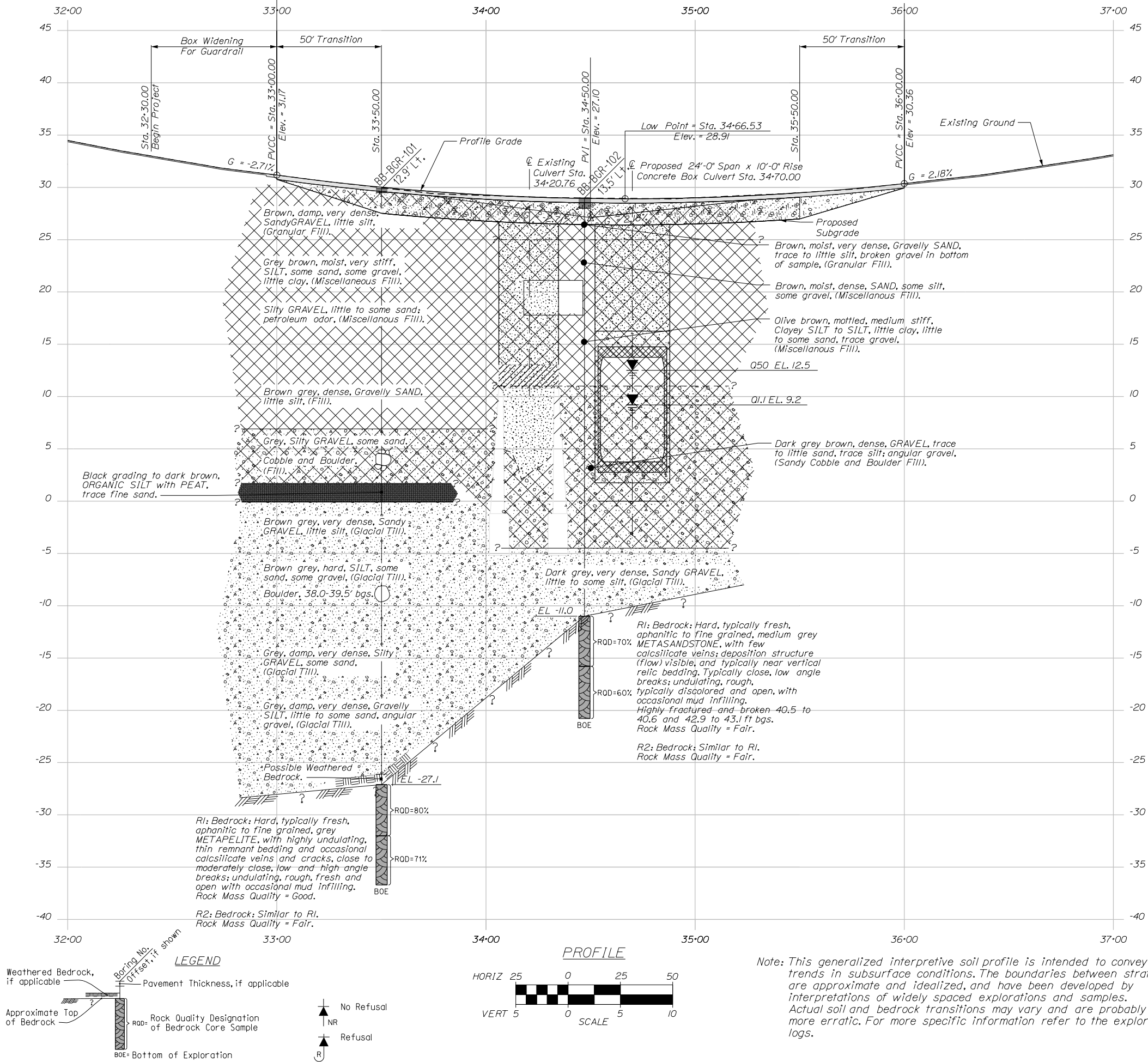
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 E = 24.14'

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
STP-2187(400)		BRIDGE NO. 2319	
WIN		21874.00	
BRIDGE PLANS			

PROJ. MANAGER	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
CHECKED-REVIEWED		MAR 2018	T. WHITE		
DESIGN-DETAILED					
DESIGNS-DETAILED					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

GOOSE RIVER BRIDGE	WALDO COUNTY	BORING LOCATION PLAN
GOOSE RIVER		
BELFAST		

SHEET NUMBER	2
OF 4	



STATE OF MAINE DEPARTMENT OF TRANSPORTATION		STP-2187(400)		BRIDGE NO. 2319		WIN 21874.00		BRIDGE PLANS	
GOOSE RIVER BRIDGE GOOSE RIVER BELFAST		WALDO COUNTY		INTERPRETIVE SUBSURFACE PROFILE		SHEET NUMBER		3 OF 4	
PROJ. MANAGER	BY	DATE	SIGNATURE	P.E. NUMBER	DATE				
CHECKED-REVIEWED		JUL 2018	T. WHITE						
DESIGN DETAILING									
REVISIONS 1									
REVISIONS 2									
REVISIONS 3									
REVISIONS 4									
FIELD CHANGES									

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

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**GOOSE RIVER BRIDGE
U.S. ROUTE 1 AND STATE ROUTE 3 OVER GOOSE RIVER
BELFAST, MAINE**

Prepared by:

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Reviewed by:

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Waldo County
WIN 21874.00

Soils Report 2019-03
Bridge No. 2319

Fed. No. STP-2187(400)
February 7, 2019

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Sheet 2 – Boring Location Plan
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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 Goose River Bridge, which carries U.S. Route 1 and State Route 3 over Goose River in Belfast, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation design recommendations, and geotechnical parameters for design of the new bridge structure.

The existing bridge was constructed in 1921 and widened in 1944. It is a 25-foot buried concrete slab that is simply supported on granite faced cast-in-place gravity abutments on the original portion of the bridge, and cast-in-place concrete gravity abutments on the widened portion. There is no bottom slab in the structure; the original and widened portions of the abutments are founded on native soils. According to the 2014 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge is rated 4 (poor condition) because of large spalls, heavy scaling, and wide cracks in the concrete slab and abutments. The 2016 Underwater Inspection Report indicates there is no evidence of scour. The structure has a FHWA Sufficiency Rating of 56.9.

The proposed replacement structure will be a 24-foot span by 12-foot rise precast concrete box culvert on a 30-degree skew. The concrete box culvert will have 1-foot tall precast headwalls and toe walls extending one foot below calculated scour depth. The upstream and downstream ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) sideslopes. The box culvert will be embedded approximately 3 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill, bearing on compacted native soils.

The new box culvert will be located east of the existing bridge. No changes are proposed to the horizontal and vertical alignments. Traffic will be maintained on a two-lane temporary detour bridge located on the downstream side. The bridge replacement project will last one construction season.

2.0 GEOLOGIC SETTING

Goose River Bridge carries U.S. Route 1 and State Route 3 over the Goose River, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Searsport Quadrangle Open-File No. 14-23 (2014), indicates the surficial soils at the bridge site consist of the Presumpscot Formation overlying sand and gravel marine fans and gravely esker deposits. The Presumpscot Formation consists of clay, silts and sands that washed out of the Lake Wisconsin glacier and accumulated on the sea floor when the relative sea level was higher than present. This sequence of glaciomarine clay and silt, marine fans and eskers has nearby contacts to glacial till, which is a heterogeneous mixture of sand, silt, clay and stones.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the project site as the Penobscot Formation consisting of metamorphosed shaly sediments (slates, schists, quartzites).

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings. Boring BB-BGR-101 was drilled west of the existing bridge and boring BB-BGR-102 was drilled east of the bridge. The boring locations are shown on Sheet 2 – Boring Location Plan.

Test borings BB-BGR-101 and BB-BGR-102 were drilled between January 9 and 19, 2018 by New England Boring Contractors (NEBC) of Hermon, Maine. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

The borings were performed using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The NEBC 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 July 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.869 for both borings. The hammer efficiency factor (0.869) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

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

4.0 LABORATORY TESTING

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

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered generally consisted of Miscellaneous Fill, Cobble and Boulder Fill, Organic Silt and Peat, and Glacial Till underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following

paragraphs summarize the subsurface conditions encountered:

5.1 Miscellaneous Fill

A layer of Miscellaneous Fill was encountered in both borings. The Fill materials ranged from granular and silty fill to a cobble and boulder fill. The thickness of the Fill unit encountered was approximately 28 to 33 feet at the boring locations. The fill layers encountered generally consisted of:

- Grey brown and olive brown, moist, silt, some sand, trace to little clay, trace to some gravel;
- Brown and brown grey, damp to moist, gravelly sand to sandy gravel, trace to little silt;
- Brown, moist, sand, some silt, some gravel;
- Olive brown, mottled, clayey silt, little fine sand;
- Olive brown, mottled, silt, some sand, little clay, trace gravel.
- Dark grey brown, gravel, trace to little sand, trace silt.
- Silty gravel, little to some sand;
- Cobbles;
- Boulders.

SPT N-values in the fill layers ranged from 6 to 119 blows per foot (bpf), indicating the layer is medium stiff to very dense in consistency. Grain size analyses of the Fill material resulted in the soil being classified as A-1-a, A-2-4, and A-4 under the AASHTO Soil Classification System and SM and ML under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 8 to 17 percent.

5.2 Organic Silt and Peat

A layer of Organic Silt and Peat was encountered in boring BB-BGR-101 beneath the Fill unit. The thickness of the deposit encountered was approximately 1.8 feet at the boring location.

A grain size analysis conducted on a sample of the Organic Silt resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and CL under the USCS. The moisture content of the sample tested was approximately 53 percent.

5.3 Glacial Till

Glacial Till was encountered in the borings. The thickness of the deposit encountered ranged from approximately 7 to 26 feet at the boring locations. The Glacial Till generally consisted of:

- Brown to dark grey, sandy gravel, little to some silt;
- Brown grey, silt, some sand, some gravel;
- Grey, damp, silty gravel, some sand;
- Grey, damp, gravelly silt, little to some sand.

SPT N-values in the Glacial Till deposit ranged from 43 to 178 bpf indicating the soil is dense to

very dense in consistency. Grain size analyses conducted on samples of the Glacial Till deposit resulted in the samples being classified as A-1-b and A-4 under the AASHTO Soil Classification System and GM and SM under the USCS. The natural water content of the samples ranged from approximately 8 to 9 percent.

5.4 Bedrock

Bedrock was encountered and cored in borings BB-BGR-101 and BB-BGR-102. 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-BGR-101	33+50.1	13.5 L	57.0	-27.1	80, 71
BB-BGR-102	34+47.2	14.1 L	40.0	-11.0	70, 60

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

The bedrock at the site is identified as medium grey, aphanitic to fine grained, Metapelite and Metasandstone, hard, typically fresh, joints/fractures at low and high angle, close to moderately close, open and with mud infilling, and with calcsilicate veins and cracks. Detailed bedrock descriptions and the RQD core run are provided on the boring logs on Sheet 4 – Boring Logs and in Appendix A – Boring Logs.

5.5 Groundwater

Groundwater was observed in BB-BGR-102 at approximately 21.8 feet below the ground surface (bgs) and Elev. 7.2. Water was introduced into the boreholes during drilling operations. Therefore, water levels may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The draft Preliminary Design Report (PDR) dated April 20, 2018 investigated two structure replacement and two rehabilitation options. The alternatives were:

- Structure replacement with a 102-foot pile supported integral abutment bridge;
- Structure replacement with a 24-foot span and 12-foot rise precast concrete box culvert;
- Short term rehabilitation of the structure;
- Rehabilitation of the substructure and replacement of the slab.

After consideration of all of the alternatives, the precast concrete box was selected. This option provides lowest overall construction and maintenance costs, minimizes adjustments to the horizontal and vertical alignments, simplifies the diversion of water, and allows the deeper parts of the existing foundation to be left in place. The bottom slab of the precast concrete box culvert will be embedded into the streambed to accommodate 2 feet of engineered streambed material creating a natural streambed. Plain riprap aprons will be constructed at the culvert inlet and outlet. The box culvert will be constructed to the east of the existing bridge with a re-aligned channel.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 24-foot span by 12-foot rise 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 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.

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 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 (AASHTO) Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2017.

The loading specified for the design of the box culvert shall be Modified HL-93 Strength I, which increases the HS-20 design truck wheel loads 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 design earth loads from the soil envelope. The backfill properties are as follows: $\phi = 32^\circ$, $\gamma = 125$ pcf. For vertical earth pressure, the maximum load factor for at rest earth pressure from LRFD Table 3.4.1-2 shall apply and wheel loads should be distributed through earth fills according to the provisions of LRFD Article 3.6.1.2.6.

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 by 1 foot concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert's inlet walls will be slope-tapered to match the 2H:1V sideslopes of the roadway embankment. The outlet walls will be slope-tapered and skewed at 60°. The left

and right inlet and outlet walls will share the same base slab. The sloped 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 force effects resulting from creep, temperature, and shrinkage deformations of the concrete box culvert. 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. Wingwall sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a , of 0.31 assuming a level backslope. Wingwall sections that are independent of the box culvert and have a backslope of 2H:1V should be designed using the Rankine active earth pressure coefficient of 0.46. See Appendix C – Calculations for supporting documentation.

7.1.3 Precast Concrete 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 of approximately -1.0 feet. The subgrade soils at this elevation are expected to be dense to very dense in consistency. These soils are characterized as having adequate bearing resistance.

For a precast concrete box culvert with a base width of 26 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 40 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 10 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. In no instance shall the bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$. See Appendix C – Calculations for supporting calculations.

7.1.5 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 255 pounds per cubic inch (pci). See Appendix C – Calculations for references.

7.2 Settlement

The Glacial Till unit encountered at the bearing elevation is dense in consistency. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. Little to no increase in vertical overburden pressure is expected. As a result, any settlement is anticipated to be small and will occur relatively quickly.

Any loose or soft fill material encountered at the subgrade should be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill. The exposed subgrade shall then be thoroughly compacted. With these provisions, post-construction settlement of the replacement structure is anticipated to be minimal.

7.3 Subgrade Preparation

The box culvert shall be placed on a 1-foot-thick layer of compacted Granular Borrow – Material for Underwater Backfill. The compacted Granular Borrow layer shall be placed on a subgrade consisting of compacted, undisturbed soil. The soils encountered during the subsurface investigation at the elevation of the bedding layer generally consisted of dense to very dense Glacial Till. Unsuitable soils (i.e. loose or soft soils), if encountered at the subgrade elevation, and loose or soft zones observed during compaction, should be excavated to expose competent, firm material and replaced with compacted Granular Borrow. A thin layer of Organic Silt and Peat encountered in BB-BGR-101 near the bearing elevation shall be excavated and replaced with Granular Borrow if encountered. Any cobbles or boulders encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow.

7.4 Frost Protection

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Belfast has a design freezing index (DFI) of approximately 1450 F-degree days. A water content of 10% was used for coarse-grained soils. These components correlate to a frost depth of 6.7 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Belfast, Maine has an air DFI from the Modberg database of approximately 1188 F-degree days. A water content of 10% was used. These components correlate to a frost depth of approximately 4.1 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on soil be designed with an embedment of approximately 6.7 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.5 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and inlet/outlet wall 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.

The slopes shall be armored with a 3-foot-thick layer of riprap conforming to MaineDOT Standard Specification Section 703.26 Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot-thick 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 beneath the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 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 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.

The proposed box culvert will be bedded on a 1-foot-thick layer of Material for Underwater Backfill conforming to Standard Specification 703.19, with a bottom of Excavation elevation of approximately -1.0 feet. Based on the soils encountered in the borings, dense to very dense, coarse grained soils will be present at this elevation.

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.

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 Goose River Bridge in Belfast, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

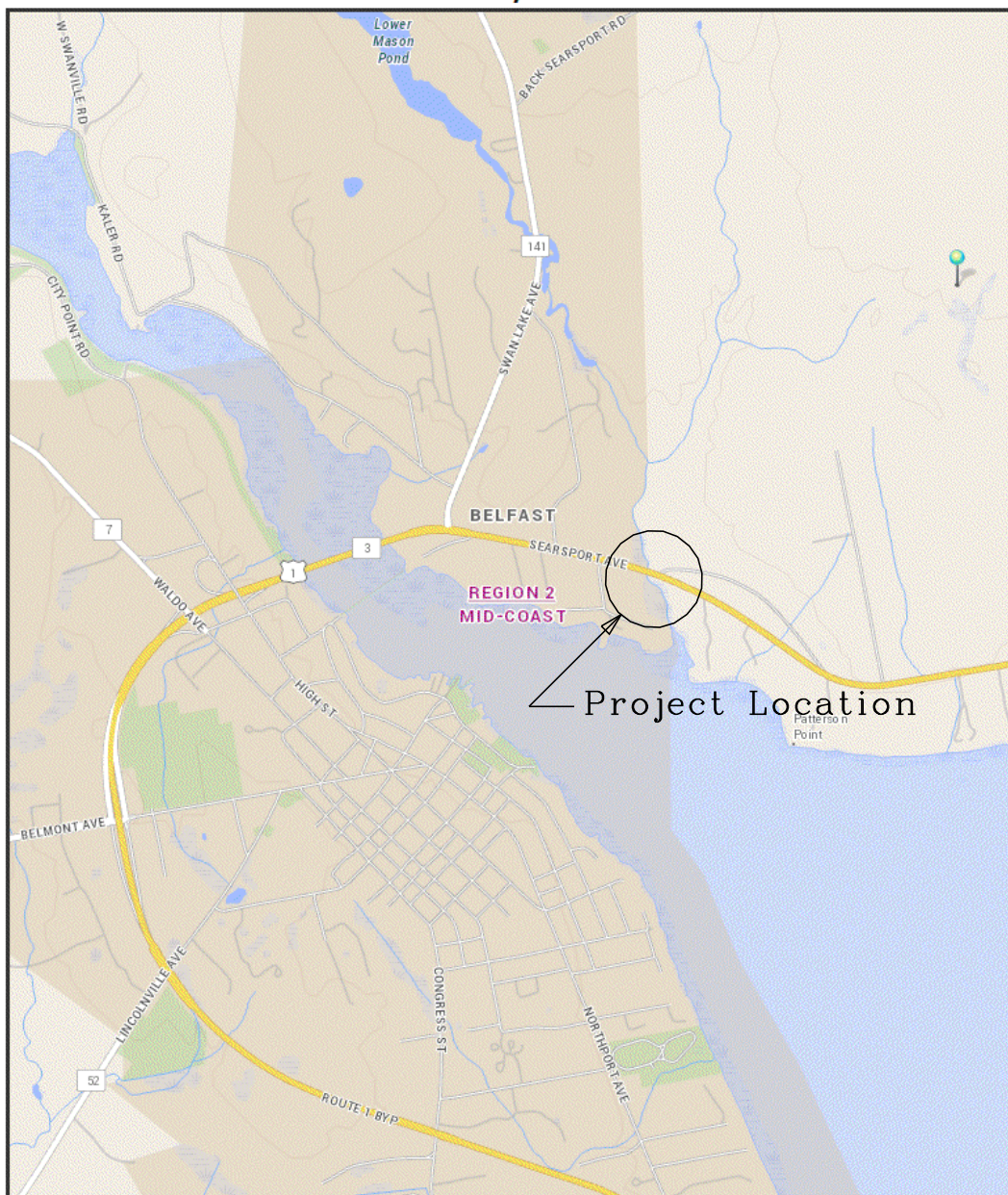
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface investigation 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 a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Sheets



BELFAST, MAINE

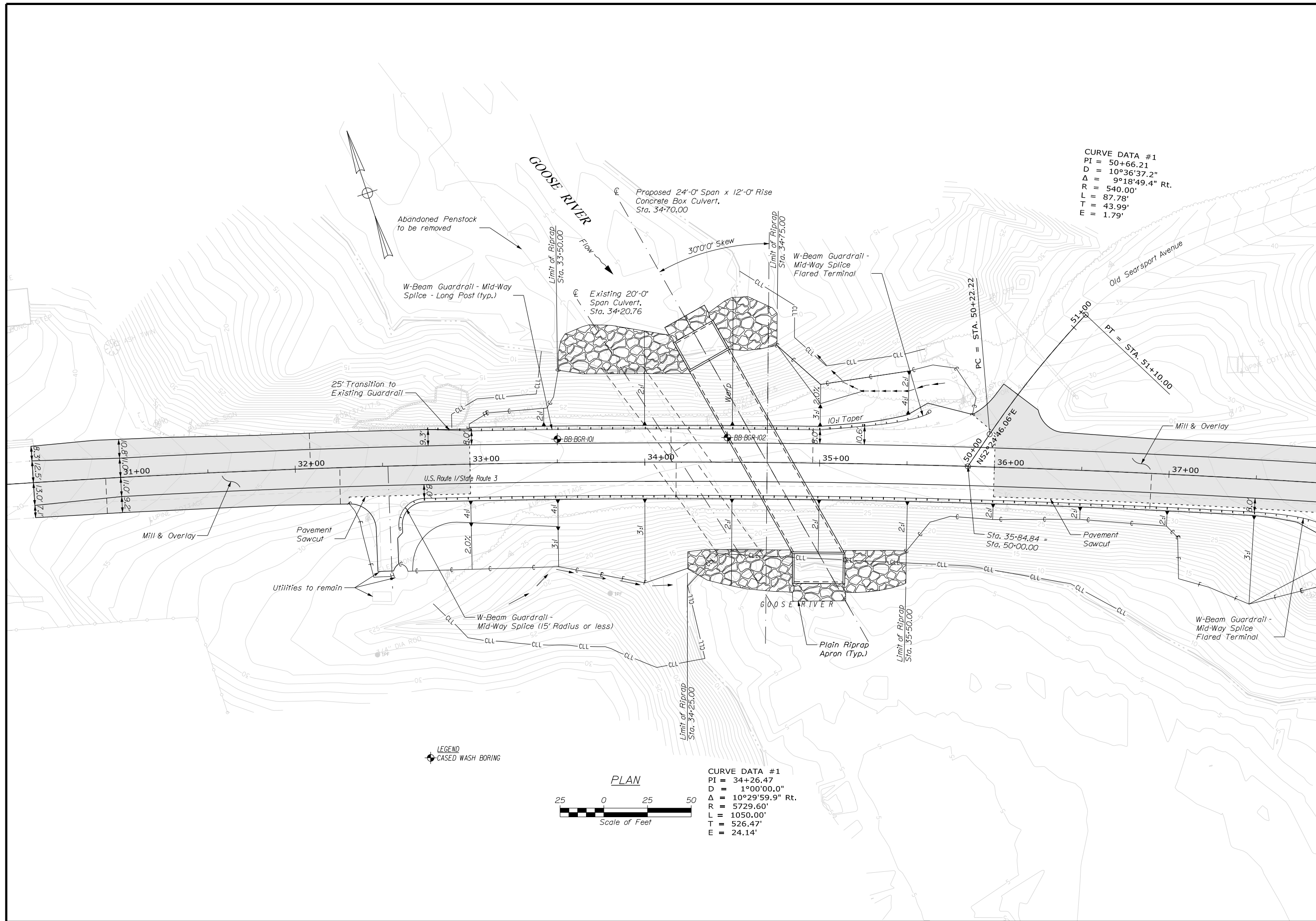


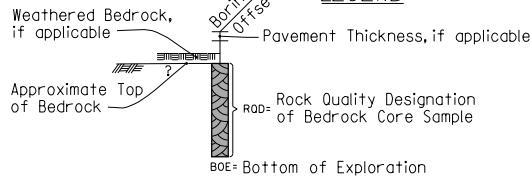
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0.4 Miles
1 inch = 0.45 miles

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Time: 7:20:01 AM

SHEET NUMBER <div style="font-size: 2em; text-align: center;">1</div> OF 4	GOOSE RIVER BRIDGE GOOSE RIVER BELFAST WALDO COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
		STP-2187(400)	
	LOCATION MAP	WIN 21874.00 BRIDGE NO. 2319 BRIDGE PLANS	





Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goose River Bridge #2319 carries U.S. Route 1 / State Route 3 over Location: Belfast, Maine				Boring No.: BB-BGR-101	
US CUSTOMARY UNITS				WIN: 21874.00					
Drillers:	New England Boring	Elevation (ft.):	29.9	Auger ID/OD:	5" Solid Stem				
Operator:	Schaefer/Titus	Sampler:	WMBSS	Sampler:	Standard Split Spoon				
Logged By:	Be Schaefer/Be Titus	Rtg Type:	Mobile Drill B-59	Home Wt./Fall:	140w/30"				
Date Start/Finish:	1/19/2018/09/40-13:45	Drilling Method:	Cased Wash Boring	Core Barrel:	NO-2				
Boring Location:	33+50.1, 13.5 ft L.	Casing ID/OD:	HE & NW	Water Level:	NOTE: USED WASH COVER AT 23.4 ft bgs.				
Home Efficiency Factor: 0.869		Home Type: Automatic B		Home Type: Hydraulic B		Rope & Corehead B			
Drill/Trip Test Q = Split Spoon Sample q = Unconsolidated Split Spoon Sample U = Thin Wall Tube Sample W = Unconsolidated Thin Wall Tube Sample N = Fluid Vane Shear Test R = Push/Pull Resistance S = Push/Pull Resistance T = Push/Pull Resistance		S = Push/Pull Resistance U = Thin Wall Tube Sample W = Unconsolidated Thin Wall Tube Sample N = Fluid Vane Shear Test R = Push/Pull Resistance S = Push/Pull Resistance T = Push/Pull Resistance		S = Push/Pull Resistance U = Thin Wall Tube Sample W = Unconsolidated Thin Wall Tube Sample N = Fluid Vane Shear Test R = Push/Pull Resistance S = Push/Pull Resistance T = Push/Pull Resistance		S = Push/Pull Resistance U = Thin Wall Tube Sample W = Unconsolidated Thin Wall Tube Sample N = Fluid Vane Shear Test R = Push/Pull Resistance S = Push/Pull Resistance T = Push/Pull Resistance			
Sample Information		Sample Information		Sample Information		Sample Information			
Depth (ft.)	Sample No.	Pen./Rec. (ft.)	Sample Depth (ft.)	Blades (ft.)	Blades (ft.)	Blades (ft.)	Blades (ft.)	Blades (ft.)	Blades (ft.)
0									
10	24/17	2.00 - 4.00	28/32/12/11	44	64				
20	24/15	5.00 - 7.00	4/4/7/6	11	16				
30	24/9	10.00 - 12.00	10/7/4/10	11	16				
40	24/0	15.00 - 17.00	5/14/18/13	32	46				
50	24/12	27.00 - 29.00	20/15/6/7	21	30				
60	24/16	30.00 - 32.00	25/45/31/40	76	110				
70	24/11	35.00 - 37.00	11/14/18/15	30	43				
80	21.6/18	45.00 - 46.80	33/60/63/50/3.6	123	178				
90	4.8/4.8	50.00 - 50.40	50/4.8"	---	---				
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Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM		
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)	
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	trace	0 - 10	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	little	11 - 20	
					some	21 - 35	
					adjective (e.g. sandy, clayey)	36 - 50	
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY		
		GC	Clayey gravels, gravel-sand-clay mixtures.				
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	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).		
		(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.			
	SILTS AND CLAYS (liquid limit greater than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	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.			
		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.	Consistency of Cohesive soils				
	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.	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				
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					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		
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 mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))					Rock Quality Designation (RQD): RQD (%) = <u>sum of the lengths of intact pieces of core* > 4 inches</u> length of core advance *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality Rock Mass Quality RQD (%) Very Poor ≤25 Poor 26 - 50 Fair 51 - 75 Good 76 - 90 Excellent 91 - 100		
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goose River Bridge #2319 carries U.S. Route 1 / State Route 3 over Goose River Location: Belfast, Maine				Boring No.: BB-BGR-101 WIN: 21874.00			
Driller: New England Boring				Elevation (ft.): 29.9				Auger ID/OD: 5" Solid Stem			
Operator: Schaefer/Titus				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: Be Schonewald				Rig Type: Mobile Drill B-59				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 1/19/2018; 09:40-13:45				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 33+50.1, 13.5 ft Lt.				Casing ID/OD: HW & NW				Water Level*: None Observed, caved at 23.4			
Hammer Efficiency Factor: 0.869				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA	29.5		5" HMA.	G#303091 Combined w/ 3D A-4, ML WC=15.0% LL=25 PL=22 PI=3 G#303091 Combined w/ 2D G#303092 A-1-a, SM WC=9.7%	
	1D	24/17	2.00 - 4.00	28/32/12/11	44	64				Brown, damp, very dense, fine to coarse Sandy GRAVEL, little silt, (Granular Fill). Frost depth to 3.0 ft bgs.		
5	2D	24/15	5.00 - 7.00	4/4/7/6	11	16	14			Grey brown, moist, very stiff, SILT, some sand, some gravel, little clay, (Miscellaneous Fill).		
							33					
							30					
							82					
							84					
10	3D	24/9	10.00 - 12.00	10/7/4/10	11	16	48			Layered: upper layer similar to 2D, and lower layer Silty GRAVEL, little to some sand; petroleum odor, (Miscellaneous Fill).		
							35					
							42					
							47					
							54					
15	MD	24/0	15.00 - 17.00	5/14/18/13	32	46	60					
							78					
							75					
							76					
							88					
20	4D	24/6	20.00 - 22.00	10/11/12/14	23	33	58			Brown grey, dense, Gravelly SAND, little silt, (Fill).		
							64					
							73					
							89					
25							67	6.9	Becomes very boney; difficult to penetrate.			




Remarks:
 Hammer ID #B-24

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 3

Boring No.: BB-BGR-101

<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>				<div>Project: Goose River Bridge #2319 carries U.S. Route 1 / State Route 3 over Goose River</div> <div>Location: Belfast, Maine</div>				<div>Boring No.: BB-BGR-101</div> <div>WIN: 21874.00</div>																																																																																																																																																																																																																																																																																						
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Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr></thead><tbody><tr><td rowspan="4">25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12"></td><td>Telescope NW Casing at 25.0 ft bgs, spun to 27.0 ft bgs, through Cobbles and Boulders.</td><td rowspan="12">G#303094 A-4, CL WC=52.6%</td></tr><tr><td>5D</td><td>24/12</td><td>27.00 - 29.00</td><td>20/15/6/7</td><td>21</td><td>30</td><td>103</td><td>Grey, Silty GRAVEL, some sand.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>108</td><td>5D (28.2-29.0 ft bgs) Black grading to dark brown, ORGANIC SILT with PEAT, some sand, trace gravel. 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	5D	24/12	27.00 - 29.00	20/15/6/7	21	30	103	Grey, Silty GRAVEL, some sand.																																																																																																																																																																																																																																																																																						
							108	5D (28.2-29.0 ft bgs) Black grading to dark brown, ORGANIC SILT with PEAT, some sand, trace gravel. Roller Coned ahead to 35.0 ft bgs.																																																																																																																																																																																																																																																																																						
30	6D	24/16	30.00 - 32.00	25/45/31/40	76	110	RC			-0.1		Brown grey, very dense, Sandy GRAVEL, little silt, (Glacial Till).	G#303095 A-1-b, GM WC=8.8%																																																																																																																																																																																																																																																																																	
35	7D	24/11	35.00 - 37.00	11/14/16/15	30	43						Brown grey, hard, SILT, some sand, some gravel, (Glacial Till).	G#303096 A-4, SM WC=8.0%																																																																																																																																																																																																																																																																																	
40										Attempted to core; Boulder from 38.0-39.5 ft bgs.																																																																																																																																																																																																																																																																																				
45	8D	21.6/18	45.00 - 46.80	33/60/63/50(3.6)	123	178	RC			Grey, damp, very dense, Silty GRAVEL, some sand, (Glacial Till).																																																																																																																																																																																																																																																																																				
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Operator: Schaefer/Titus						Datum: NAVD88						Sampler: Standard Split Spoon																																																																																																																												
Logged By: Be Schonewald						Rig Type: Mobile Drill B-59						Hammer Wt./Fall: 140#/30"																																																																																																																												
Date Start/Finish: 1/19/2018; 09:40-13:45						Drilling Method: Cased Wash Boring						Core Barrel: NQ-2"																																																																																																																												
Boring Location: 33+50.1, 13.5 ft Lt.						Casing ID/OD: HW & NW						Water Level*: None Observed, caved at 23.4																																																																																																																												
Hammer Efficiency Factor: 0.869						Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																																																																																																																																		
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Boring No.: BB-BGR-101																																																																																																																																								

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goose River Bridge #2319 carries U.S. Route 1 / State Route 3 over Goose River Location: Belfast, Maine				Boring No.: BB-BGR-102 WIN: 21874.00				
Driller: New England Boring				Elevation (ft.): 29.0				Auger ID/OD: 5" Solid Stem				
Operator: Schaefer/TiTus				Datum: NAVD88				Sampler: Standard Split Spoon				
Logged By: Be Schonewald				Rig Type: Mobile Drill B-59				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 1/9,11/2018				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"				
Boring Location: 34+47.2, 14.1 ft Lt.				Casing ID/OD: HW & NW				Water Level*: 21.8 ft, caved at 25.5 ft bgs.				
Hammer Efficiency Factor: 0.869				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA	28.0		12" HMA.	G#303097 A-2-4, SM WC=7.8% G#303098 A-4, ML WC=16.6% Non-Plastic	
	1D	20.4/14	2.00 - 3.70	29/43/39/50(2.4)	82	119				Brown, moist, very dense, Gravelly SAND, trace to little silt, broken gravel in bottom of sample, (Granular Fill).		
5	2D	24/13	5.00 - 7.00	24/23/6/3	29	42	50			Brown, moist, dense, SAND, some silt, some gravel, (Miscellaneous Fill).		
							26					
							17					
							21					
							23					
10	3D	24/8	10.00 - 12.00	WOH/2/2/3	4	6	10			Olive brown, mottled, medium stiff, Clayey SILT, little fine sand; possibly reworked, (Miscellaneous Fill).		
							14					
							19					
							20					
							21					
15	4D	24/13	15.00 - 17.00	2/2/3/4	5	7	13			Olive brown, mottled, medium stiff, SILT, some sand, little clay, trace gravel, (Miscellaneous Fill).		
							20					
							46					
							53					
							65					
20	5D	24/1	20.00 - 22.00	29/20/13/12	33	48				Dark grey brown, dense, GRAVEL, trace to little sand, trace silt; angular gravel, (Sandy Cobble and Boulder Fill). Changed to NW casing.		
25												
Remarks: Hammer ID #B-24.												
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 3 Boring No.: BB-BGR-102		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Goose River Bridge #2319 carries U.S. Route 1 / State Route 3 over Goose River Location: Belfast, Maine		Boring No.: BB-BGR-102 WIN: 21874.00			
Driller: New England Boring		Elevation (ft.): 29.0		Auger ID/OD: 5" Solid Stem					
Operator: Schaefer/TiTus		Datum: NAVD88		Sampler: Standard Split Spoon					
Logged By: Be Schonewald		Rig Type: Mobile Drill B-59		Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 1/9,11/2018		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"					
Boring Location: 34+47.2, 14.1 ft Lt.		Casing ID/OD: HW & NW		Water Level*: 21.8 ft, caved at 25.5 ft bgs.					
Hammer Efficiency Factor: 0.869		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q ₀ = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test									
Depth (ft.)	Sample Information							Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows		
25									Attempted to rock core, boulder from 24.8-26.8 ft bgs. Pop through boulder at 26.8 ft bgs, advanced hole by spinning NW Casing. Dark grey, very dense, fine to coarse Sandy GRAVEL, little to some silt, (Glacial Till). Attempt to core rock at 37.5 ft bgs; jam at 39.6 ft bgs. Top of Bedrock at Elev. -11.0 ft. R1: Bedrock: Hard, typically fresh, aphanitic to fine grained, medium grey METASANDSTONE, with few calcsilicate veins; deposition structure (flow) visible, and typically near vertical relic bedding. Typically close, low angle breaks; undulating, rough, typically discolored and open, with occasional mud infilling. Highly fractured and broken 40.5 to 40.6 and 42.9 to 43.1 ft bgs. Rock Mass Quality = Fair. R1: Core Times (min:sec) 40.0-41.0 ft (1:55) 41.0-42.0 ft (1:40) 42.0-43.0 ft (1:50) 43.0-44.0 ft (1:30) 44.0-44.8 ft (---) 97% Recovery R2: Bedrock: Similar to R1, Highly fractured and broken: 45.9 to 46.0, 46.4 to 46.8 and 47.2 to 47.3 ft bgs. Rock Mass Quality = Fair R2: Core Times (min:sec) 44.8-45.8 ft (2:10) 45.8-46.8 ft (2:25)
30									
35	6D	10.8/5	35.00 - 35.90	46/50(4.8")	---				
40	R1	57.6/56	40.00 - 44.80	RQD = 70%			NQ-2		
45	R2	60/60	44.80 - 49.80	RQD = 60%					
50									
Remarks: Hammer ID #B-24.									
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 2 of 3 Boring No.: BB-BGR-102	

[illegible]

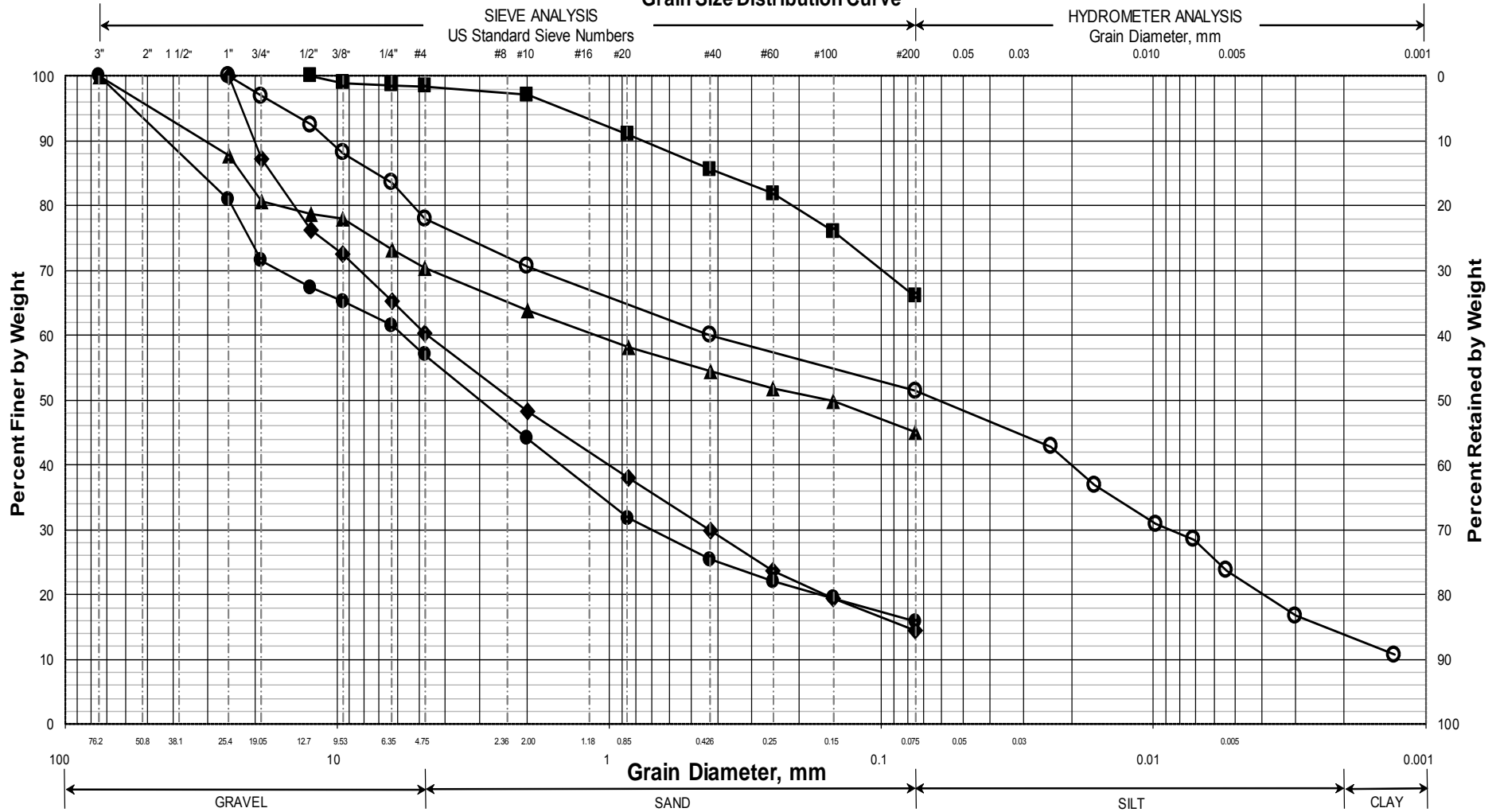
Appendix B

Laboratory Test Results

Work Number: 21874.00

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

Maine Department of Transportation Grain Size Distribution Curve

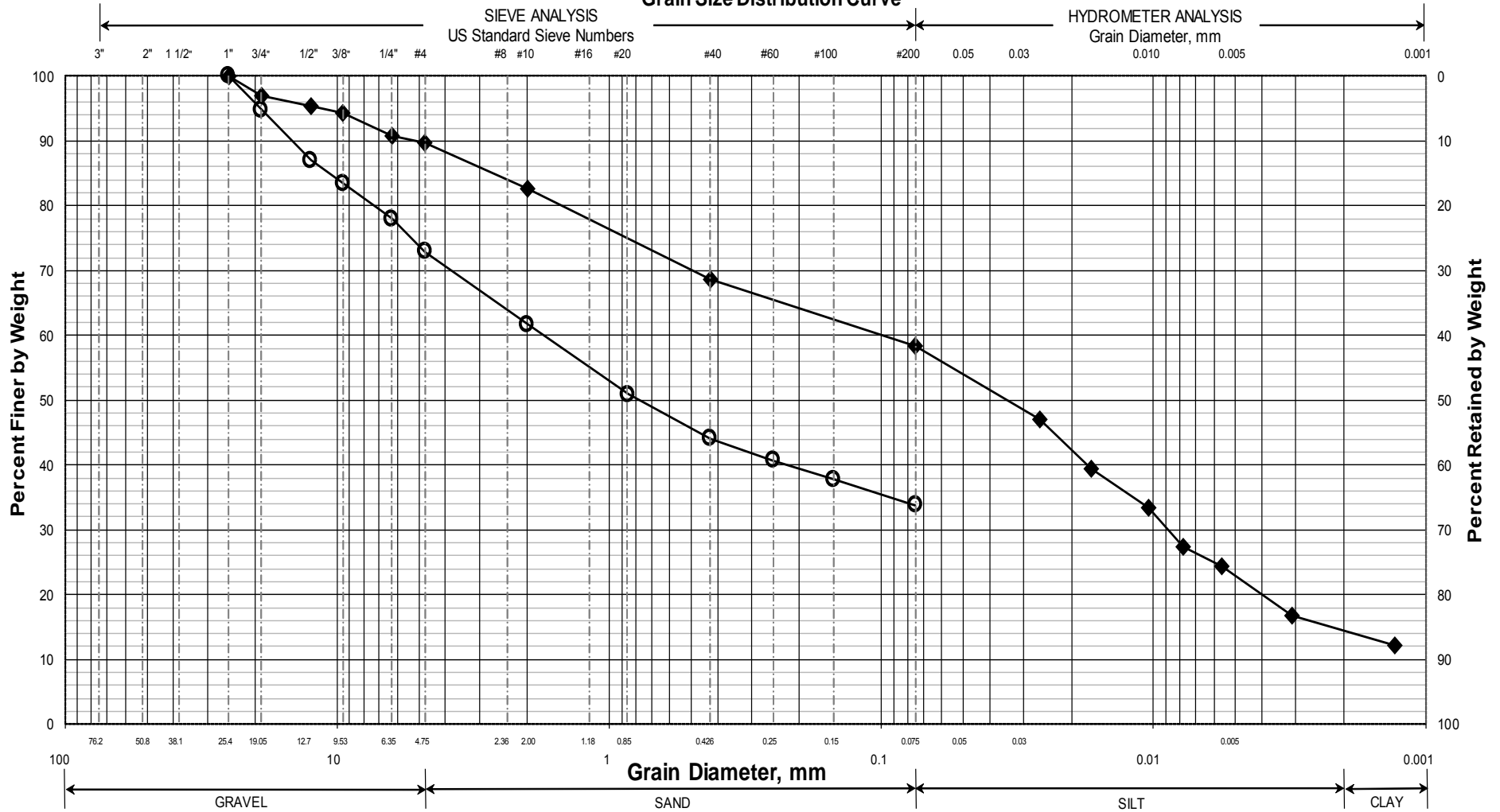


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-BGR-101/2D&3D	33+50.1	13.5 LT	5-7/10-12	SILT, some sand, some gravel, little clay.	15	25	22	3
◆	BB-BGR-101/4D	33+50.1	13.5 LT	20.0-22.0	Gravelly SAND, little silt.	9.7			
■	BB-BGR-101/5D	33+50.1	13.5 LT	27.0-29.0	SILT, some sand, trace gravel.	52.6			
●	BB-BGR-101/6D	33+50.1	13.5 LT	30.0-32.0	Sandy GRAVEL, little silt.	8.8			
▲	BB-BGR-101/7D	33+50.1	13.5 LT	35.0-37.0	SILT, some gravel, some sand.	8			
X									

WIN
021874.00
Town
Belfast
Reported by/Date
WHITE, TERRY A 1/24/2019

Maine Department of Transportation Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-BGR-102/2D	34+47.2	14.1 LT	5.0-7.0		7.8			
◆	BB-BGR-102/4D	34+47.2	14.1 LT	15.0-17.0		16.6			NP
■									
●									
▲									
×									

WIN	
021874.00	
Town	
Belfast	
Reported by/Date	
WHITE, TERRY A	1/24/2019



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **303091** Boring No./Sample No. **BB-BGR-101/2D&3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **1/19/2018** Received **2/9/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **33+50.1** Offset, ft: **13.5** LT Dbfg, ft: **5-7/10-12**

WIN/Town **021874.00 - BELFAST** Sampler: **BE SCHONEWALD**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	96.9
½ in. [12.5 mm]	92.5
⅜ in. [9.5 mm]	88.2
¼ in. [6.3 mm]	83.6
No. 4 [4.75 mm]	78.0
No. 10 [2.00 mm]	70.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	60.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	51.4
[0.0238 mm]	42.8
[0.0165 mm]	36.9
[0.0098 mm]	30.9
[0.0071 mm]	28.5
[0.0054 mm]	23.7
[0.0030 mm]	16.7
[0.0013 mm]	10.7

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	25
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	3
Specific Gravity, Corrected to 20°C (T 100)	2.71
Loss on Ignition, % (T 267)	
Water Content (T 265), %	15.0

Consolidation (T 216)

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

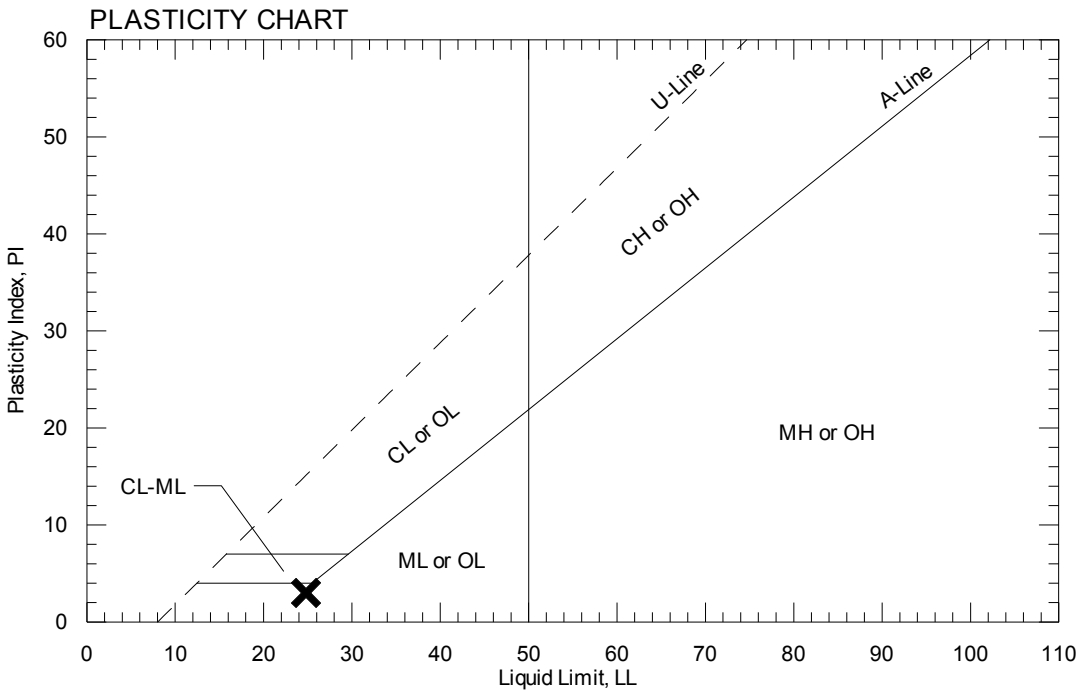
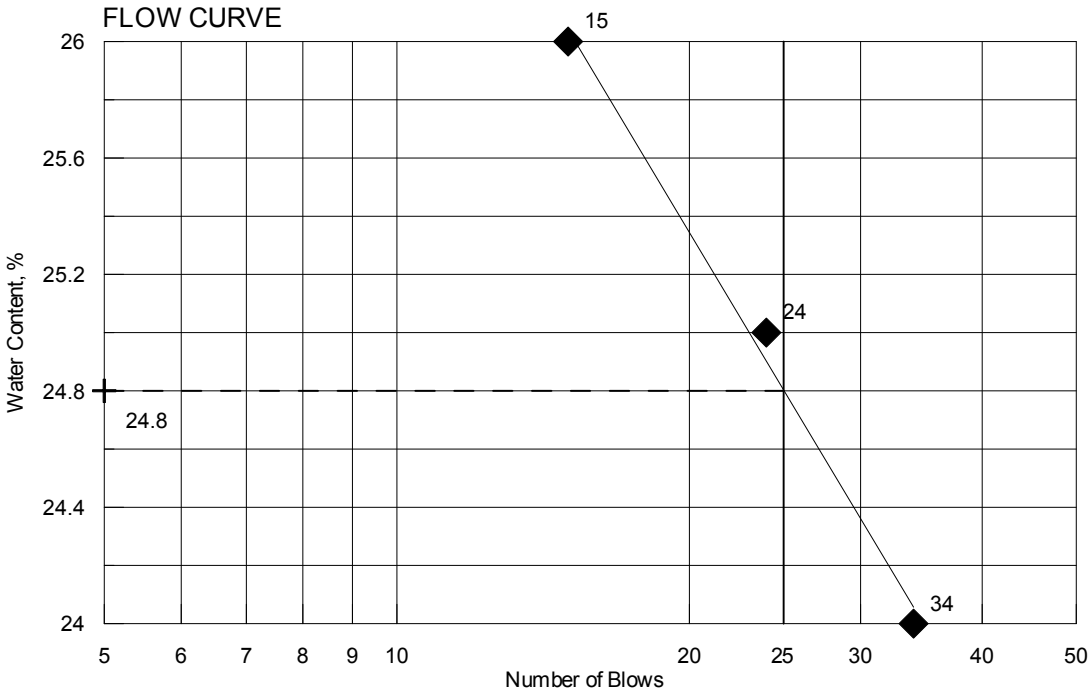
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **2/16/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Belfast	Reference No.	303091
WIN	021874.00	Water Content, %	15
Sampled	1/19/2018	Liquid Limit @ 25 blows (T 89), %	25
Boring No./Sample No.	BB-BGR-101/2D&3D	Plastic Limit (T 90), %	22
Station	33+50.1	Plasticity Index (T 90), %	3
Depth	5-7/10-12	Tested By	BBURR





GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **303098** Boring No./Sample No. **BB-BGR-102/4D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **1/9/2018** Received **2/9/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **34+47.2** Offset, ft: **14.1** LT Dbfg, ft: **15.0-17.0**

WIN/Town **021874.00 - BELFAST** Sampler: **BE SCHONEWALD**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	97.0
½ in. [12.5 mm]	95.3
⅜ in. [9.5 mm]	94.4
¼ in. [6.3 mm]	90.8
No. 4 [4.75 mm]	89.7
No. 10 [2.00 mm]	82.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	68.7
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	58.3
[0.0261 mm]	47.1
[0.0170 mm]	39.4
[0.0104 mm]	33.4
[0.0078 mm]	27.3
[0.0056 mm]	24.3
[0.0031 mm]	16.7
[0.0013 mm]	12.2

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.69
Loss on Ignition, % (T 267)	
Water Content (T 265), %	16.6

Consolidation (T 216)

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

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Reported by: **GREGORY LIDSTONE**

Date Reported: **2/16/2018**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Earth Pressure

Earth Pressure

Soil Parameters:

Assume existing material removed and replaced with material with properties similar to Soil Type 4, MaineDOT BDG Section 3.6.1.

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

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

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

Outlet walls fixed to box - At-Rest Earth Pressure - Rankine Theory

Reference: Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2

Formula for normally consolidated soils.

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

$$K_o = 0.47$$

Recommend: At-Rest Earth Pressure Coefficient, $K_o = 0.47$

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 cantiliver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

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

6.1 AT-REST LATERAL PRESSURES

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

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

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

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

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

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

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

^a Unloading cycle.

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

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

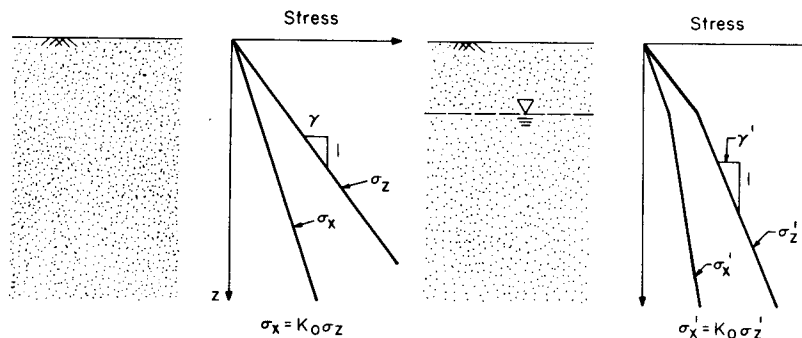


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

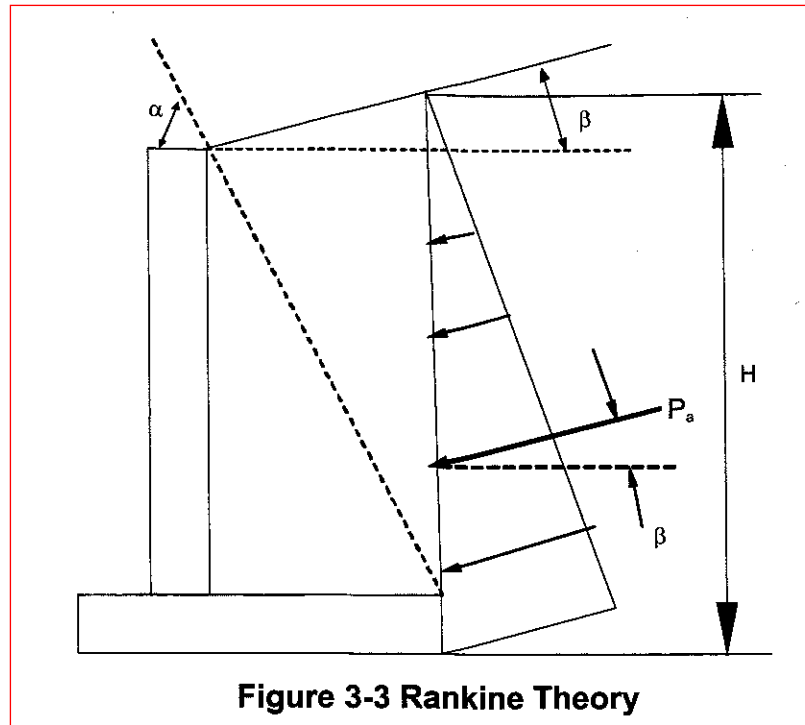


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing Resistance

Objective:

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

Given:

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

Assumptions:

1. The box culvert's embedment into the streambed is conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. The one foot thick layer of proposed Granular Borrow bedding material is neglected.
3. The proposed bearing elevation is approximately 0.0 feet.
4. Proposed finish roadway grade elevation is approximately 28.9 feet at the low point.
5. Proposed precast concrete box base is 26 feet wide.
6. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site. Use design N-value of 48 bpf for the consistency of the soils encountered at the box bearing elevation, based on BB-BGR-102 5D.
7. The bottom of the box culvert will be submerged for the structure's design life.

1. Estimate the factored bearing resistance at the Service Limit State:

The use of presumptive values may be used when sufficient knowledge of geological conditions at or near the structure site exists. AASHTO LRFD Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Medium dense to dense	8-14	10

The glacial till unit is dense in consistency. Recommend 10 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Assumed Foundation Width, Depth, and Water Surface

$$B := 26\text{ft}$$

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

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

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

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

$$\gamma_{\text{above}} := 125\text{pcf}$$

MaineDOT Bridge Design Guide p.
3-3
Soil Type 4

Foundation soils:

Foundation soils based on BB-BGR-102 5D

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

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

$$w_{\text{sat}} := 8.8\%$$

Moisture content of BB-BGR-101 5D.

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

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

$$\gamma_{1\text{sat}} = 145.8 \cdot \text{pcf}$$

$$N_{\text{design}} := 48$$

$$\phi := 40 \cdot \text{deg}$$

Peck, Hanson and Thornburn

Cohesion $c := 0$

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)

$$N_c := 75.3$$

$$N_q := 64.2$$

$$N_\gamma := 109.4$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume $L = 2B$

$$L := 2 \cdot B$$

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

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

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

$$s_c = 1.4$$

$$s_\gamma = 0.8$$

$$s_q = 1.4$$

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

$$\frac{D_f}{B} = 0.04$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 107.4$$

$$N_{\gamma m} = 87.5$$

$$N_{qm} = 91.1$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 88.6 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 39.9 \cdot \text{ksf}$$

Recommend a limiting value for the factored bearing resistance of 40 ksf for box bottom slabs 26 ft or greater on compacted granular fill.

3.4 Various Unit-Weight Relationships

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

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

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

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

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft ³	kN/m ³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

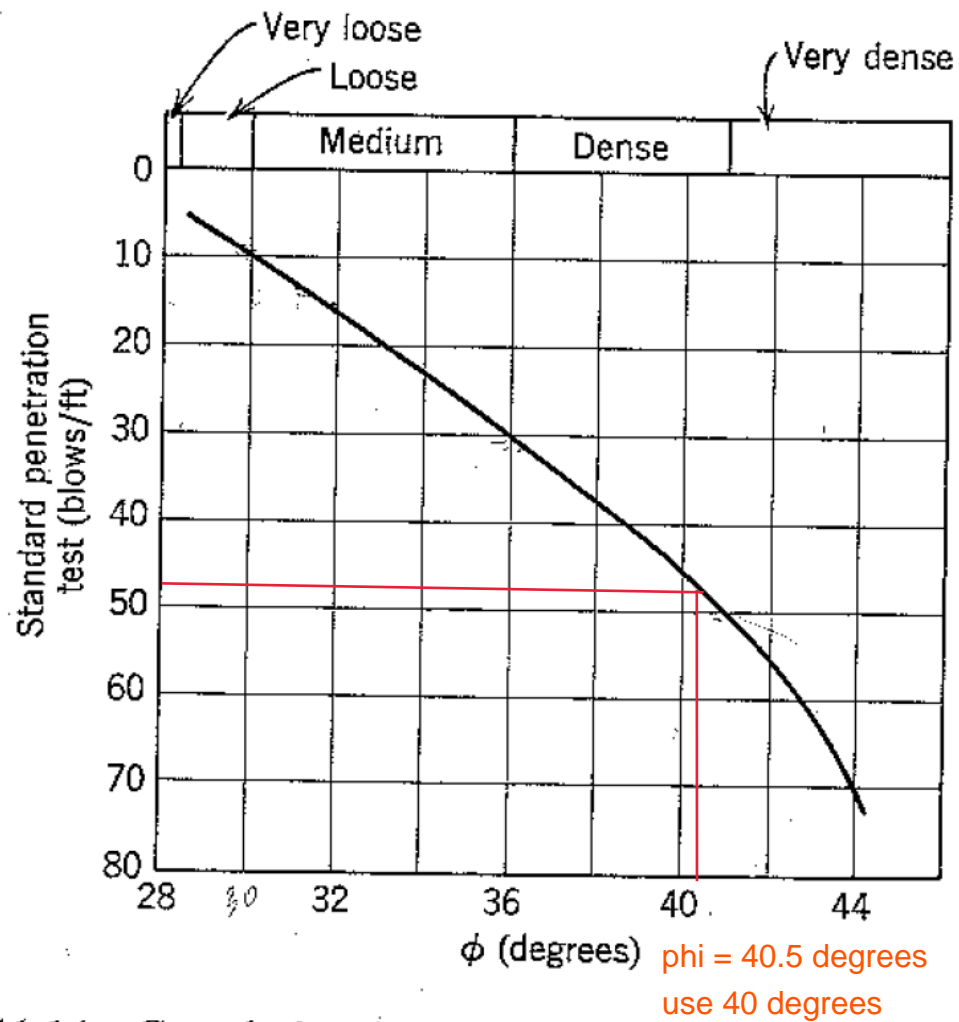


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

CHAPTER 3 - LOADS

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

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

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

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

C10.6.3.1.2a

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

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

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

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

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

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

in which:

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

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

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

where:

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

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

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

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

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

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

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

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

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

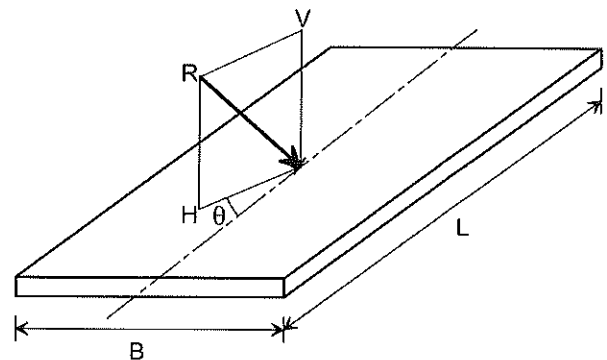


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

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

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

10.5.5.2.2—Spread Footings

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

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

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

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

CI0.5.5.2.2

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

Method/Soil/Condition			Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	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.

Modulus of Subgrade Reaction

Objective:

Estimate the modulus of subgrade reaction for the box culvert design

Given:

1. Limited lab data and SPT N-values.

Assumptions:

1. The proposed bearing elevation is approximately Elev. 0.0 feet.
2. Proposed finish roadway grade elevation is approximately 28.9 feet.
3. Proposed precast concrete box is 26.0 feet wide and 128 feet long (excluding slab connecting wingwalls).
4. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site.
5. The bottom of the box culvert will be submerged for the structure's design life.

Estimate the subgrade modulus for the precast box culvert

Published values of subgrade modulus in submerged, dense, sand:

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

Range of modulus of subgrade reaction

Dense sand: $k_s = 236 - 472$ pci

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

Range of modulus of subgrade reaction

Dense submerged coarse-grained soils: K_{v1} , 159 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: $k_{0.3} (k_1) = 516$ pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:

Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand

Submerged sand, proposed: $k_{s1} = 347$ pci

Range of published ranges is wide for use in design. Use Terzaghi's recommended value, $k_{s1} = 347$ pci for a 1 ft x 1 ft plate and adjust to the dimensions of the box culvert base. (Width B = 26 ft, Length L = 128 ft)

Square to rectangle base adjustment:

$$k_{s1} := 347 \text{ pci} \quad B := 26 \text{ ft} \quad L := 128 \text{ ft}$$

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

$$k = 255 \cdot \text{pci}$$

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

for either a horizontal or lateral modulus of subgrade reaction is

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

for either horizontal or vertical members

for depth variation

interest below ground

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

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 $B_s \approx 0$.
 used with the proper interpretation of the bearing-capacity equations (the d_i factors dropped) to give

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

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

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

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

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

depth of interest, say, the length of a pile

depth of interest

estimate of the exponent

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

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

compute

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

TABLE 9-1

Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

Bowles 5th Ed. Table 9-1 pg. 505

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

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

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:

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

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

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

For center:

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

From these we may write

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

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

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

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

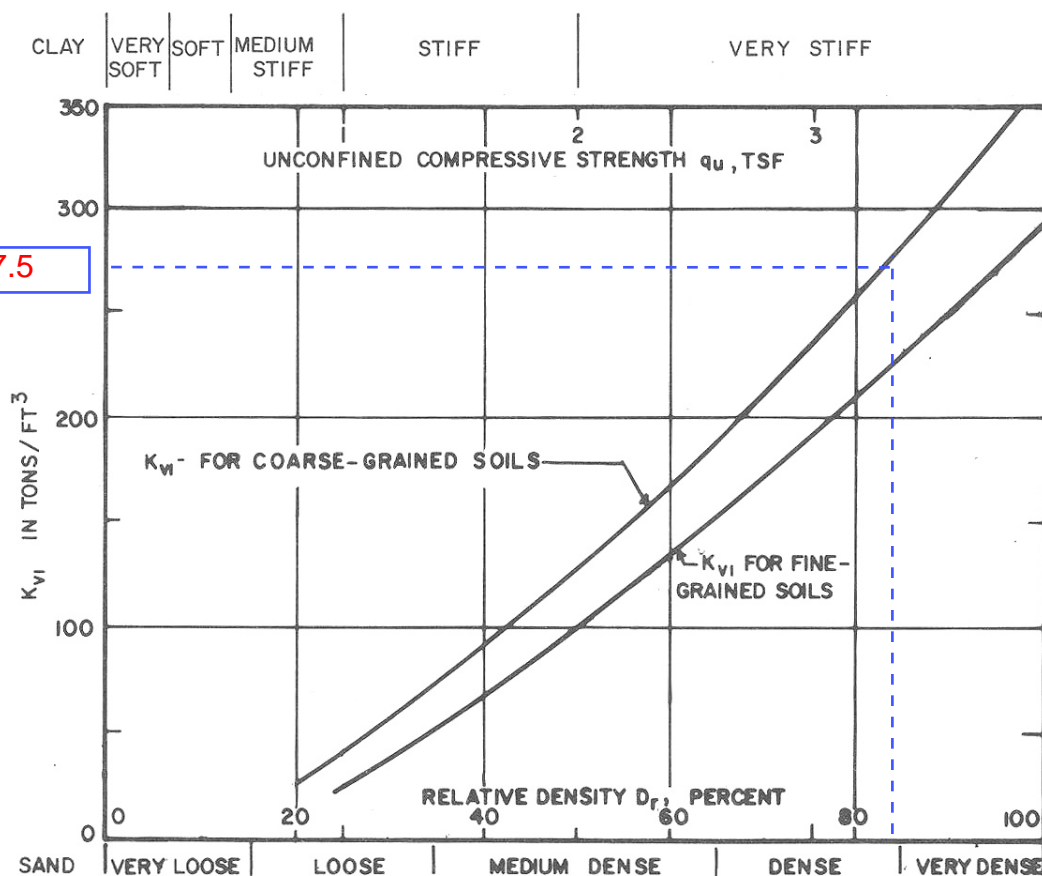
You should note that k_s does not depend on the contact pressure of the base q_o .

For corner:

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

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

$$275 / 2 = 137.5$$



DEFINITIONS

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

D = DEPTH OF FOOTING BELOW GROUND SURFACE

K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

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

$$137.5 \text{ ton/ft}^3 * 1.157 = 159.1 \text{ lb/in}^3$$

COARSE-GRAINED SOILS

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

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

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

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

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

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

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

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

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

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

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

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

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

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

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

or

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

where

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

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

Soil type	$k_{0.3}(k_1)$ MN/m ³
Dry or moist sand:	
Loose	8–25
Medium	25–125
Dense	125–375
Saturated sand:	
Loose	10–15
Medium	35–40
Dense	130–150
Clay:	
Stiff	10–25
Very stiff	25–50
Hard	>50

140 MN/m³ * 3.684 = 516 lb/in³

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 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 \times 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
Submerged sand, proposed values	25	80	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

$$300 \text{ tons/cu. ft} \times 1.16 = 347 \text{ lb/in}^3$$

problem can be corrected by using the corrected values. Experience shows that the corrected values are roughly equal to the original values (Fig. 3) or for a more exact equation (8):

If applied to the contact pressures p per unit of area of the supporting concentrated half of the ultimate equation (9).

V:
f

Values
Range
Proposed

For rec

* High

If the subgrade is 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 value in the case of medium sand if its real value is equal to the lower limiting value of 60 tons/cu. ft.

The proposed values of \bar{k}_{s1} for medium sand, Table 1, are based on the results of tests on normally consolidated beams and rafts supported on perfectly rigid sand.

The \bar{k}_{s1} values of the tests can be used for the design of such tests is too small. The test results can be used for the design of the block should be used.

If the contact pressure p is used, the value is:

For $l = \infty$, $k_{s1} =$ loaded subgrade

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: **Belfast, Maine**

Case 1 - coarse grained granular fill soils W=10%

$$DFI_1 := 1400 \quad d_1 := 79.2 \cdot \text{in}$$

$$DFI_2 := 1500 \quad d_2 := 82.1 \cdot \text{in}$$

Approximate DFI at project = 1450 find frost depth by interpolation:

$$DFI_3 := 1450$$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1) \cdot (d_2 - d_1)}{(DFI_2 - DFI_1)} \quad d_3 = 80.7 \cdot \text{in}$$

$$\text{Depth of Frost Penetration} \quad d_3 = 6.7 \cdot \text{ft}$$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

--- ModBerg Results ---

Project Location: Belfast, Maine

Air Design Freezing Index = 1188 F-days
N-Factor = 0.80
Surface Design Freezing Index = 950 F-days
Mean Annual Temperature = 45.5 deg F
Design Length of Freezing Season = 118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	49.2	10.0	105.0	23	28	1.1	1.0	1,512

t = Layer thickness, in inches.

w% = Moisture content, in percentage of dry density.

d = Dry density, in lbs/cubic ft.

Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).

Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).

Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).

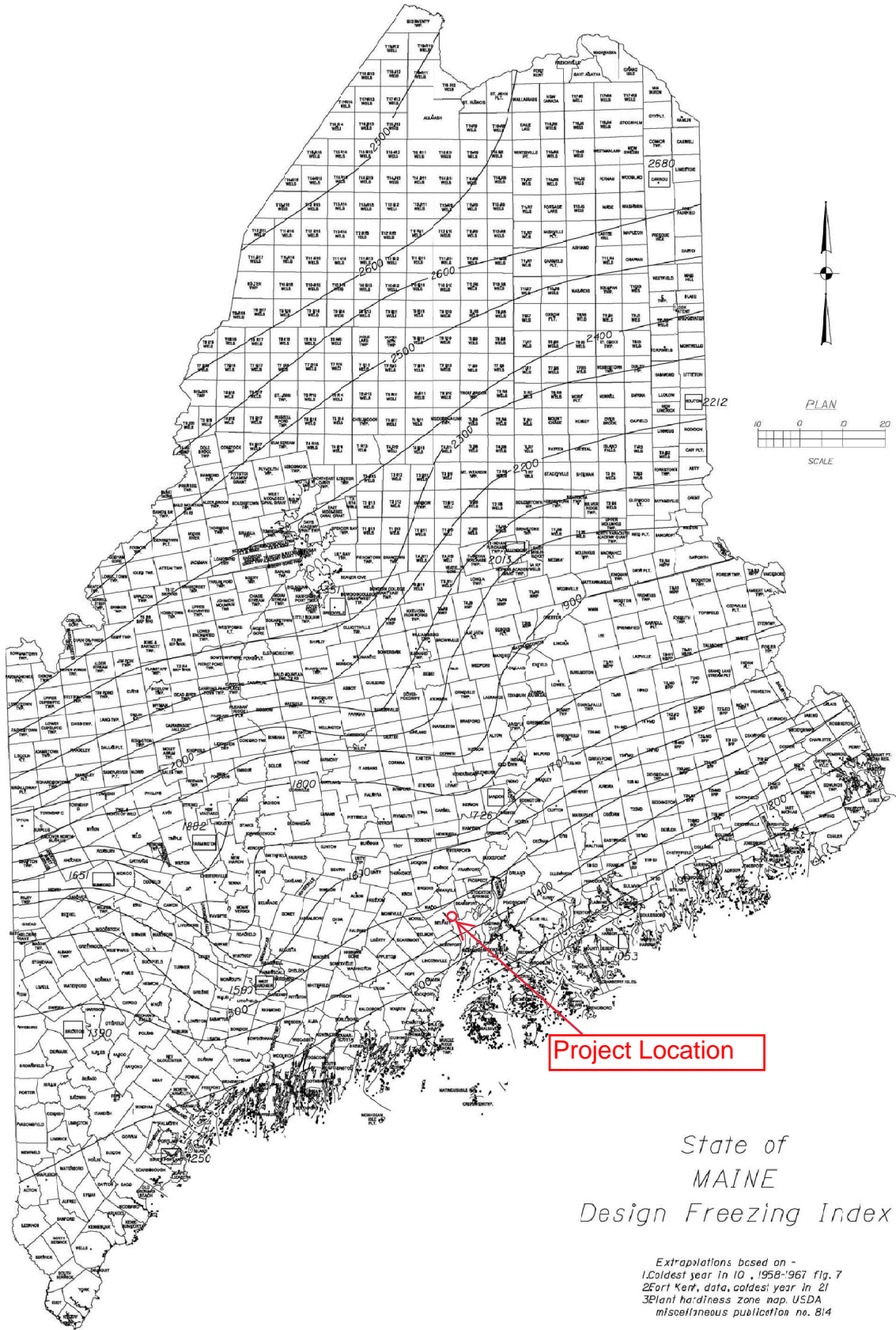
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

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

Total Depth of Frost Penetration = 4.10 ft = 49.2 in.

Recommendation: 6.7 feet for design of foundations constructed on coarse grained soils

Figure 5-1 Maine Design Freezing Index Map



5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

Interpolate
DFI = 1450