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**MAINE DEPARTMENT OF TRANSPORTATION**  
Letter of Transmittal

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Attached is one (1) copy of Soils Report 2018-07, "GEOTECHNICAL DESIGN REPORT For the Replacement of: SHELDON BRIDGE, POORS MILL ROAD, WARREN BROOK, BELFAST, MAINE" dated: February 6, 2018.

This report is available in TEDOCS as Document # 1686267.

att: 1 of 2018-07

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

**GEOTECHNICAL DESIGN REPORT**

*For the Replacement of:*

**SHELDON BRIDGE  
POORS MILL ROAD OVER WARREN BROOK  
BELFAST, MAINE**

*Prepared by:*

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Waldo County  
WIN 21666.00

Soils Report 2018-07  
Bridge No. 5557

Fed. No. STP-2166(600)  
February 6, 2018

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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 Sheldon Bridge, which carries Poors Mill Road over Warren Brook 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 structure was constructed in 1954 and consists of 21-foot span structural plate arch on concrete, strip footings, approximately 65 feet in length. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culvert is rated 4 (poor condition) because of minor scaling and moderate pitting. The downstream side of the culvert was reinforced with a concrete collar after the end plates were damaged. The plate arch is buckling on the upstream side and granite stones forming the upstream headwall have fallen into the channel. The footings are exposed up to 18 inches. The structure has a FHWA Sufficiency Rating of 68.7.

The proposed replacement structure will be a 21-foot span by 9-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 on approximately the same horizontal alignment as the existing plate arch. Poors Mill Road will be closed at the project site and traffic will be maintained via a signed detour. The bridge replacement project will last one construction season.

## **2.0 GEOLOGIC SETTING**

Sheldon Bridge carries Poors Mill Road over Warren Brook approximately .93 miles west of Marsh Road, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of Maine (2013) indicates the surficial soil unit in the vicinity of the bridge is the Presumpscot Formation, which consists of glaciomarine silt, clay and sand. This soil unit typically overlies an irregular surface of glacial till and may include areas of till exposed at the ground surface. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Glacial till includes two varieties: basal till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the project site as the Bucksport Formation consisting of biotite granofels interlayered with calc-silicate granofels.

### **3.0 SUBSURFACE INVESTIGATION**

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

Test borings BB-BWB-101 and BB-BWB-102 were drilled on March 8, 2017 by S.W. Cole Explorations, LLC. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – 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 S.W. Cole drill rig is equipped with a 140-pound, rope and cathead hammer falling 30 inches. No correction of N-values is required for the N-values obtained with the standard rope and cathead system where common practice assumes rope and cathead systems have a theoretical 60 percent hammer efficiency. The theoretical hammer efficiency factor, the N-values, and the  $N_{60}$  values 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 three (3) standard grain size analyses with natural water content. 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.

Bedrock laboratory testing consisted of one unconfined compression test with determination of elastic moduli. The result of bedrock laboratory test is included in Appendix B – Laboratory Test Results. Bedrock laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

## **5.0 SUBSURFACE CONDITIONS**

Subsurface conditions encountered generally consisted of granular fill soils and glacial till, underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

### **5.1 Fill**

A layer of fill was encountered in the borings. The thickness of the fill unit encountered was approximately 16 to 19 feet at the boring locations. The unit encountered generally consisted of:

- Brown, moist to wet, medium sand, some to little gravel, little to trace silt;
- Grey, wet, sandy gravel, little silt;
- Brown, wet, gravel, trace sand, trace silt, with wood debris.

Based on drilling attitude, it is inferred that cobbles were present from approximately 2.9 to 3.5 feet below ground surface (bgs) and 14.4 to 15.3 feet bgs in BB-BWB-101.

SPT  $N_{60}$ -values in the layer ranged from 21 to 77 blows per foot (bpf), indicating the layer is medium dense to very dense in consistency. One (1) grain size analysis resulted in the material being classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 10 percent.

### **5.2 Glacial Till**

Glacial till was encountered beneath the fill in the borings. The thickness of the glacial till deposit encountered ranged from approximately 3 to 7 feet thick at the boring locations. The glacial till deposit generally consisted of:

- Brown, wet, fine sand, some silt, some coarse gravel;
- Brown, wet, medium sand, some silt, some fine gravel.

SPT  $N_{60}$ -values in the glacial till deposit ranged from 23 to 52 bpf indicating the soil is medium dense to dense in consistency. Two (2) grain size analyses conducted on samples of the glacial till resulted in the samples being classified as A-2-4 under the AASHTO Soil Classification System and SM under the USCS. The natural water content of the samples tested ranged from approximately 9 to 11 percent.

### 5.3 Bedrock

Bedrock was encountered and cored in borings BB-BWB-101 and BB-BWB-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-BWB-101	2+74.2	9.2 ft Rt	23.5	120.1	87, 85
BB-BWB-102	2+96.4	9.5 ft Lt	22.4	121.7	73, 55

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

The bedrock at the site is identified as dark greenish-grey, metamorphic, gneiss, hard, very slight weathering, joints/fractures are moderately dipping, close to moderately close, and healed, with white calcite vein infilling. Detailed bedrock descriptions and the RQD core run are provided on the boring logs on Sheet 3 – Boring Logs and in Appendix A – Boring Logs. One laboratory unconfined compressive strength with elastic modulus test was conducted on one bedrock sample. The testing yielded an unconfined compressive strength of 20,981 psi and a Young's modulus value of 7,320 psi.

### 5.4 Groundwater

Groundwater elevations were recorded at 11.4 feet bgs and 13.3 feet bgs in the borings at the completion of the borings. 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 STRUCTURE ALTERNATIVES

Two alternatives with openings of at least 1.2 times the bankfull width (BFW) were considered for the replacement bridge structure. The structure types and spans were:

- A 21-foot span precast concrete box, and
- Aluminum structural plate box culvert with wingwalls.

A 21-foot span precast concrete box culvert has the advantages of efficiency of installation, low maintenance and lifecycle costs. An aluminum box culvert is efficient to install, because it can be placed in sections, and the light-weight aluminum culvert does not require heavy equipment to place. The precast concrete box culvert was chosen as the preferred alternative.

Refer to the project Draft Preliminary Design Report (PDR), dated June 14, 2017 for more information on the analysis and discussion of structure alternatives. 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, while maintaining the existing thalweg.

## **7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS**

### **7.1 Precast Concrete Box Culvert Design**

The proposed replacement structure will consist of a 21-foot span by 9-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. The riprap apron should be embedded 6 inches into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be 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, 7<sup>th</sup> Edition, 2014 with up to 2016 interim revisions (LRFD).

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 outlet and inlet walls will be slope-tapered to match the 2H:1V sideslopes of the roadway embankment. The left and right 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 and deformations resulting from creep, temperature, and shrinkage 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 130 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 23 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 16.8 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 5.0 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 where the volume of soil displaced by the foundation will result in a lower net applied stress. 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 104 pounds per cubic inch (pci).

## **7.2 Settlement**

The granular fill 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 medium dense to dense fill. 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. 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 a DFI from the Modberg database of approximately 950 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 wingwalls 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. Riprap aprons will be installed at both ends of the culvert.

## **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 box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow. 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 Granular Borrow with a bottom of excavation elevation of approximately 129.1 feet. Based on the soils encountered in the borings, 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 for specific application to the proposed replacement of Sheldon Bridge in Belfast, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

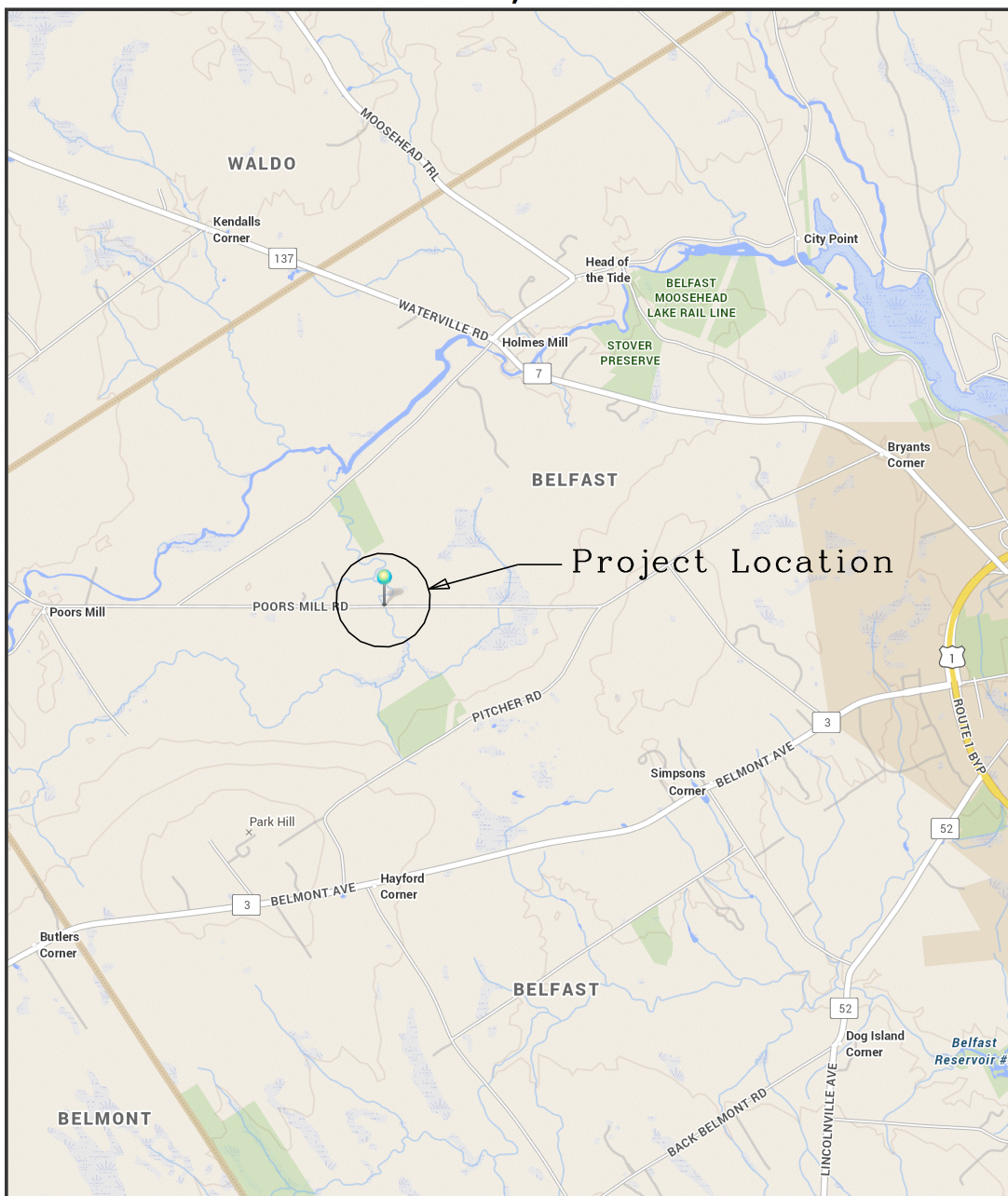
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

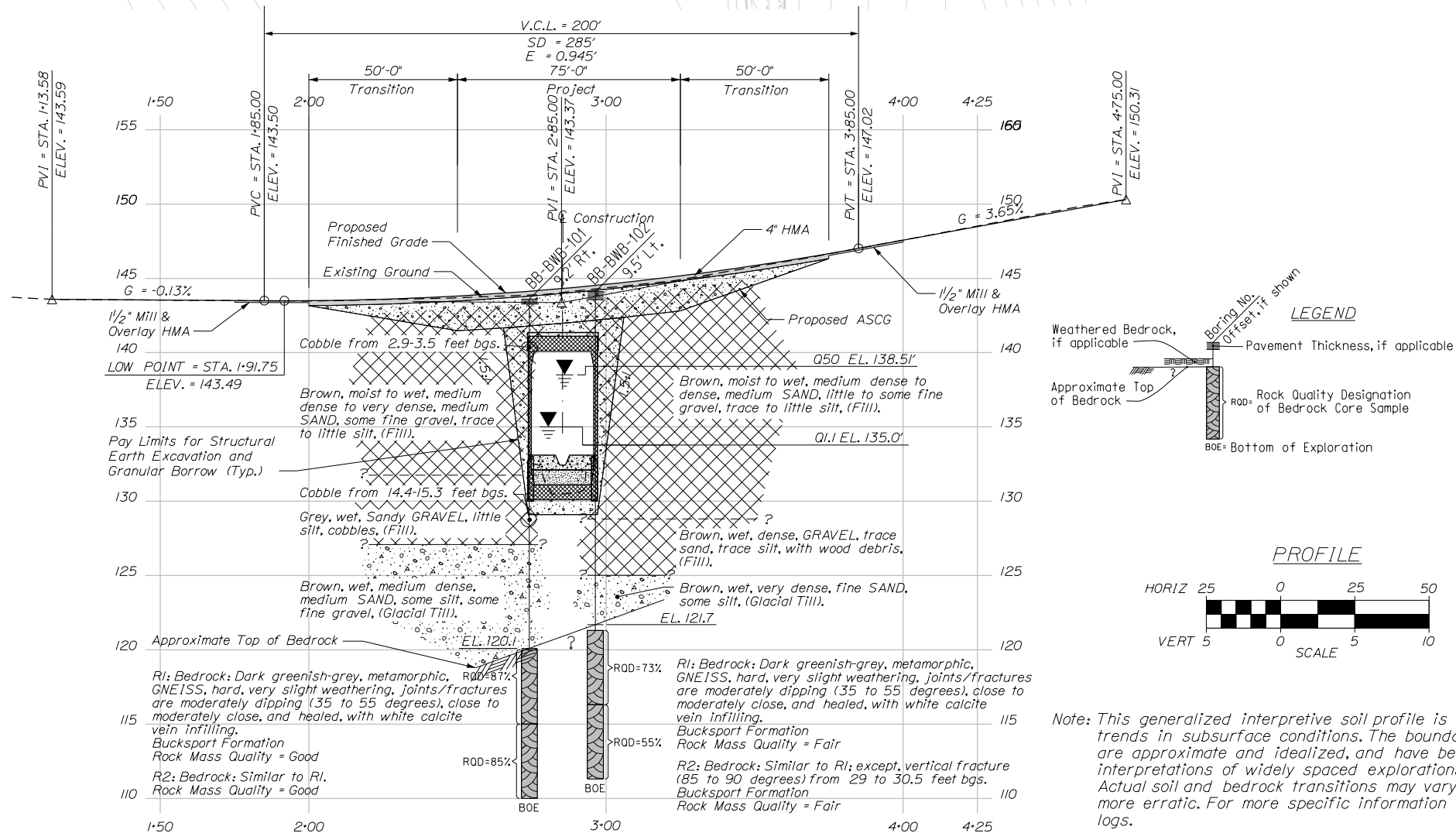
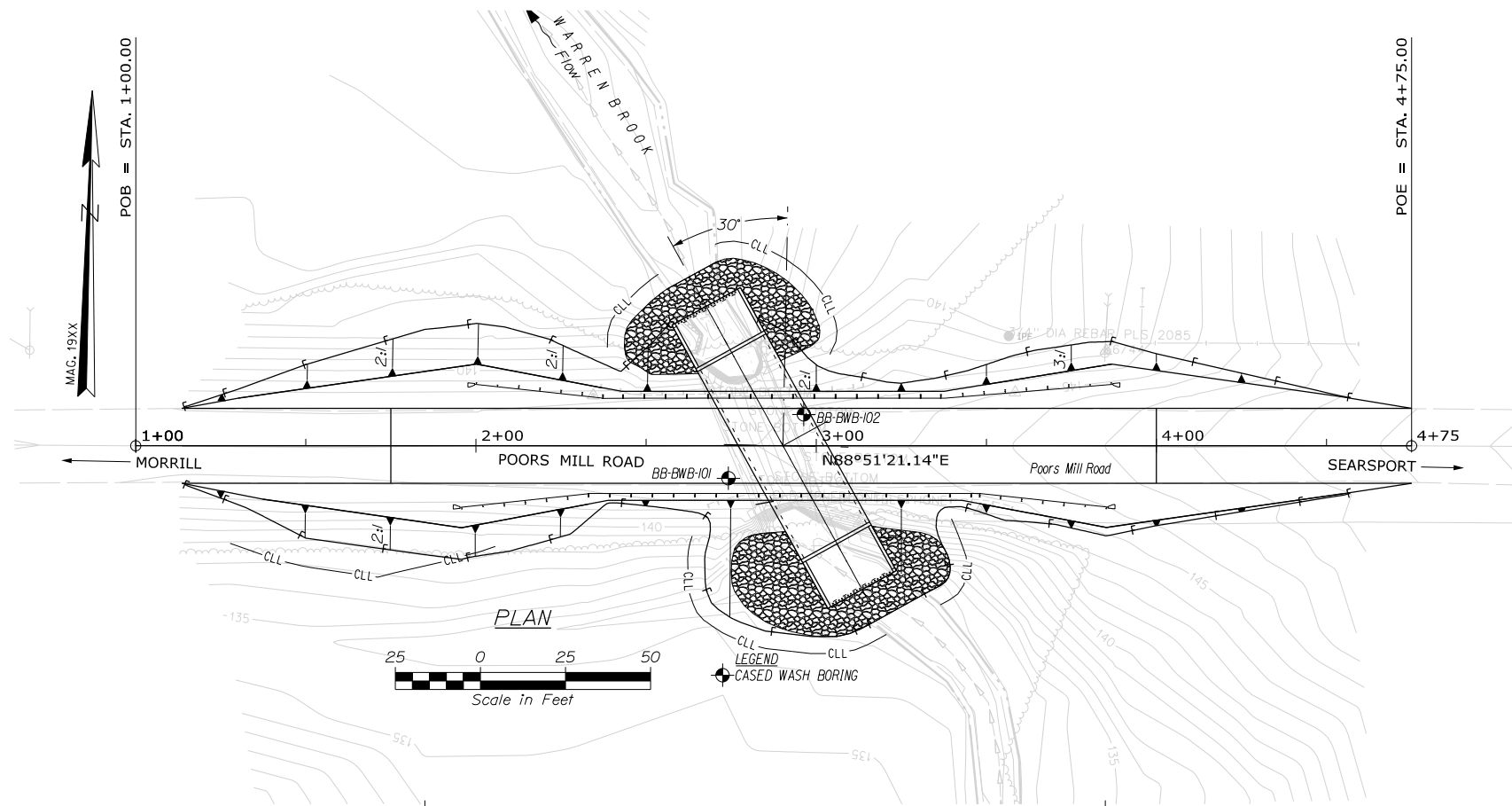


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0.7 Miles  
1 inch = 0.75 miles

Date: 11/3/2017  
Time: 8:54:26 AM

<p><b>SHEET NUMBER</b></p> <p><b>1</b></p> <p>OF 3</p>	<p><b>SHELDON BRIDGE</b></p> <p><b>WARREN BROOK</b></p> <p>BELFAST WALDO COUNTY</p>	<p><b>STATE OF MAINE</b></p> <p><b>DEPARTMENT OF TRANSPORTATION</b></p>
		<p><b>STP-2166(600)</b></p>
	<p><b>LOCATION MAP</b></p>	<p><b>WIN</b></p> <p><b>21666.00</b></p> <p>BRIDGE NO. 5557 BRIDGE PLANS</p>



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

PROJ. MANAGER	J. KITTREDE	BY	DATE
CHECKED-REVIEWED	K. NASH	MLACHANCE	8/6/2017
DESIGN-DETAILED	N. SHERWOOD	T. WHITE	MAR 2017
DESIGN-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
SIGNATURE	P.E. NUMBER	DATE	

SHELDON BRIDGE  
WARREN BROOK  
WALDO COUNTY  
BELFAST  
BORING LOCATION PLAN &  
INTERPRETIVE SUBSURFACE PROFILE



[illegible]



## **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															
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	some	21 - 35															
			GC	Clayey gravels, gravel-sand-clay mixtures.	adjective (e.g. sandy, clayey)	36 - 50															
	SANDS  (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	<b>TERMS DESCRIBING DENSITY/CONSISTENCY</b>  <u>Coarse-grained soils</u> (more than half of material is larger than No. 200 sieve); Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).																
		(little or no fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.																	
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures																	
			SC	Clayey sands, sand-clay mixtures.																	
FINE-GRAINED SOILS  (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS  (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	<b>Density of Cohesionless Soils</b>  Very loose Loose Medium Dense Dense Very Dense			<b>Standard Penetration Resistance N-Value (blows per foot)</b>  0 - 4 5 - 10 11 - 30 31 - 50 > 50														
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																		
		OL	Organic silts and organic silty clays of low plasticity.																		
		SILTS AND CLAYS  (liquid limit greater than 50)	MH							Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.											
	CH		Inorganic clays of high plasticity, fat clays.																		
	OH		Organic clays of medium to high plasticity, organic silts.																		
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.	<b>Consistency of Cohesive soils</b>  Very Soft Soft Medium Stiff  Stiff  Very Stiff Hard			<b>SPT N-Value (blows per foot)</b>  WOH, WOR, WOP, <2 2 - 4 5 - 8  9 - 15  16 - 30 >30			<b>Approximate Undrained Shear Strength (psf)</b>  0 - 250 250 - 500 500 - 1000  1000 - 2000  2000 - 4000 over 4000			<b>Field Guidelines</b>  Fist easily penetrates Thumb easily penetrates Thumb penetrates with moderate effort Indented by thumb with great effort Indented by thumbnail Indented by thumbnail with difficulty							
		<b>Rock Quality Designation (RQD):</b> RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^*}{\text{length of core advance}}$  *Minimum NQ rock core (1.88 in. OD of core)  Correlation of RQD to Rock Mass Quality <table><tr><th>Rock Mass Quality</th><th>RQD (%)</th></tr><tr><td>Very Poor</td><td>≤25</td></tr><tr><td>Poor</td><td>26 - 50</td></tr><tr><td>Fair</td><td>51 - 75</td></tr><tr><td>Good</td><td>76 - 90</td></tr><tr><td>Excellent</td><td>91 - 100</td></tr></table>															Rock Mass Quality	RQD (%)	Very Poor	≤25	Poor
Rock Mass Quality	RQD (%)																				
Very Poor	≤25																				
Poor	26 - 50																				
Fair	51 - 75																				
Good	76 - 90																				
Excellent	91 - 100																				
<b>Desired Soil Observations (in this order, if applicable):</b> 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					<b>Desired Rock Observations (in this order, if applicable):</b> 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))																
<b>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</b>					<b>Sample Container Labeling Requirements:</b> <table><tr><td>WIN</td><td>Blow Counts</td></tr><tr><td>Bridge Name / Town</td><td>Sample Recovery</td></tr><tr><td>Boring Number</td><td>Date</td></tr><tr><td>Sample Number</td><td>Personnel Initials</td></tr><tr><td>Sample Depth</td><td></td></tr></table>							WIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth	
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Sheldon Bridge (No. 5557) carries Poors Mill Road over Warren Brook Location: Belfast, Maine		Boring No.: BB-BWB-101		
Drilling Contractor: S.W. Cole Explorations, LLC				Elevation (ft.): 143.6		Auger ID/OD: 5-inch Solid Stem		
Operator: K. Hanscom				Datum: NAVD 88		Sampler: Standard Split Spoon		
Logged By: N. Strout				Rig Type: Diedrich D50		Hammer Wt./Fall: 140#/30" and 300#/16"		
Date Start/Finish: 3/8/2017				Drilling Method: Cased Wash Boring		Core Barrel: NQ2 (2-inch-diameter)		
Boring Location: Sta 2+74.2, 9.2 feet Rt.				Casing ID/OD: HW 4"/4.5" and NW 3"/3.5"		Water Level*: 13.3 feet bgs		
<div> <div> Definitions: D = Spilt Spoon Sample S = Sample off Auger Flights B = Bucket Sample off Auger Flights MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MV = Unsuccessful Field Vane Shear Test Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer </div> <div> MU = Unsuccessful Thin Wall Tube Sample Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing </div> <div> WO1P = Weight of 1 Person S<sub>u</sub> = Peak/Remolded Field Vane Undrained Shear Strength (psf) S<sub>u</sub>(lab) = Lab Vane Undrained Shear Strength (psf) q<sub>p</sub> = Unconfined Compressive Strength (ksf) N-value = Raw Field SPT N-value T<sub>v</sub> = Pocket Torvane Shear Strength (psf) WC = Water Content, percent ≈ ≈ Similar or Equal too </div> <div> LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test </div> </div>								
Depth (ft.)	Sample Information						Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows		
0						SSA	143.27	4-inch-thick layer of pavement Brown, moist, very dense, SAND, some gravel, trace silt, (Fill). Cobbles from 2.9 to 3.5 feet bgs. Similar to Sample 1D; except medium dense.
	1D	23/20	1.00 - 2.92	31/39/38/100-5"	77			
5						HW		Brown, wet, medium dense, medium SAND, some fine gravel, little silt, (Fill). Elev. approx. 130 BOF Cobbles from 14.4 to 15.3 feet bgs. Grey, wet, Sandy GRAVEL, little silt, cobbles, (Fill).
	2D	24/16	5.00 - 7.00	22/16/12/11	28	78		
10								Brown, wet, medium dense, medium SAND, some silt, some fine gravel, (Glacial Till). Similar to Sample 5D. Top of Bedrock at Elevation 120.1 feet. RC from 23.5 to 23.6 feet bgs. R1: Bedrock: Dark greenish-grey, metamorphic, GNEISS, hard, very slight
	3D	24/8	10.00 - 12.00	11/11/12/16	23	50		
15								Top of Bedrock at Elevation 120.1 feet. RC from 23.5 to 23.6 feet bgs. R1: Bedrock: Dark greenish-grey, metamorphic, GNEISS, hard, very slight
	4D	5/3	15.30 - 15.72	100-5"	- -	OPEN		
20								Top of Bedrock at Elevation 120.1 feet. RC from 23.5 to 23.6 feet bgs. R1: Bedrock: Dark greenish-grey, metamorphic, GNEISS, hard, very slight
	5D	24/14	17.00 - 19.00	16/16/14/16	30	37		
25								Top of Bedrock at Elevation 120.1 feet. RC from 23.5 to 23.6 feet bgs. R1: Bedrock: Dark greenish-grey, metamorphic, GNEISS, hard, very slight
	6D	24/9	20.00 - 22.00	10/11/12/19	23	22		
	R1	60/57	23.60 - 28.60	RQD = 87%		100 NQ2	120.10	


Remarks:  
-bgs = below existing ground surface (roadway)  
-Sampler driven with 140# hammer with 30" drop  
-Casing driven with 300# hammer with 16" drop  
-Groundwater at 13.3 feet bgs with NW casing to 23.5 feet  
-Groundwater level at 9 feet bgs after pulling casing  
-Borehole caved at 10 feet bgs

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 2  
Boring No.: BB-BWB-101

[illegible]

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Sheldon Bridge (No. 5557) carries Poors Mill Road over Warren Brook <b>Location:</b> Belfast, Maine				<b>Boring No.:</b> BB-BWB-102 <b>WIN:</b> 21666.00																																																																																																																																																																																																																																																																																																																						
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<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>						Project: Sheldon Bridge (No. 5557) carries Poors Mill Road over Warren Brook Location: Belfast, Maine				Boring No.: BB-BWB-102				
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Operator: K. Hanscom						Datum: NAVD88				Sampler: Standard Split Spoon				
Logged By: N. Strout						Rig Type: Diedrich D50				Hammer Wt./Fall: 140#/30" and 300#/16"				
Date Start/Finish: 3/6/2017						Drilling Method: Cased Wash Boring				Core Barrel: NQ2 (2-inch-diameter)				
Boring Location: Sta 2+96.4, 9.5 feet Lt.						Casing ID/OD: HW 4"/4.5"				Water Level*: 11.4 feet bgs				
Definitions:														
D = Spilt Spoon Sample MU = Unsuccessful Thin Wall Tube Sample Attempt WO1P = Weight of 1 Person														
S = Sample off Auger Flights R = Rock Core Sample Su = Peak/Remolded Field Vane Undrained Shear Strength (psf)														
B = Bucket Sample off Auger Flights SSA = Solid Stem Auger Su(lab) = Lab Vane Undrained Shear Strength (psf)														
MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger qp = Unconfined Compressive Strength (ksf)														
U = Thin Wall Tube Sample RC = Roller Cone N-value = Raw Field SPT N-value														
MV = Unsuccessful Field Vane Shear Test Attempt WOH = Weight of 140lb. Hammer Ty = Pocket Torvane Shear Strength (psf)														
V = Field Vane Shear Test PP= Pocket Penetrometer WOR/C = Weight of Rods or Casing WC = Water Content, percent ≈ = Similar or Equal too														
LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test														
Sample Information														
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks					Laboratory Testing Results/AASHTO and Unified Class.
25	R2	60/60	27.80 - 32.80	RQD = 55%			111.30		Formation Rock Mass Quality = Fair R1: Core Times (min:sec) 22.8-23.8 feet (5:06) 23.8-24.8 feet (4:33) 24.8-25.8 feet (4:28) 25.8-26.8 feet (4:20) 26.8-27.8 feet (3:47) R2: Bedrock: Similar to R1; except, vertical fracture (85 to 90 degrees) from 29 to 30.5 feet bgs. Bucksport Formation Rock Mass Quality = Fair R2: Core Times (min:sec) 27.8-28.8 feet (3:46) 28.8-29.8 feet (2:27) 29.8-30.8 feet (2:43) 30.8-31.8 feet (3:04) 31.8-32.8 feet (2:55)  Bottom of Exploration at 32.80 feet below ground surface.					
30														
35														
40														
45														
50														
Remarks: -bgs = below existing ground surface (bgs) -Sampler driven with 140# hammer with 30" drop -Casing driven with 300# hammer with 16" drop -Borehole caved at 12 feet bgs														
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.														
Page 2 of 2														
Boring No.: BB-BWB-102														

## **Appendix B**

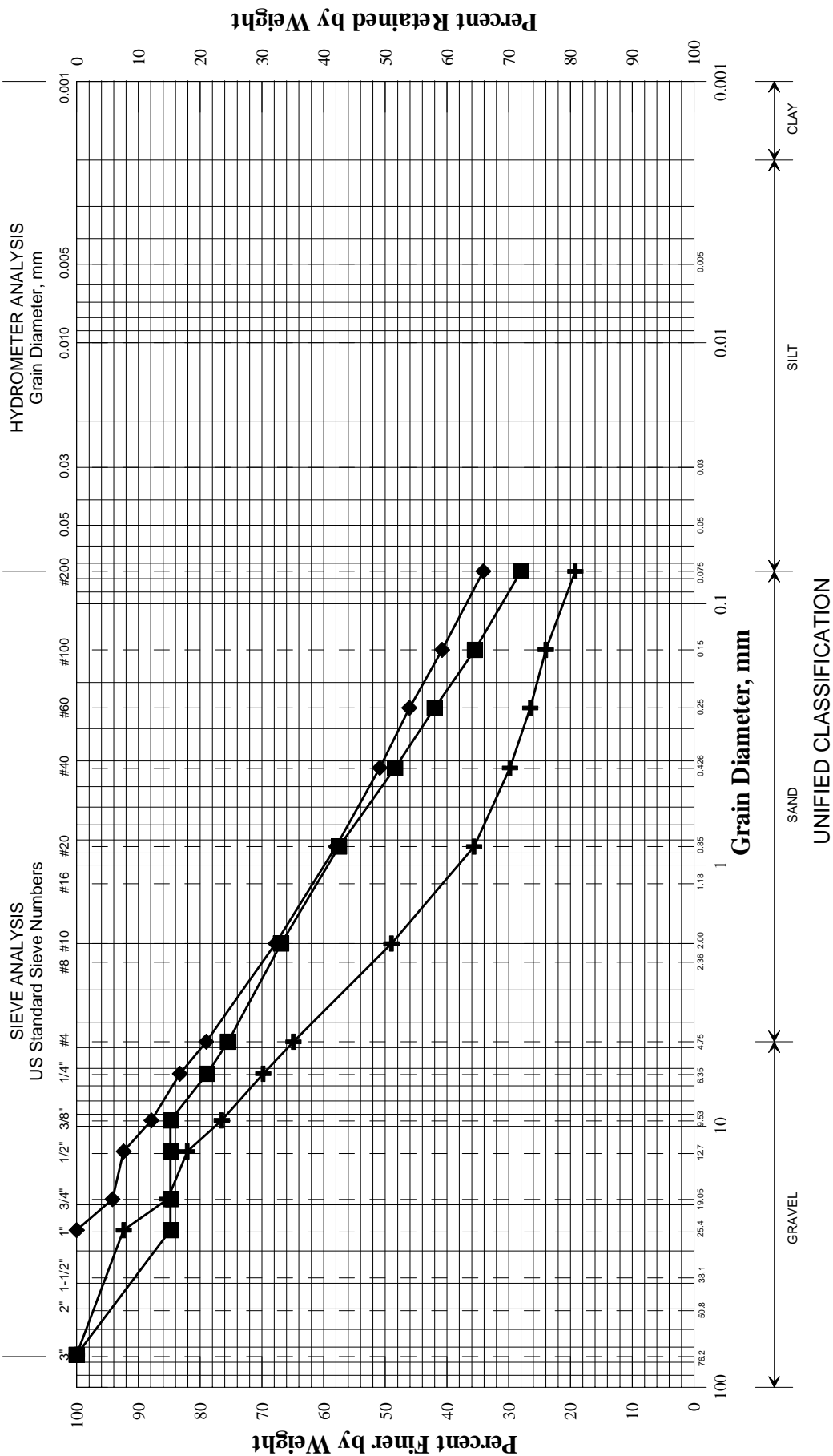
### Laboratory Test Results

**Work Number: 21666.00**

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



State of Maine Department of Transportation  
GRAIN SIZE DISTRIBUTION CURVE



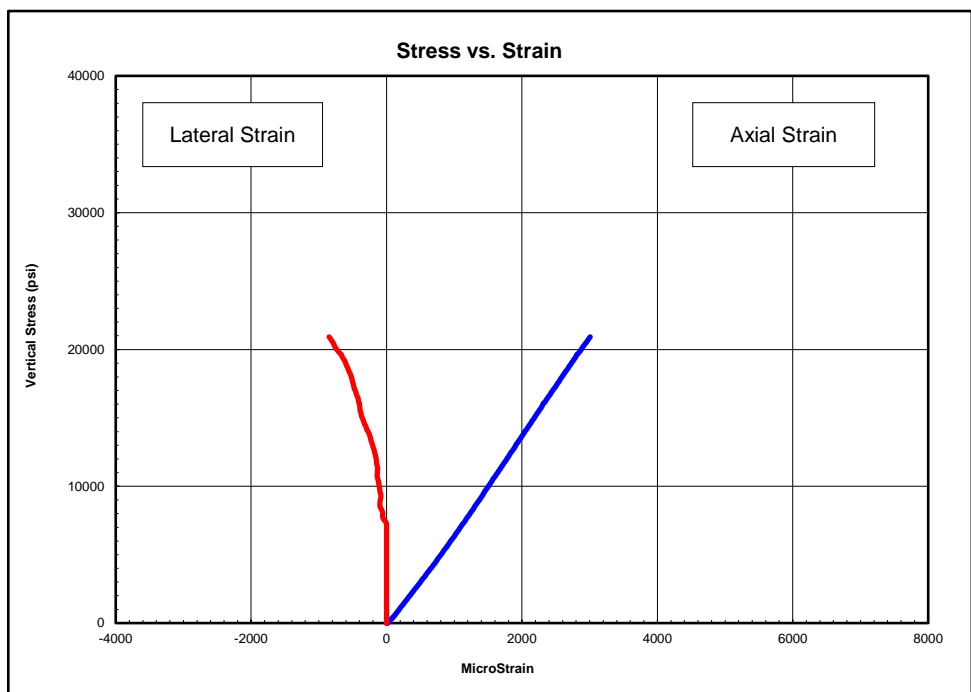
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-BWB-101/3D	2+74.2	9.2 RT	10.0-12.0	SAND, some gravel, little silt.	10.0			
◆	BB-BWB-101/5D	2+74.2	9.2 RT	17.0-19.0	SAND, some silt, some gravel.	10.6			
■	BB-BWB-102/5D	2+96.4	9.5 LT	20.0-22.0	SAND, some silt, some gravel.	9.1			
●									
▲									
×									

WIN	021666.00	Town	Belfast	Reported by/Date	WHITE, TERRY A 4/5/2017
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Client:	Maine DOT
Project Name:	Sheldon Bridge
Project Location:	Belfast, ME
GTX #:	306175
Test Date:	3/24/2017
Tested By:	trm/rlc
Checked By:	jsc
Boring ID:	BB-BWD-102
Sample ID:	R1
Depth, ft:	23.35-23.71
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

## Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D

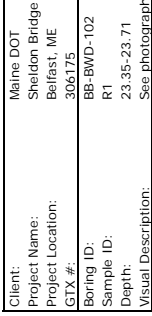


Peak Compressive Stress: 20,981 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

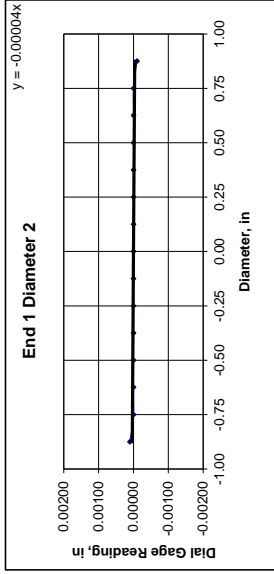
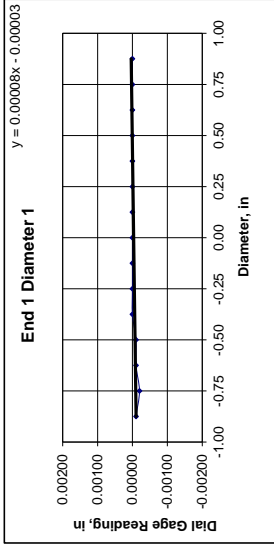
Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2100-7700	6,730,000	0.08
7700-13300	7,370,000	0.20
13300-18800	7,320,000	0.45

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.  
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.  
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.  
Calculations assume samples are isotropic, which is not necessarily the case.



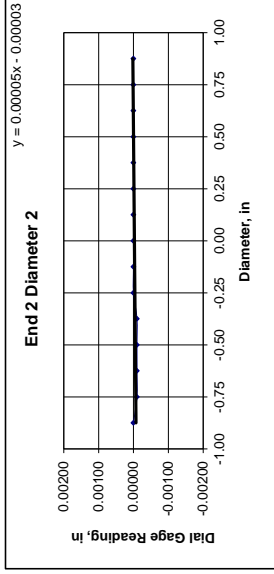
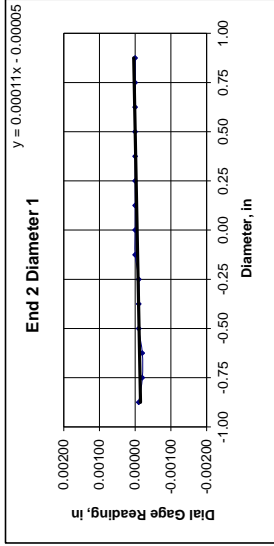
Client:	Maine DOT	Test Date:	3/23/2017
Project Name:	Sheldon Bridge	Tested By:	trm/rjc
Project Location:	Belfast, ME	Checked By:	jsc
GTX #:	306175		
Boring ID:	BB-BWD-102		
Sample ID:	R1		
Depth:	23.35-23.71 ft		
Visual Description:	See photographs		

## UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

[illegible]

DIAMETER 1		
End 1:	Slope of Best Fit Line	0.00089
	Angle of Best Fit Line:	0.00458
End 2:	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00630
	Maximum Angular Difference:	0.00172

**Parallelism Tolerance Met?**  
Spherically Seated **YES**



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00229
End 2:	
Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00286
Maximum Angular Difference:	0.00057

**Parallelism Tolerance Met?**  
Spherically Seated **YES**

<b>PERPENDICULARITY (Procedure P1)</b>					
(Calculated from End Flatness and Parallelism measurements above)					
	Difference, Maximum and Minimum (in.)	Slope Diameter (in.)	Angle <sup>a</sup> Slope	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq 0.25^\circ$
END 1					
Diameter 1, in.	0.00020	1.975	0.0010	YES	
Diameter 2, in (rotated 90°)	0.00020	1.975	0.0010	YES	YES
END 2					
Diameter 1, in.	0.00020	1.975	0.0010	YES	
Diameter 2, in (rotated 90°)	0.00010	1.975	0.00005	YES	

Client:	Maine DOT
Project Name:	Sheldon Bridge
Project Location:	Belfast, ME
GTX #:	306175
Test Date:	3/24/2017
Tested By:	trm/rlc
Checked By:	jsc
Boring ID:	BB-BWD-102
Sample ID:	R1
Depth, ft:	23.35-23.71



After cutting and grinding



After break

## **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 cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

## 6.1 AT-REST LATERAL PRESSURES

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

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

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

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  $\phi'$  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  $\alpha$  is a dimensionless coefficient. Experimental data have confirmed this relationship, and Mayne and Kulhawy (1982) showed that, for most soils,  $\alpha$  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 <sup>a</sup>	OCR = 5 <sup>a</sup>	OCR = 10 <sup>a</sup>
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

<sup>a</sup> 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.

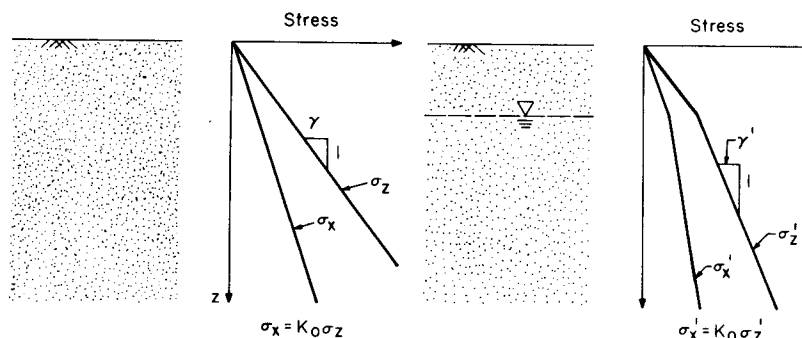
Because the  $K_0$  value in a given soil often varies with depth, and the soil types themselves may change with depth, the at-rest lateral pressure distribution is typically not linear as shown in Figure 6.2. Self-boring pressuremeter tests in clays with overconsolidated profiles induced by desiccation 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  $\beta$  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:

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

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

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

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

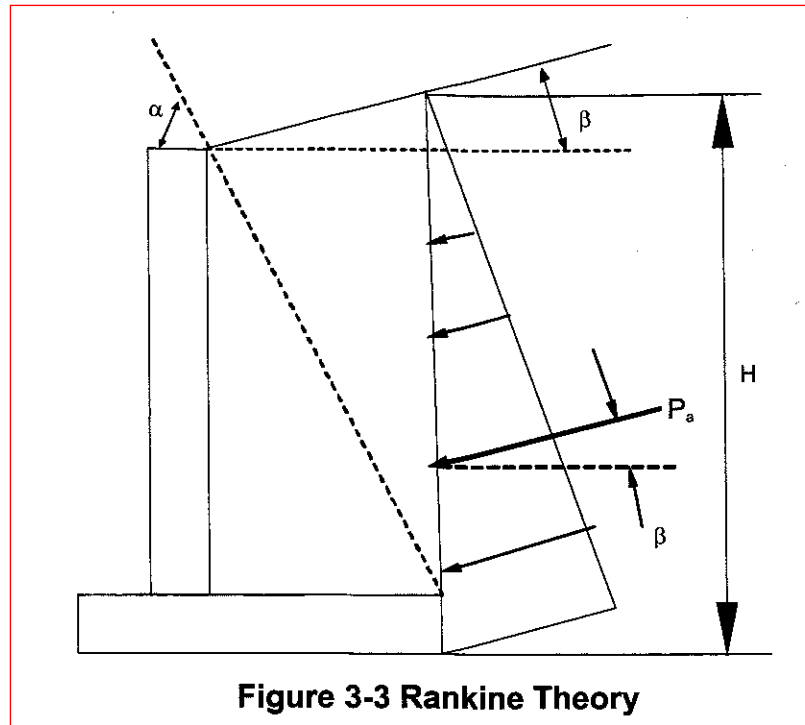


Figure 3-3 Rankine Theory

The resultant earth pressure force,  $P_a$ , is oriented at an angle,  $\beta$ , 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  $\beta$  value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with  $\beta^*$ , 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:

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

$\phi$  = 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 130 feet.
4. Proposed finish roadway grade elevation is approximately 143.4 feet.
5. Proposed precast concrete box base is 23' 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 30 bpf for the consistency of the soils encountered at the box bearing elevation, based on BB-BWP-101 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
Fine to medium sand, silty or clayey medium to coarse sand	Medium dense to dense	4-8	5

***The lower fill unit is medium dense to dense in consistency. Recommend 5 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

$$B := 23\text{ft}$$

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

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

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

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

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

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

Foundation soils:

Foundation soils based on BB-BWB-101 5D

$$\gamma_{1d} := 121 \cdot \text{pcf}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:  
Table 3.2 Dry, dense angular sand - dry unit weight

$$w_{\text{sat}} := .091$$

Moisture content of BB-BWB-102 5D.

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

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

$$\gamma_{1\text{sat}} = 132 \cdot \text{pcf}$$

$$N_{\text{design}} := 30$$

$$\phi := 36 \cdot \text{deg}$$

Kulhawy and Mayne, Manual on Estimating Soil Properties p.  
4-15:

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

Table 4-3 & Figure N vs. phi

### Nominal Bearing Resistance for Strength Limit States

Reference: Munfakh, et al (2001) LRFD Article 10.6.3.1.2a

Bearing Capacity Factors (Ref: LRFD Table 10.6.3.1.2a-1)

$$N_c := 50.6$$

$$N_q := 37.8$$

$$N_\gamma := 56.3$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

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.

$$C_{wq} := .5 \quad C_{w\gamma} := 0.5 \quad c_1 := 0$$

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

$$\frac{D_f}{B} = 0$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 69.5$$

$$N_{\gamma m} = 45$$

$$N_{qm} = 51.5$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[ c_1 \cdot N_{cm} + \gamma_{above} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{sat} \cdot \overrightarrow{(B \cdot N_{\gamma m})} \cdot C_{w\gamma} \right]$$

$$q_n = 37.4 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 16.8 \cdot \text{ksf}$$

**Recommend a limiting value for the factored bearing resistance of 16.8 ksf or 8.4 tsf, for box bottom slabs 23 ft or greater on compacted granular fill.**

### 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, $\phi$	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$ , Concrete to Soil	Interface Friction, Angle, Concrete to Soil $\delta$
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 ( $\phi$ ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

### 3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for  $\gamma$ ,  $\gamma_d$ , and  $\gamma_{\text{sat}}$  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  $\gamma$ ,  $\gamma_d$ , and  $\gamma_{\text{sat}}$

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

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



Table 4-3

N VERSUS  $\bar{\phi}_{tc}$  RELATIONSHIPS

N Value (blows/ft or 305 mm)	Relative Density	Approximate $\bar{\phi}_{tc}$ (degrees)	
		(a)	(b)
0 to 4	very loose	< 28	< 30
4 to 10	loose	28 to 30	30 to 35
10 to 30	medium	30 to 36	35 to 40
30 to 50	dense	36 to 41	40 to 45
> 50	very dense	> 41	> 45

a - Source: Peck, Hanson, and Thornburn (12), p. 310.

b - Source: Meyerhof (13), p. 17.

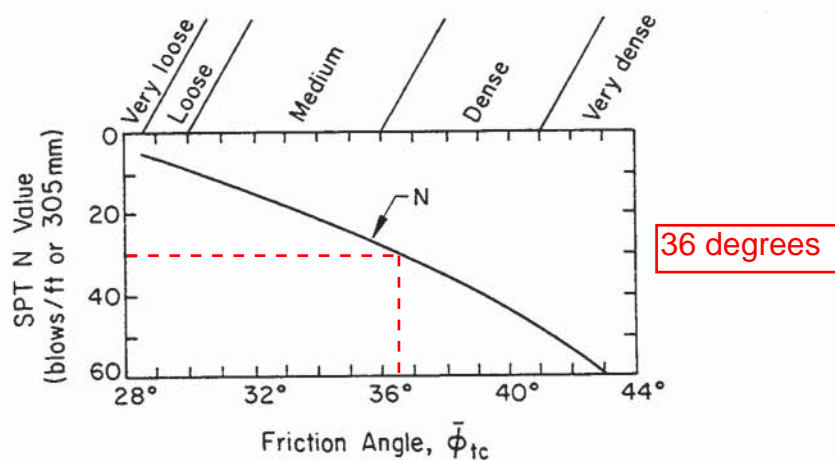


Figure 4-12. N versus  $\bar{\phi}_{tc}$

Source: Peck, Hanson, and Thornburn (12), p. 310.

can be approximated as follows:

$$\bar{\phi}_{tc} \approx \tan^{-1} [0.1 + 0.38 \log (q_c/\bar{\sigma}_{vo})] \quad (4-12)$$

Adjustments to this figure and equation for soils of different compressibility and stress history should be made as described in Section 2.

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

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

where:

$B$  = footing width (ft)

$L$  = footing length (ft)

$H$  = unfactored horizontal load (kips)

$V$  = unfactored vertical load (kips)

$\theta$  = 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  $\theta$  angle in Eq. 10.6.3.1.2a-9.

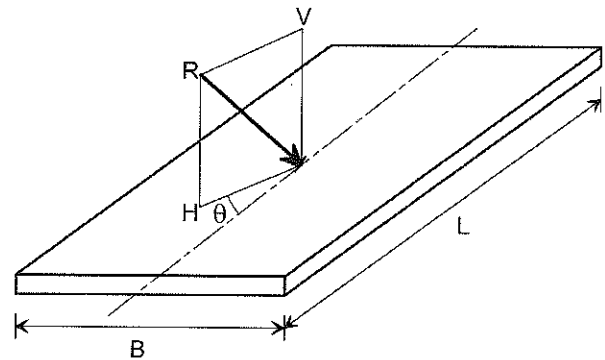


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

$\phi_f$	$N_c$	$N_q$	$N_\gamma$	$\phi_f$	$N_c$	$N_q$	$N_\gamma$
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

Modulus of Subgrade Reaction

**Objective:**

Estimate the modulus of subgrade reaction for the box culvert design

**Given:**

1. Limited lab data

**Assumptions:**

1. The proposed bearing elevation is approximately 130 feet.
2. Proposed finish roadway grade elevation is approximately 143.4 feet.
3. Proposed precast concrete box is 23.0 feet wide and 63.0 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, medium dense, sand:

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

Range of modulus of subgrade reaction

Medium dense sand:  $k_s = 35 - 295$  pci

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

Range of modulus of subgrade reaction

Medium dense submerged coarse-grained soils:  $K_{v1}$ , 75 - 133 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 medium dense sand:  $k_{0.3}$  ( $k_1$ ) = 129-147 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:  $k_{s1} = 92$  pci

Many of the published ranges are wide or are unconservative for use in design. Use Terzaghi's recommended value,  $k_{s1} = 92$  pci for a 1 ft x 1 ft plate and adjust to the dimensions of the box culvert base. (Width B = 23 ft, Length L = 63 ft)

Square to rectangle base adjustment:

$$k_{s1} := 133 \text{ pci} \quad B := 23.0 \text{ ft} \quad L := 63.0 \text{ ft}$$

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

$$k = 105 \cdot \text{pci}$$

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 equa-

the  $d_i$  factors dropped) to give

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

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

to estimate  $k_s$ . In these equations the Terzaghi or Hansen bearing-

ed. The  $C$  factor is 40 for SI units and 12 for Fps, using the same

at a 0.0254-m and 1-in. settlement but with no SF, since this equa-

here there is concern that  $k_s$  does not increase without bound with

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

h of interest

imate of the exponent

to estimate a value of  $k_s$  to determine the correct order of magnitude

obtained using one of the approximations given here. Obviously if a

three times larger than the table range indicates, the computations

possible gross error. Note, however, if you use a reduced value of

or 12 mm) instead of 0.0254 m you may well exceed the table range.

computational error (or a poor assumption) is found then use judgment

table values are intended as guides. The reader should not use, say,

ven as a "good" estimate.

l in Fig. 9-9c (and used in your diskette program FADBEMLP as

estimated at some small value of, say, 6 to 25 mm, or from inspection

if a load test was done. It might also be estimated from a triaxial

ultimate" or at the maximum pressure from the stress-strain plot.

$\bar{\epsilon}_{max}$  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

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

Soil	$k_s$ , kN/m <sup>3</sup>	$k_s$ , lb/in <sup>3</sup>
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:

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

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

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

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

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

For center:

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

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

From these we may write

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

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

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

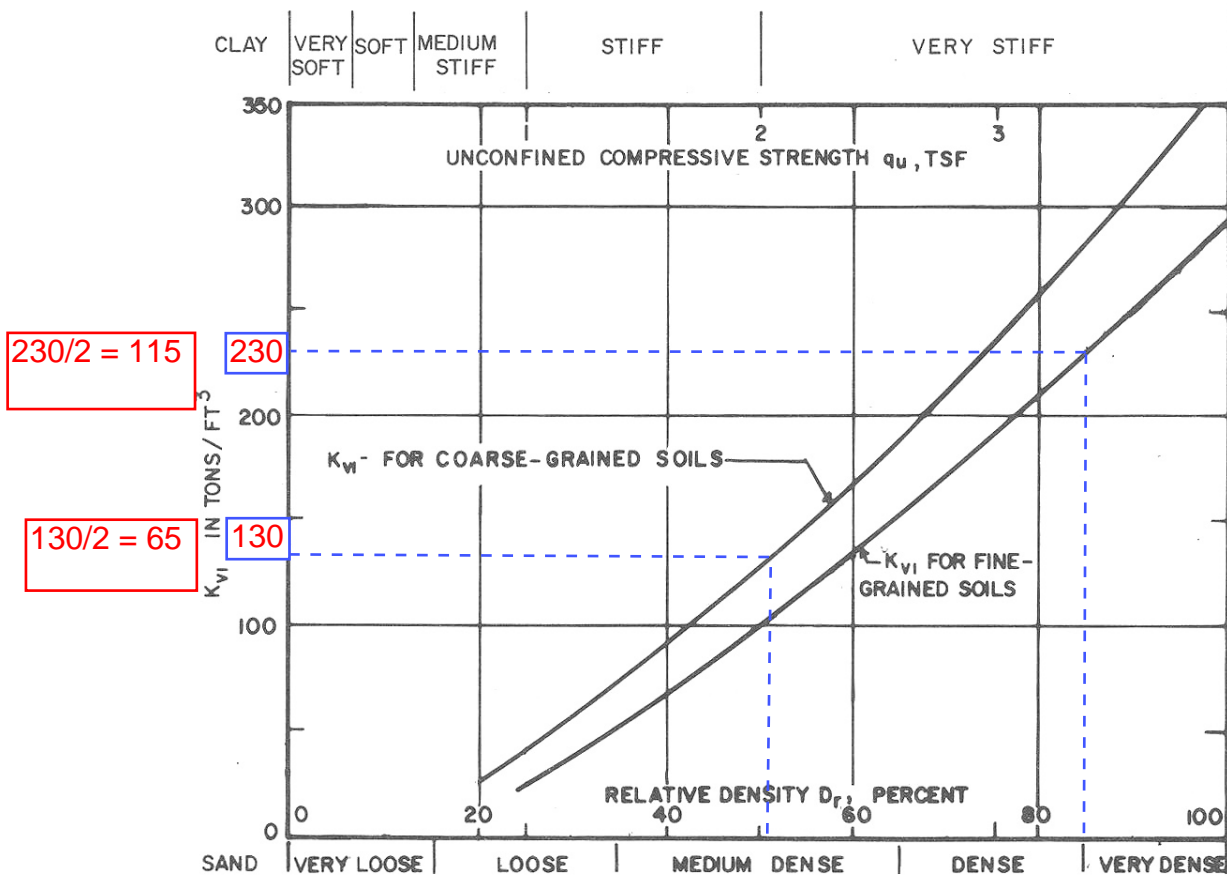
$$k_s = \frac{1}{0.61(0.07765)(4 \times 0.597)(0.8)} = 11.05 \text{ MN/m}^3$$

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

For corner:

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

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



### DEFINITIONS

$\Delta H_i$  = 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

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

$$115 * 1.15 = 133 \text{ pci}$$

$$65 * 1.15 = 75 \text{ pci}$$

### 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
- $\mu_s$  = Poisson's ratio of soil

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

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

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

the 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

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

$$80 \text{ ton/cu. ft} \cdot 1.15 = 92 \text{ pci}$$

problem can be cor headings. Experien sand is roughly equ (Fig. 3) or for a mat equation (8) :

If applied to sp contact pressures su unit of area of the l porting concentrate half of the ultimat equation (9).

V:  
f:

Values  
Range  
Propos

For rec

\* High

If the subgrade ately in simple pr basis of our presen numerical values o pressures which ar The latter is indep

The proposed v medium sand, Tab of the loaded area normally consolida beams and rafts sl perfectly rigid.

The  $\bar{k}_{s1}$  values of the tests can be of such tests is to the test results can of the block shoul

If the contact the value :

For  $l = \infty$ ,  $k_{s1} =$  loaded subgrade 1



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

### **Foundations on Sandy Soils**

For foundations on sandy soils,

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

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

### **Foundations on Clays**

For foundations on clays,

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

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

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

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

Method 1:

where

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

Frost

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

$$DFI_3 := 1450$$

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

Depth of Frost Penetration

$$d_3 = 6.7 \cdot \text{ft}$$

**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).

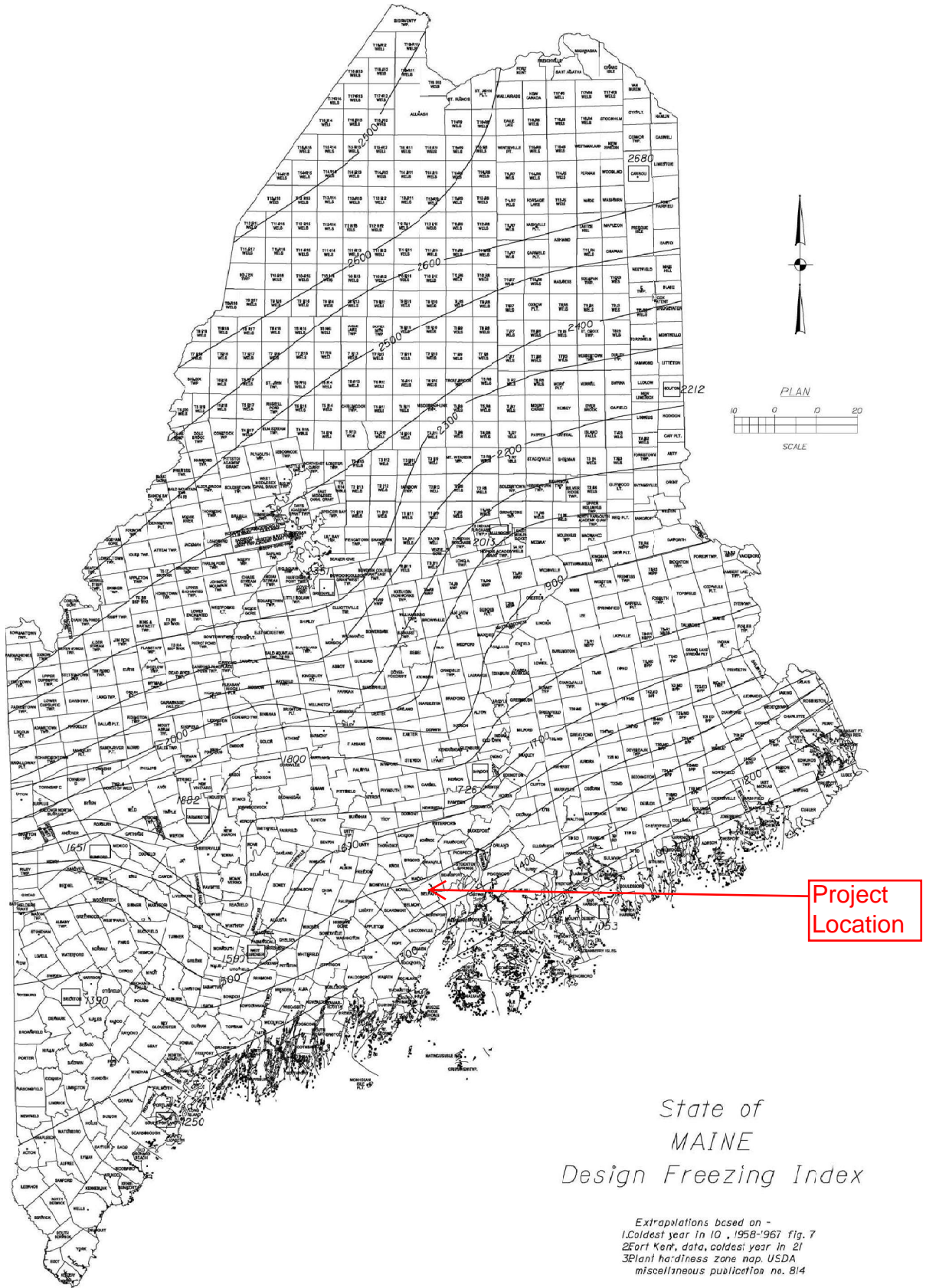
L = Latent heat of fusion, in BTU / cubic ft.

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

Recommendation: 6.7 feet for design of foundations constructed on coarse grained soils
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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,  
Design Freezing  
Index = 1450  
Frost Penetration = 80.7"