

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

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**GOWER FARM BRIDGE
U.S. ROUTE 2 & STATE ROUTE 27 OVER BEALES BROOK
FARMINGTON, MAINE**

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Franklin County
WIN 21880.00

Soils Report 2019-27
Bridge No. 2324

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Table of Contents

1.0	INTRODUCTION.....	1
2.0	GEOLOGIC SETTING	1
3.0	SUBSURFACE INVESTIGATION	2
4.0	LABORATORY TESTING	2
5.0	SUBSURFACE CONDITIONS	2
5.1	GRANULAR FILL.....	3
5.2	FINE GRAINED FILL	3
5.3	STREAM ALLUVIUM	3
5.4	GLACIOMARINE DEPOSIT	3
5.5	GLACIAL TILL	4
5.6	BEDROCK	4
5.7	GROUNDWATER	5
6.0	STRUCTURE ALTERNATIVES	5
7.0	GEOTECHNICAL DESIGN RECOMMENDATIONS	5
7.1	PRECAST CONCRETE BOX CULVERT DESIGN	5
7.1.1	PRECAST CONCRETE BOX CULVERT HEADWALLS.....	6
7.1.2	PRECAST CONCRETE INLET AND OUTLET WALLS.....	6
7.1.3	PRECAST CONCRETE TOE WALLS.....	6
7.1.4	BEARING RESISTANCE	6
7.1.5	MODULUS OF SUBGRADE REACTION	7
7.2	SETTLEMENT	7
7.3	SUBGRADE PREPARATION	7
7.4	FROST PROTECTION.....	8
7.5	SCOUR AND RIPRAP	8
7.6	SEISMIC DESIGN CONSIDERATIONS	8
7.7	CONSTRUCTION CONSIDERATIONS	9
8.0	CLOSURE	10

Tables

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

Sheets

Sheet 1 – Location Map

Sheet 2 – Boring Location Plan

Sheet 3 – Interpretive Subsurface Profile

Sheet 4 – Boring Logs

Appendices

Appendix A – Boring Logs

Appendix B – Laboratory Test Results

Appendix C – Calculations

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Gower Farm Bridge, which carries U.S. Route 2 and State Route 27 over Beales Brook in Farmington, 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 1958 and consists of a corrugated steel pipe with a 13-foot span and a 13-foot, 10-inch rise on a 30-degree skew. According to the 2018 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culvert is rated 4 and is considered to be in poor but serviceable condition. The pipe is deformed and the bottom and sides are corroded with holes. The structure has a NBI Sufficiency Rating of 46.6.

The proposed replacement structure will be a 20-foot span by 12-foot rise precast concrete box culvert on a 26-degree skew. The concrete box culvert will have 1-foot tall precast headwalls and 2-foot deep toe walls. 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 into the streambed and 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 layer of Granular Borrow – Material for Underwater Backfill, underlain by geotextile and bearing on compacted native soils.

Gower Farm Bridge will be replaced using staged construction. Farmington Falls Road (U.S. Route 2) will be kept open to traffic at the project site by means of staged construction and alternating one-way traffic. The bridge replacement project will last one construction season.

2.0 GEOLOGIC SETTING

Gower Farm Bridge carries U.S. Route 2 and State Route 27 over Beales Brook approximately 0.14 miles west of Dump Road, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology of the Farmington Quadrangle, Open-File Map No. 86-29 (1986), indicates the surficial soil units in the vicinity of the bridge are glacial stream deposits (sand and gravel) and alluvial deposits (sand, gravel, and silt). Glacial stream deposits were deposited by meltwater streams during the melting of the Late Wisconsinan glacier. At this site, the soils were deposited at the ice margin, and therefore may include floodplain and delta outwash units. Glacial stream deposits are commonly overlapped or buried by glaciomarine deposits (clay, silt and sand) and may include minor glacial till, a heterogeneous mixture of sand, silt, clay and stones.

The MGS Reconnaissance Bedrock Geology of the Farmington Quadrangle, Open-File Map No. 78-16 (1978), cites the bedrock at the project site as Sangerville Formation, consisting of metasandstone (greywacke), metasilstone, and metapelite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two borings in the roadway. Boring BB-FBB-101 was drilled outside the southwest corner of the existing steel pipe and boring BB-FBB-102 was drilled outside the northeast corner. The boring locations are shown on Sheet 2 – Boring Location Plan.

Borings BB-FBB-101 and BB-FBB-102 were drilled between February 12 and 14, 2018 by Northern Test Boring (NTB) of Gorham, 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 NTB 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 June 2017 prior to the borings. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.977 for both borings. The hammer efficiency factor (0.977) 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 MaineDOT 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 borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of six standard grain size analyses with natural water contents, two standard grain size analyses with hydrometers 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 fill soils underlain, in order of increasing depth, by stream alluvium, glaciomarine deposits, glacial till and 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 Granular Fill

A layer of fill was encountered in both borings. The thickness of the fill unit encountered was approximately 16 to 17 feet at the boring locations. The unit encountered generally consisted of:

- Brown to brown-grey, dry to moist, fine to medium sand, trace to some silt, trace coarse sand, trace gravel.

All SPT N_{60} -values in the layer were greater than 50 blows per foot (bpf), indicating the layer is very dense in consistency. Three SPT's failed to advance the minimum eighteen inches required to record an N-value indicating the presence of cobbles. Grain size analyses conducted on samples of the fill indicated the soil is classified as A-2-4 and A-3 under the AASHTO Soil Classification System and SM and SP-SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested were approximately 5 and 14 percent.

5.2 Fine Grained Fill

A layer of fine grained fill was encountered beneath the granular fill in boring BB-FBB-101. The thickness of the fine grained fill unit encountered was approximately 2 feet at the boring location. The fine grained fill encountered generally consisted of: grey, wet, fine sandy silt.

A SPT N_{60} -value in the cohesive fill was 15 bpf indicating the soil was stiff in consistency.

5.3 Stream Alluvium

A deposit of stream alluvium was encountered beneath the fill unit in the borings. The thickness of the alluvial deposit encountered was approximately 16 to 17 feet at the boring locations. The deposit encountered generally consisted of:

- Grey to grey-brown, wet, fine to medium sand, trace coarse sand, trace to little silt, trace gravel, wood pieces;
- Grey, wet, sand, trace to little gravel, trace silt.

SPT N_{60} -values in the layer ranged from 5 to 20 bpf, indicating the layer is loose to medium dense in consistency. Grain size analyses conducted on samples of the marine sands indicated the soil is classified as A-1-b, A-2-4 and A-3 under the AASHTO Soil Classification System and SW-SM, SM, SP-SM, and SP under the USCS. The natural water contents of the samples tested ranged from approximately 13 to 28 percent.

5.4 Glaciomarine Deposit

A glaciomarine deposit was encountered beneath the alluvial sands in the borings. The thickness of the glaciomarine deposit encountered was approximately 10 to 11 feet at the boring locations. The glaciomarine deposit encountered generally consisted of:

- Grey, wet, silt, little to some clay, trace to little fine sand;
- Grey, wet, silt, little gravel.

SPT N₆₀-values in the layer ranged from 5 to 44 bpf, indicating the layer is medium stiff to hard in consistency. Grain size analyses conducted on samples of the glaciomarine deposit indicated the soil is classified as A-4 under the AASHTO Soil Classification System and CL under the USCS. The natural water contents of the samples tested ranged from approximately 33 to 35 percent. Two Atterberg limits tests indicated the samples were non-plastic.

5.5 Glacial Till

A deposit of glacial till was encountered beneath the glaciomarine deposit in the borings. The thickness of the glacial till encountered was approximately 17 feet in boring BB-FBB-101. The glacial till deposit generally consisted of:

- Grey, wet, gravelly sand, little silt;
- Grey, moist to wet, sand, some gravel, little silt;
- Cobbles;
- Boulders.

SPT N₆₀-values in the layer ranged from 21 to greater than 50 bpf, indicating the layer is medium dense to very dense in consistency. Roller coring and rock coring techniques were required to advance the test boring through the glacial till deposit in BB-FBB-102. Samples recovered by rock core tooling indicated the presence of cobbles and boulders.

5.6 Bedrock

Bedrock was encountered and cored in boring BB-FBB-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 (%)
BB-FBB-102	5+2.8	12.3 Lt	60.5	285.9	88

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

The bedrock at the site is identified as grey, fine grained, metasiltstone and metasandstone, hard, fresh, horizontal to moderately dipping joint set, close to moderately close spacing, tight joints, with quartz veins. The RQD of the bedrock was 88 percent corresponding to a rock mass quality of good. Detailed bedrock descriptions and the RQD core run are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.7 Groundwater

Groundwater was not observed in the boreholes. 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 20-foot span precast concrete box;
- A 53-foot single span integral abutment bridge.

A 20-foot span precast concrete box culvert has the advantages of efficiency of installation, low maintenance and low lifecycle costs. The precast concrete box also allows for staged construction and maintenance of one-lane of traffic during construction. The precast concrete box culvert was chosen as the preferred alternative.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 20-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 into the streambed and engineered streambed material will be placed inside the culvert to create a natural streambed. The riprap apron should be embedded 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, 8th Edition, 2017 (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 forces resulting from creep, temperature, and shrinkage deformations of the concrete box culvert. Passive pressure forces 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 constructed on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill. Prior to placing and compacting the bedding material we recommend hand-deploying a heavy, non-woven stabilization/reinforcement geotextile on the native soil subgrade. The bearing elevation of the Granular Borrow bedding will be approximately Elevation 325. The subgrade soils at this elevation are expected to be loose to medium dense, saturated alluvial sands. It is anticipated that the precast concrete box culvert will be adequately supported by the sand deposit overlying the glaciomarine silt deposit.

For a precast concrete box culvert with a base width of 22 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 4.4 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 3.0 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. 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 35 pounds per cubic inch (pci).

7.2 Settlement

The alluvial sands encountered at the bearing elevation are loose to medium 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 Excavation and Subgrade Preparation

The alluvial sands below the groundwater level are saturated and loose and will easily become disturbed by construction activities. The excavation will require care to maintain bottom stability and bearing capacity. The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to allow construction of the culvert bedding in the dry;
- Use a smooth-edged bucket to avoid disturbance of the subgrade;
- Careful grade control will be necessary to avoid over excavation;
- Construct the box on a 1-foot thick bed of Granular Borrow underlain by a heavy non-woven stabilization/reinforcement geotextile;
- Hand-deploy the geotextile on the prepared soil subgrade prior to placing and compacting the Granular Borrow culvert bedding.

The culvert bedding material shall meet the requirement of Granular Borrow for Underwater Backfill. The bedding material shall be placed in lifts of 6 to 8 inches thick loose measure and compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

Unsuitable soils (i.e. loose or soft soils) if encountered at the subgrade elevation, 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, Farmington has an air design freezing index (DFI) of approximately 1870 F-degree days. A water content of 20% was used for coarse grained soils. These components correlate to a frost depth of 6.3 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Farmington, Maine has an air DFI from the Modberg database of approximately 2023 F-degree days. A water content of 20% was used. These components correlate to a frost depth of approximately 6.1 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on soil be designed with an embedment of approximately 6.3 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. Riprap slopes 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. Riprap slopes shall be constructed no steeper than a maximum 2H:1V extending from the edge of the roadway down to the existing ground surface.

The 2014 Inspection Report notes major damage to the stream banks. The stream is very erodible and there is extensive head-cutting downriver. Streambed reconstruction and sideslope armoring with riprap may be required.

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 shall be constructed on a 1-foot thick layer of bedding material meeting the requirements of Standard Specification Section 703.19, Granular Borrow for Underwater Backfill. The culvert bedding material will be placed on Class 1 Stabilization/Reinforcement Geotextile meeting Standard Specification 722.01. The geotextile should be hand-deployed on the prepared soil subgrade prior to installing the culvert bedding. Adjoining sections of the stabilization geotextile should be overlapped by a minimum of 1 foot.

The fine to medium sand (Alluvium) at the box culvert subgrade elevation is below the groundwater table, is saturated and loose, and will easily become disturbed by construction activities. The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering to limit disturbance of the sand subgrade and limit the risk of excavation bottom heave, is recommended to allow the bearing pad construction in the dry. Cofferdams may be required to divert flow away from the new culvert location during construction.
- The contractor shall protect the soil subgrade from unnecessary construction traffic to protect the excavation subgrade from disturbance and rutting.
- Use a smooth-edged bucket to avoid disturbance of the subgrade.
- Careful grade control will be necessary to avoid over excavation.
- The stabilization/reinforcement geotextile shall be hand-deployed on the prepared soil subgrade prior to installing the culvert bedding material.

The saturated loose sands have potential to “flow” when disturbed. It is possible there may be subsidence associated with the withdrawal of the temporary sheeting and shoring required for staged construction. This may require final shimming and repair of each phase of the roadway.

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.

Earthwork and excavations will expose loose or soft soils. These soils are susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance or rutting occur, the Contractor shall remove and replace the materials with compacted granular borrow or crushed stone.

Saturated soils and water seepage will 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.

Wood was encountered in boring BB-FBB-101 at the approximate elevation of the proposed box culvert base. Wood or timber cribbing may impede construction efforts and the contractor should assume excavation and removal is necessary.

The abutments of a pre-existing bridge, built prior to the existing corrugated steel pipe, are likely buried in the eastern approach to the bridge. Implications of the remnants of previous bridge structures buried in the eastern approach include difficulty installing and damage to temporary lateral earth support systems. The previous structure may impede construction efforts and the contractor should assume excavation and removal is necessary.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT for specific application to the proposed replacement of Gower Farm Bridge in Farmington, 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



FARMINGTON, MAINE

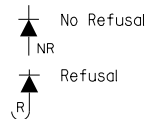
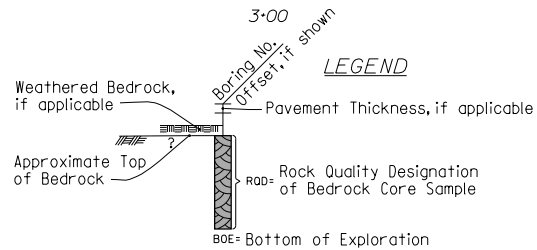
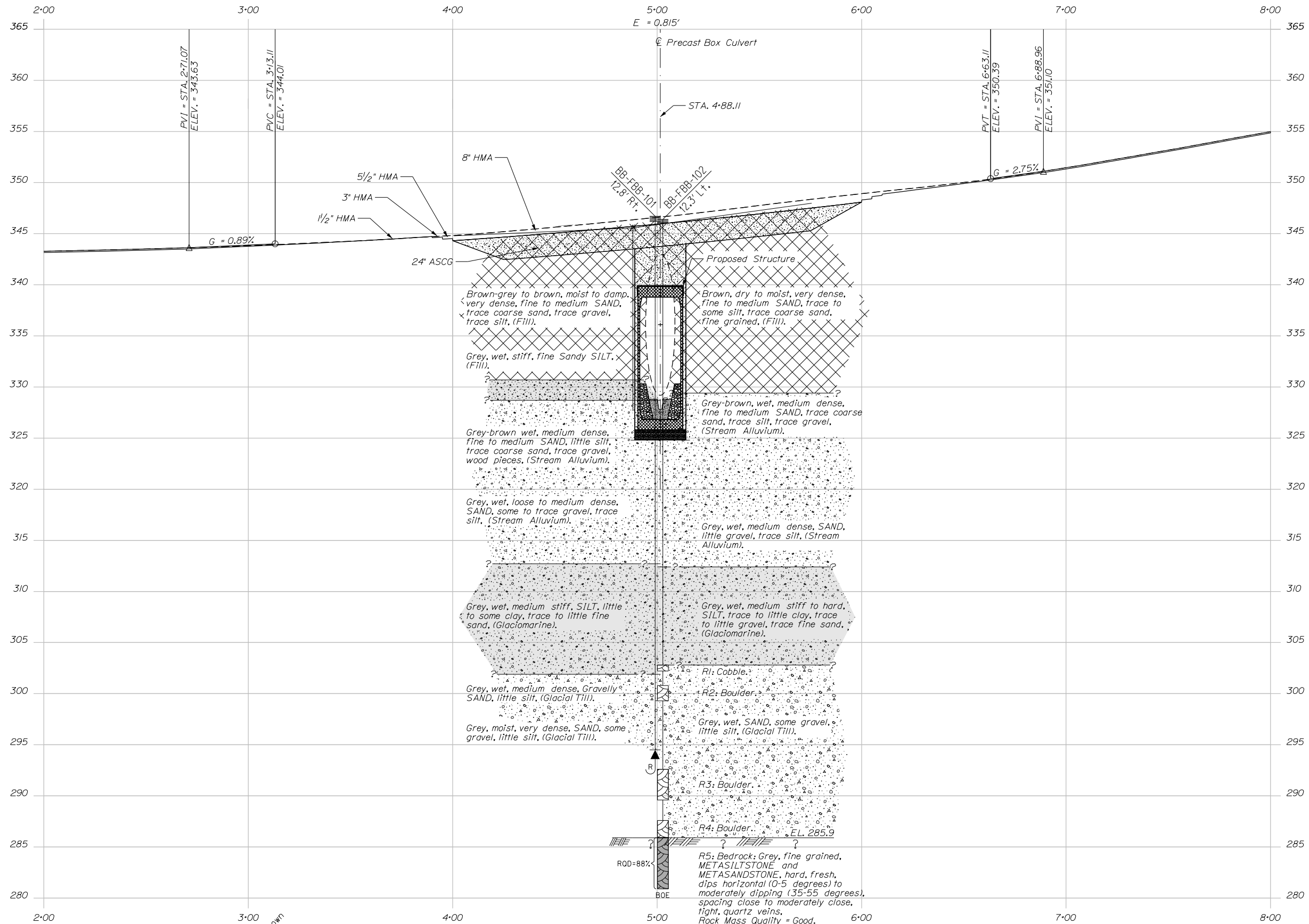


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0.25 Miles
1 inch = 0.28 miles

Date: 1/14/2019
Time: 9:45:48 AM

SHEET NUMBER 1 OF 4	GOWER FARM BRIDGE BEALES BROOK FARMINGTON FRANKLIN COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
		NHPP-2188(000)	
	LOCATION MAP	WIN 21880.00 BRIDGE NO. 2324 BRIDGE PLANS	



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

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
GOWER FARM BRIDGE		NHP-2188(000)	
BEALES BROOK		WIN	
FRANKLIN COUNTY		BRIDGE NO. 2324	
FARMINGTON		21880.00	
INTERPRETIVE SUBSURFACE PROFILE		BRIDGE PLANS	
SHEET NUMBER		3	
		OF 4	

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM			
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES				
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10		
					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)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	TERMS DESCRIBING DENSITY/CONSISTENCY			
		(little or no fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.				
		SANDS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	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).			
			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)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	Density of Cohesionless Soils	
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Very loose			0 - 4			
OL	Organic silts and organic silty clays of low plasticity.	Loose			5 - 10			
		Medium Dense			11 - 30			
		Dense			31 - 50			
SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.		Very Dense	> 50			
	CH	Inorganic clays of high plasticity, fat clays.		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.				
	OH	Organic clays of medium to high plasticity, organic silts.						
	HIGHLY ORGANIC SOILS	Pt		Peat and other highly organic soils.	Approximate Undrained Shear Strength (psf)		Field Guidelines	
					Consistency of Cohesive soils	SPT N-Value (blows per foot)		
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					Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *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	
					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))			
					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 Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information								

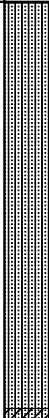
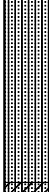
Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Gower Farm Bridge #2324 carries Routes 2/27 over Beales Brook Location: Farmington, Maine				Boring No.: BB-FBB-101 WIN: 21880.00			
Driller: Northern Test Boring				Elevation (ft.): 346.7				Auger ID/OD: 2.25/6.25"			
Operator: Nadeau/Taylor				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: A. Van Buskirk				Rig Type: Dietrich D-50 Track				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 2/12/2018; 08:30-14:30				Drilling Method: Cased Wash Boring				Core Barrel: N/A			
Boring Location: 4+99, 12.8 ft Rt.				Casing ID/OD: HW-4"				Water Level*: None Observed			
Hammer Efficiency Factor: 0.977				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample 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							HSA	346.2		6" HMA.	G#271126 A-3, SP-SM WC=5.3%
	1D	3/3	2.00 - 2.25	50(3")	---					Brown-grey, fine to medium SAND, trace gravel, trace silt, (Fill). Blocked by coarse piece of gravel.	
5	2D	24/22	5.00 - 7.00	25/24/20/16	44	72				Brown, moist, very dense, fine to medium SAND, trace silt, (Fill).	
10	3D	24/19	10.00 - 12.00	12/19/19/24	38	62				Brown, damp, very dense, fine to medium SAND, little silt, trace coarse sand, trace gravel, (Fill).	
15	4D/A	24/22	15.00 - 17.00	7/5/4/4	9	15	23	330.7		4D (15.0-16.0 ft bgs) Similar to 3D, except medium dense, (Fill).	
							40			4D/A (16.0-17.0 ft bgs) Grey-brown, wet, stiff, fine Sandy SILT, (Fill).	
							63				
							50				
20	5D	24/7	20.00 - 22.00	6/6/6/6	12	20	58	328.7		Grey-brown, wet, medium dense, fine to medium SAND, little silt, trace gravel, trace coarse sand, wood pieces, (Stream Alluvium).	G#271127 A-2-4, SM WC=28.1%
							62				
							70				
							65				
25							42				
Remarks: Automatic Hammer SN# 377											
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-FBB-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Gower Farm Bridge #2324 carries Routes 2/27 over Beales Brook Location: Farmington, Maine				Boring No.: BB-FBB-101 WIN: 21880.00																																																																																																																																																																																																																											
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<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td rowspan="5">25</td><td>6D</td><td>24/4</td><td>25.00 - 27.00</td><td>2/2/1/1</td><td>3</td><td>5</td><td>45</td><td rowspan="15">312.7</td><td rowspan="15"></td><td rowspan="5">Grey, wet, loose, SAND, trace gravel, (Stream Alluvium).</td><td rowspan="15">G#271128 A-1-b, SW-SM WC=12.5%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>51</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>64</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>77</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>57</td></tr><tr><td rowspan="5">30</td><td>7D</td><td>24/6</td><td>30.00 - 32.00</td><td>4/5/3/5</td><td>8</td><td>13</td><td>62</td><td rowspan="5">34.0</td><td rowspan="5">Grey, medium dense, SAND, some gravel, trace silt, (Stream Alluvium).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>64</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>78</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>79</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>64</td></tr><tr><td rowspan="5">35</td><td>8D</td><td>24/16</td><td>35.00 - 37.00</td><td>2/2/3/3</td><td>5</td><td>8</td><td>80</td><td rowspan="5">301.9</td><td rowspan="5">44.8</td><td rowspan="5">Grey, wet, medium stiff, SILT, some clay, little fine sand, (Glaciomarine).</td><td rowspan="5">G#271129 A-4, CL WC=35.2% Non-Plastic</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>76</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>73</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>62</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>HYD PUSH</td></tr><tr><td rowspan="5">40</td><td>9D</td><td>24/18</td><td>40.00 - 42.00</td><td>1/1/2/2</td><td>3</td><td>5</td><td></td><td rowspan="5">301.9</td><td rowspan="5">44.8</td><td rowspan="5">Grey, wet, medium stiff, SILT, little clay, trace fine sand, (Glaciomarine).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="5">45</td><td>10D</td><td>24/5</td><td>45.00 - 47.00</td><td>6/6/7/4</td><td>13</td><td>21</td><td>130</td><td rowspan="5">301.9</td><td rowspan="5">44.8</td><td rowspan="5">Grey, wet, medium dense, Gravelly SAND, little silt, (Glacial Till).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>143</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>120</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>149</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td>---</td></tr></table>												Depth (ft.)	Sample Information								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	Elevation (ft.)	25	6D	24/4	25.00 - 27.00	2/2/1/1	3	5	45	312.7		Grey, wet, loose, SAND, trace gravel, (Stream Alluvium).	G#271128 A-1-b, SW-SM WC=12.5%							51							64							77							57	30	7D	24/6	30.00 - 32.00	4/5/3/5	8	13	62	34.0	Grey, medium dense, SAND, some gravel, trace silt, (Stream Alluvium).							64							78							79							64	35	8D	24/16	35.00 - 37.00	2/2/3/3	5	8	80	301.9	44.8	Grey, wet, medium stiff, SILT, some clay, little fine sand, (Glaciomarine).	G#271129 A-4, CL WC=35.2% Non-Plastic							76							73							62							HYD PUSH	40	9D	24/18	40.00 - 42.00	1/1/2/2	3	5		301.9	44.8	Grey, wet, medium stiff, SILT, little clay, trace fine sand, (Glaciomarine).																													45	10D	24/5	45.00 - 47.00	6/6/7/4	13	21	130	301.9	44.8	Grey, wet, medium dense, Gravelly SAND, little silt, (Glacial Till).							143							120							149							---
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Maine Department of Transportation						Project: Gower Farm Bridge #2324 carries Routes 2/27 over Beales Brook Location: Farmington, Maine								Boring No.: BB-FBB-101					
Soil/Rock Exploration Log US CUSTOMARY UNITS														WIN: 21880.00					
Driller: Northern Test Boring						Elevation (ft.) 346.7						Auger ID/OD: 2.25/6.25"							
Operator: Nadeau/Taylor						Datum: NAVD88						Sampler: Standard Split Spoon							
Logged By: A. Van Buskirk						Rig Type: Dietrich D-50 Track						Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 2/12/2018; 08:30-14:30						Drilling Method: Cased Wash Boring						Core Barrel: N/A							
Boring Location: 4+99, 12.8 ft Rt.						Casing ID/OD: HW-4"						Water Level*: None Observed							
Hammer Efficiency Factor: 0.977						Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐													
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50	11D	24/9	50.00 - 52.00	13/26/42/40	68	111	70	294.5		Grey, moist, very dense, SAND, some gravel, little silt, (Glacial Till).									
							a160		a160 blows for 0.2 ft. Roller Coned ahead to 52.2 ft bgs.										
									Bottom of Exploration at 52.2 feet below ground surface. REFUSAL, broke HW Casing, abandoned 15.0 ft of HW casing in borehole.										
75																			
Remarks: Automatic Hammer SN# 377																			
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 3 of 3									
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-FBB-101									

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Gower Farm Bridge #2324 carries Routes 2/27 over Beales Brook Location: Farmington, Maine				Boring No.: BB-FBB-102 WIN: 21880.00				
Driller: Northern Test Boring				Elevation (ft.): 346.4				Auger ID/OD: 2.25/6.25"				
Operator: Nadeau/Taylor				Datum: NAVD88				Sampler: Standard Split Spoon				
Logged By: A. Van Buskirk				Rig Type: Dietrich D-50 Track				Hammer Wt./Fall: 140#/30"				
Date Start/Finish: 2/13-14/2018; 08:00-14:30				Drilling Method: HSA/Cased Wash Boring				Core Barrel: NQ-2				
Boring Location: 5+02.8, 12.3 ft Lt.				Casing ID/OD: HW-4"/NW-3"				Water Level*: None Observed				
Hammer Efficiency Factor: 0.977				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							HSA	346.1		4" HMA.	G#271130 A-2-4, SM WC=13.6%	
	1D	8.4/8.4	2.50 - 3.20	48/50(2.4")	---							Brown, dry, fine to medium SAND, trace silt, (Fill).
5	2D	24/22	5.00 - 7.00	48/60/53/46	113	184						Similar to 1D, except very dense, (Fill).
10	3D	18/17	10.00 - 11.50	19/58/64(6")								Brown, moist, very dense, fine to medium SAND, some silt, trace coarse sand, trace coarse gravel, (Fill).
15	4D	24/8	15.00 - 17.00	31/40/40/18	80	130	80					Similar to 3D, (Fill).
							59					
							31					
							49					
20	5D	24/8	20.00 - 22.00	4/5/6/5	11	18	32					Grey, wet, medium dense, fine to medium SAND, trace coarse sand, trace silt, trace gravel, (Stream Alluvium).
							53					
							60					
							84					
25							31					
Remarks: Auto hammer SN# 377.												
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-FBB-102		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Gower Farm Bridge #2324 carries Routes 2/27 over Beales Brook Location: Farmington, Maine		Boring No.: BB-FBB-102 WIN: 21880.00									
Driller: Northern Test Boring		Elevation (ft.): 346.4		Auger ID/OD: 2.25/6.25"											
Operator: Nadeau/Taylor		Datum: NAVD88		Sampler: Standard Split Spoon											
Logged By: A. Van Buskirk		Rig Type: Dietrich D-50 Track		Hammer Wt./Fall: 140#/30"											
Date Start/Finish: 2/13-14/2018; 08:00-14:30		Drilling Method: HSA/Cased Wash Boring		Core Barrel: NQ-2											
Boring Location: 5+02.8, 12.3 ft Lt.		Casing ID/OD: HW-4"/NW-3"		Water Level*: None Observed											
Hammer Efficiency Factor: 0.977		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐													
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (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								
25	6D	24/8	25.00 - 27.00	3/5/4/6	9	15	42	312.4		Grey, wet, medium dense, fine to medium SAND, trace coarse sand, trace silt, trace gravel, (Stream Alluvium).	G#271132 A-1-b, SP WC=18.2%				
							60								
							62								
							89								
							33								
30	7D	24/6	30.00 - 32.00	5/6/5/3	11	18	43					34.0		Grey, wet, medium dense, SAND, little gravel, trace silt, (Stream Alluvium).	
							55								
							63								
							70								
							RC								
35	8D	24/15	35.00 - 37.00	1/2/2/1	4	7		34.0		Grey, wet, medium stiff, SILT, little clay, trace fine sand, (Glaciomarine).	G#271133 A-4, CL WC=33.1% Non-Plastic				
40	9D	24/7	40.00 - 42.00	3/8/19/12	27	44		302.8		Grey, wet, hard, SILT, little gravel, (Glaciomarine).					
	R1	7/7	43.60 - 44.18				NQ-2					43.6		NW REFUSAL R1: Cobble from 43.6-44.2 ft bgs.	
45	R2	18/11	45.60 - 47.10				NQ-2	43.6		NW REFUSAL R2: Boulder from 45.6-47.1 ft bgs. Roller Coned ahead to 50.0 ft bgs, advanced NW Casing.					
50															
Remarks: Auto hammer SN# 377.															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3 Boring No.: BB-FBB-102					

[illegible]

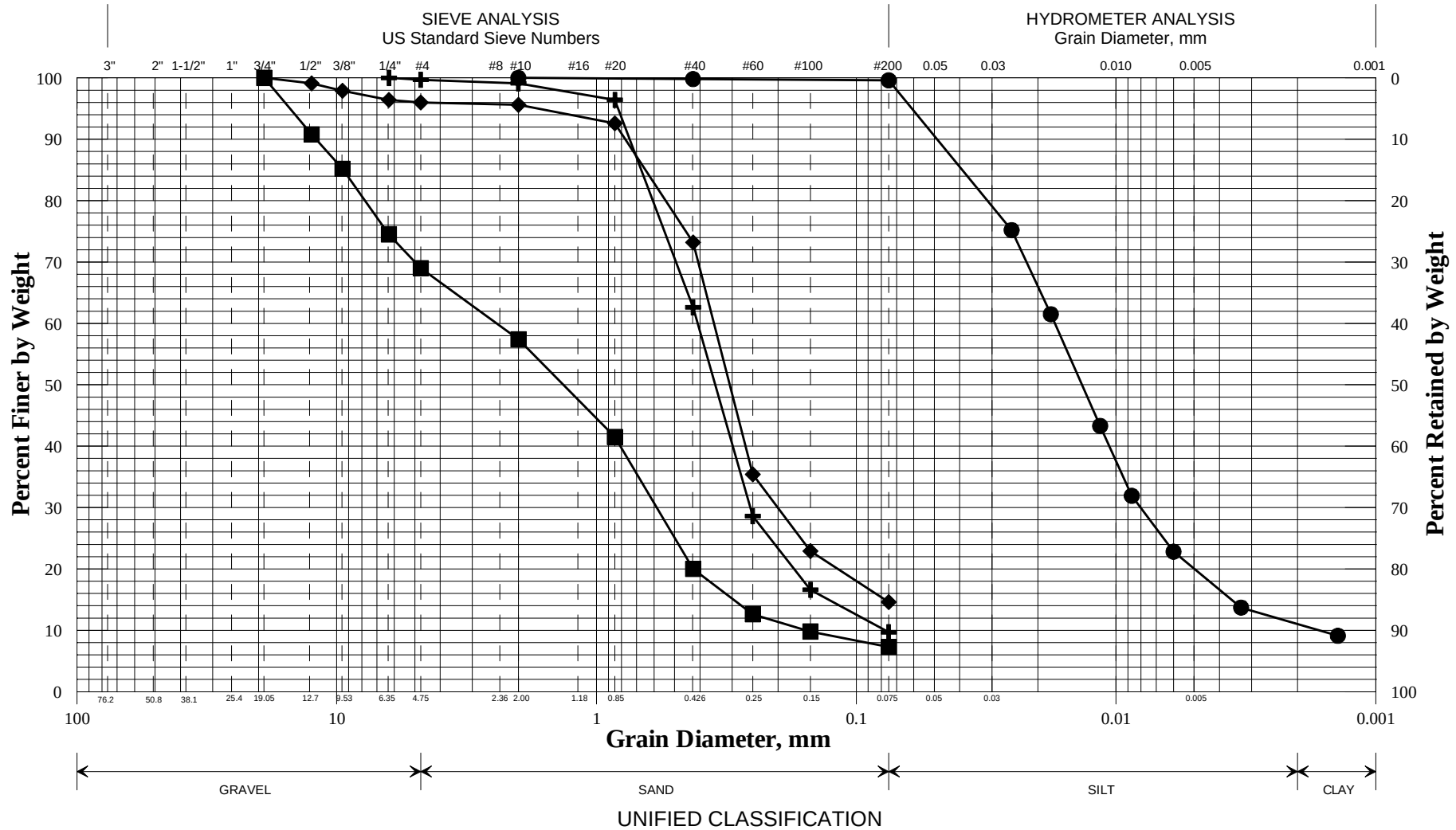
Appendix B

Laboratory Test Results

Work Number: 21880.00

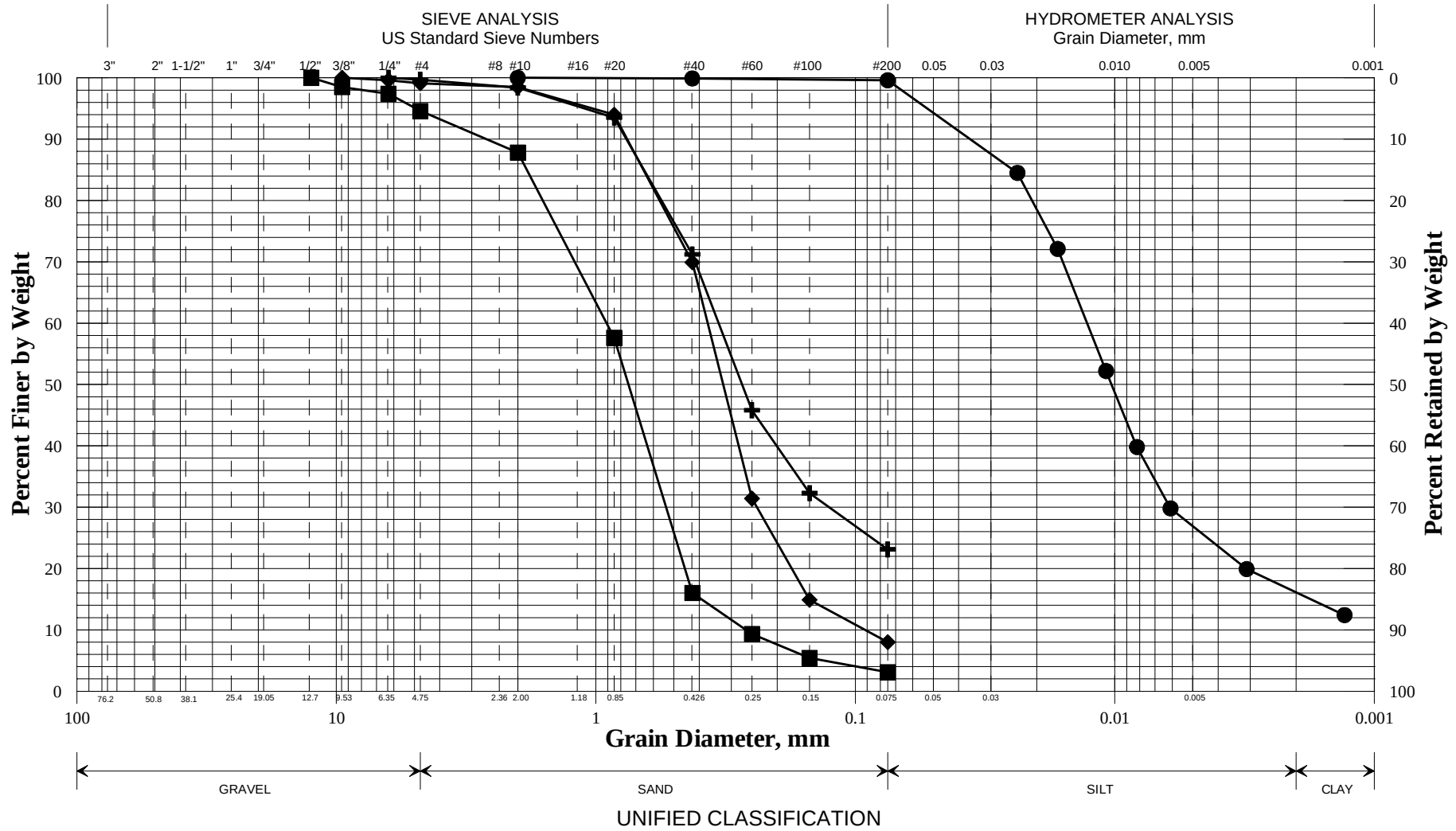
PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

[illegible]

WIN
021880.00
Town
Farmington
Reported by/Date
WHITE, TERRY A 3/28/2018

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-FBB-102/3D	5+02.8	12.3 LT	10.0-11.9	SAND, some silt, trace gravel.	13.6			
◆	BB-FBB-102/5D	5+02.8	12.3 LT	20.0-22.0	SAND, trace silt, trace gravel.	25.4			
■	BB-FBB-102/6D	5+02.8	12.3 LT	25.0-27.0	SAND, trace gravel, trace silt.	18.2			
●	BB-FBB-102/8D	5+02.8	12.3 LT	35.0-37.0	SILT, little clay, trace sand.	33.1			NP
▲									
×									

WIN	
021880.00	
Town	
Farmington	
Reported by/Date	
WHITE, TERRY A	3/28/2018



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271129** Boring No./Sample No. **BB-FBB-101/9D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **4+99** Offset, ft: **12.8** RT Dbfg, ft: **40.0-42.0**

WIN/Town **021880.00 - FARMINGTON** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.6
[0.0252 mm]	75.2
[0.0178 mm]	61.5
[0.0115 mm]	43.3
[0.0087 mm]	31.9
[0.0060 mm]	22.8
[0.0033 mm]	13.7
[0.0014 mm]	9.1

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)	2.72
Loss on Ignition, % (T 267)	
Water Content (T 265), %	35.2

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:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **3/23/2018**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271133** Boring No./Sample No. **BB-FBB-102/8D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **5+02.8** Offset, ft: **12.3** LT Dbfg, ft: **35.0-37.0**

WIN/Town **021880.00 - FARMINGTON** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.6
[0.0237 mm]	84.5
[0.0166 mm]	72.1
[0.0108 mm]	52.2
[0.0082 mm]	39.8
[0.0061 mm]	29.8
[0.0031 mm]	19.9
[0.0013 mm]	12.4

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)	2.66
Loss on Ignition, % (T 267)	
Water Content (T 265), %	33.1

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	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **3/23/2018**

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.

$$K_0 := 1 - \sin(\phi)$$

$$K_0 = 0.47$$

Recommend: At-Rest Earth Pressure Coefficient, $K_0 = 0.47$

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 $\beta := 26.56 \cdot \text{deg}$

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

Pa is oriented at an angle of β 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 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

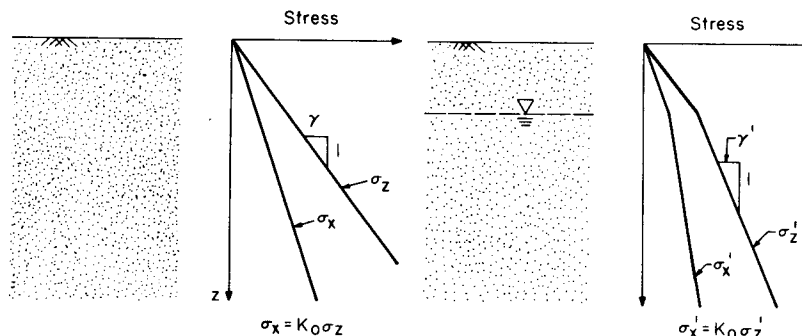


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

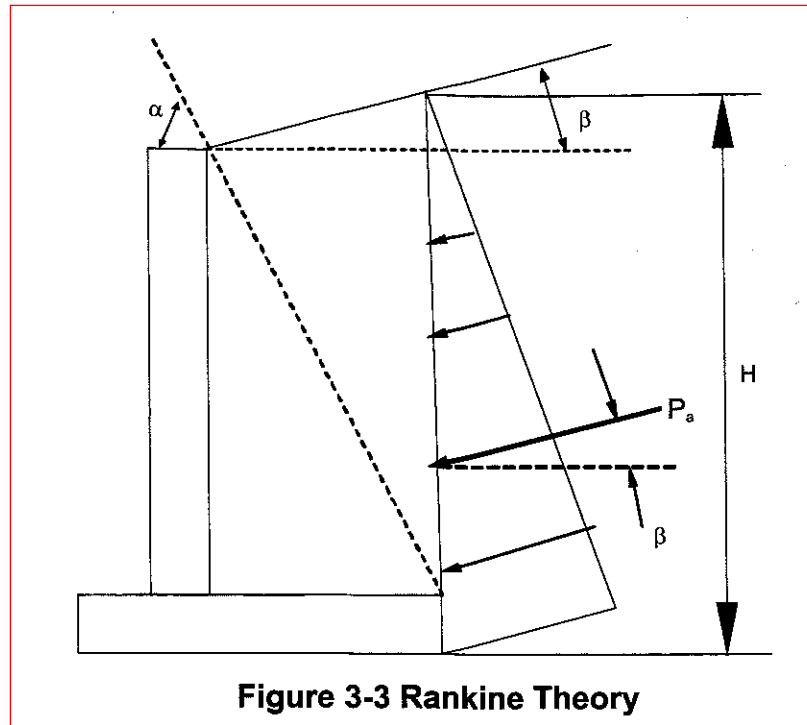


Figure 3-3 Rankine Theory

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 culvert bedding stone.
2. The one foot thick layer of proposed culvert bedding stone material is neglected.
3. The proposed bearing elevation is approximately 326.4 feet.
4. Proposed finish roadway grade elevation is approximately 347.2 feet.
5. Proposed precast concrete box base is 22 feet wide.
6. Use design N-value of 5 bpf for the consistency of the soils encountered at the box bearing elevation, based on BB-FBB-101 6D.
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	Loose	2-4	3

The alluvial sand deposit is loose to medium dense in consistency. Recommend 3 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

$$B := 22\text{ft}$$

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

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

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

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-FBB-101 6D

$$\gamma_{1d} := 92 \cdot \text{pcf}$$

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

$$w_{\text{sat}} := 28.1\%$$

Moisture content of BB-FBB-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

$$\gamma_{1\text{sat}} = 117.9 \cdot \text{pcf}$$

$$N_{\text{design}} := 5$$

$$\phi := 28 \cdot \text{deg}$$

AASHTO Table 10.4.6.2.4-1 and C10.4.6.2.4.

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)

$$N_c := 25.8$$

$$N_q := 14.7$$

$$N_\gamma := 16.7$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume $L = 2B$

$$L := 2 \cdot B$$

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

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

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

$$s_c = 1.3$$

$$s_\gamma = 0.8$$

$$s_q = 1.3$$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

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

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

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

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

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 33.2$$

$$N_{\gamma m} = 13.4$$

$$N_{qm} = 18.6$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 9.8 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$q_r = 4.4 \cdot \text{ksf}$$

Recommend a limiting value for the factored bearing resistance of 4.4 ksf, for box bottom slabs 22 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.

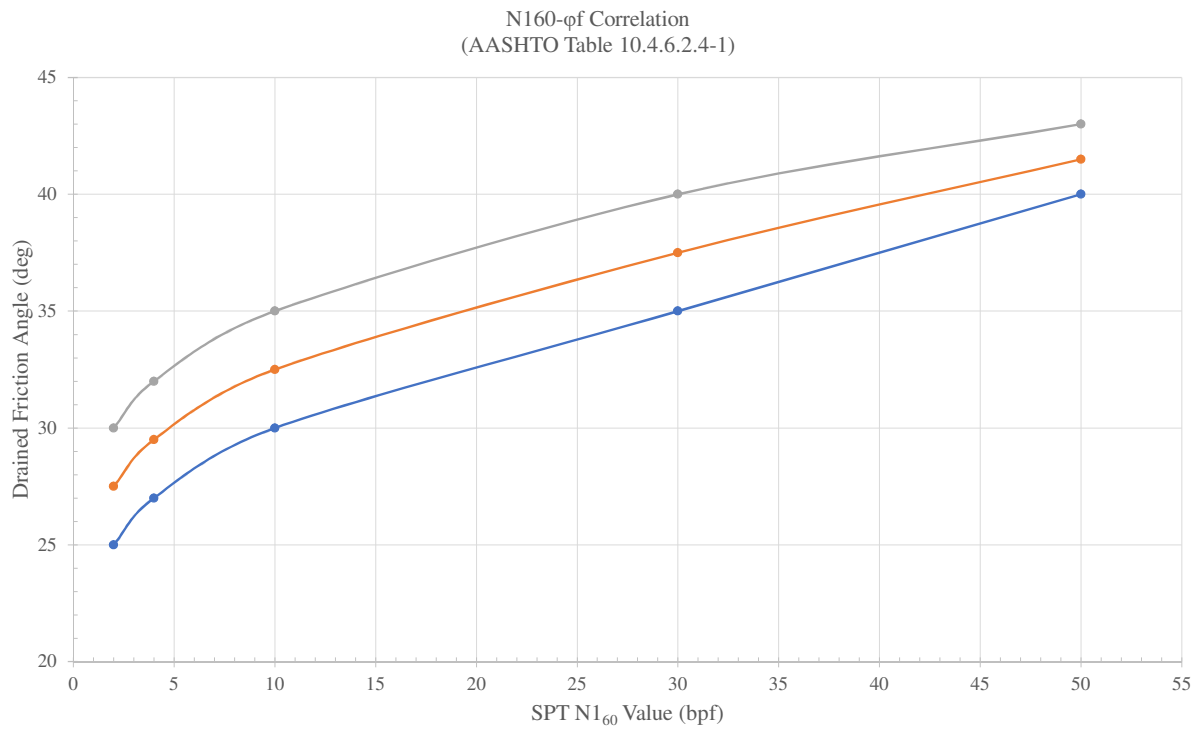
Table 3.1 Various Forms of Relationships for γ , γ_d , and γ_{sat}

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

N1 60 and Soil Internal Angle of Friction Correlation
AASHTO Table 10.4.6.2.4-1 and Commentary C10.4.6.2.4
Correlation per Peck, Hanson and Thornburn with ranges for Fine (blue), Medium (orange) and Coarse (grey) materials



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.

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.

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

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

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.

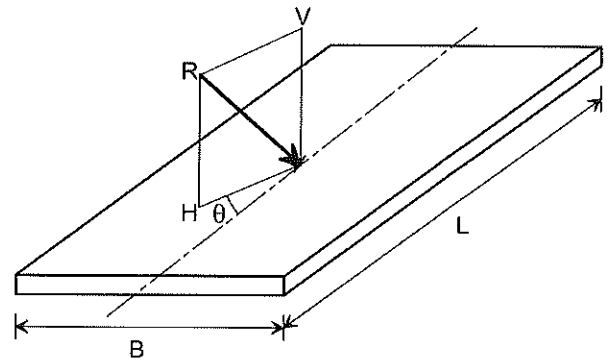


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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 base slab design.

Given:

1. Limited lab data and SPT N-values.

Assumptions:

1. The proposed bearing elevation of base slab is approximately 326 feet.
2. Proposed finished roadway grade is approximately 347.
3. Proposed precast concrete box is 22 feet wide and approximately 71 feet long (excluding slab connecting slope tapered outlet and inlet sections).
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.

Published values of subgrade modulus

Published values of subgrade modulus in medium sand:

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

Range of modulus of subgrade reaction 18 to 59 pci

Subgrade of medium dense sand, average of upper and lower limit: $k_s = 165$ pci

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

Coarse grained medium sand K_{v1} , 220 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 sand, 37-55 pci: $k_{0.3}$ (k_1) = 138 pci

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

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

Submerged medium sand: $k_{s1} = 93$ pci

Adjust Published values for dimensions of base slab

Assume a subgrade modulus of 46 pci for medium dense sand.

Value of $k_{s1} = 46$ pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B = 22 ft, Length L = 71 ft).

Square to rectangle base adjustment:

$$k_{s1} := 46 \text{ pci} \quad B := 22 \text{ ft} \quad L := 71 \text{ ft}$$

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

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

$$k = 35.4 \cdot \text{pci}$$

Recommend a subgrade modulus of 35 pci

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)

variation 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 n obtained using one of the approximations given here. Obviously if a value is three times larger than the table range indicates, the computations will have a possible gross error. Note, however, if you use a reduced value of n (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 adjust the table values are intended as guides. The reader should not use, say, a value of 10 as a "good" estimate.

shown in Fig. 9-9c (and used in your diskette program FADBEMLP as a starting point) 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

reaction k_s

Use values as guide and for comparison when using approximate equations

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

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

39 pci

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

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

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

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

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

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

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

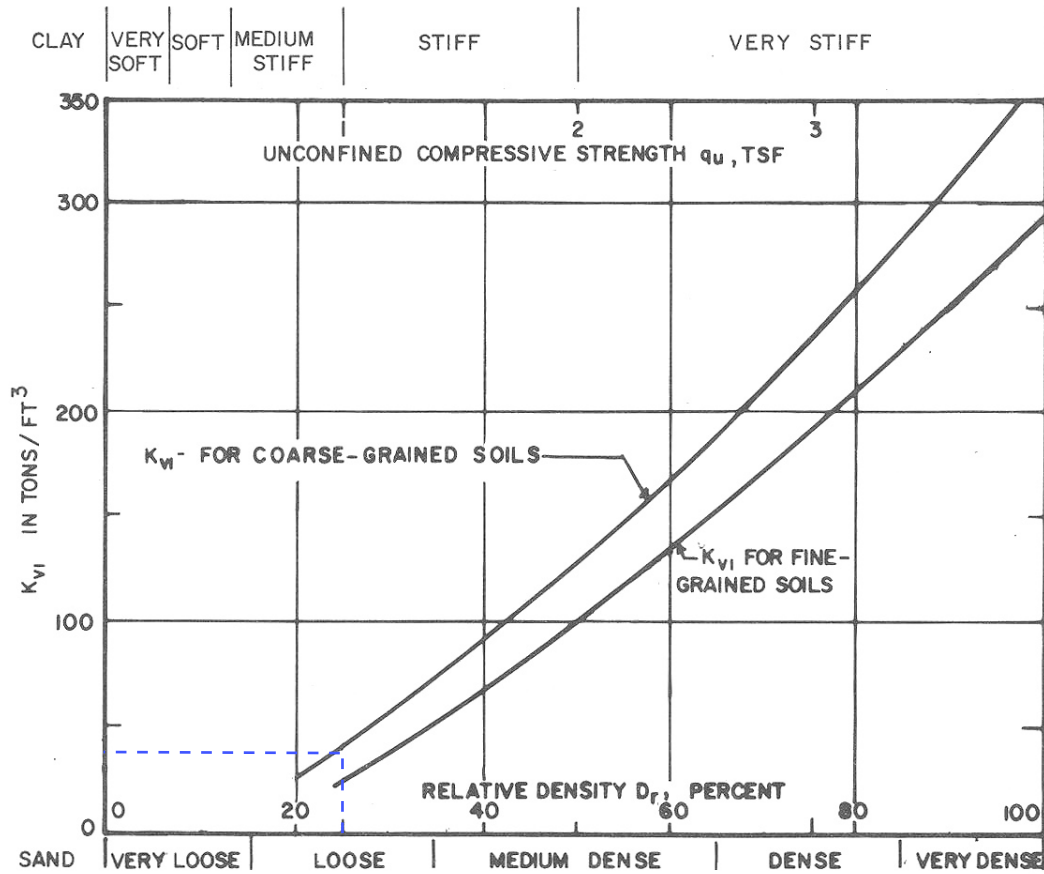
You should note that k_s does not depend on the contact pressure of the base q_o .

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)]

40 TCF =
46 PCI



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

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

COARSE-GRAINED SOILS

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

FOR $B \leq 20$ FT:

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

FOR $B \geq 40$ FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

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

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

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

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

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

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

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

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

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

or

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

where

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

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

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

Soil type	$k_{0.3}(k_1)$ MN/m ³	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

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

$$25 \text{ ton/ft}^3 \times 1.157 = 29 \text{ pci}$$

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 ultimate equation (9).

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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 l

Loose alluvium

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

DFI = 1870 degree-days.

Case 1 - coarse grained granular fill soils W=20% .

$$DFI_1 := 1800 \quad d_1 := 74.5 \cdot \text{in}$$

$$DFI_2 := 1900 \quad d_2 := 76.6 \text{ in}$$

Approximate DFI at project = 1870, find frost depth by interpolation:

$$DFI_3 := 1870$$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1) \cdot (d_2 - d_1)}{(DFI_2 - DFI_1)} \quad d_3 = 76 \cdot \text{in}$$

$$d_3 = 6.3 \cdot \text{ft}$$

Depth of Frost Penetration

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

--- ModBerg Results ---

Project Location: Farmington, Maine

Air Design Freezing Index = 2023 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1618 F-days
Mean Annual Temperature = 41.2 deg F
Design Length of Freezing Season = 145 days

Layer									
#:Type	t	w%	d	Cf	Cu	Kf	Ku	L	
1-Coarse	72.7	20.0	104.0	28	38	1.9	1.2	2,995	

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).

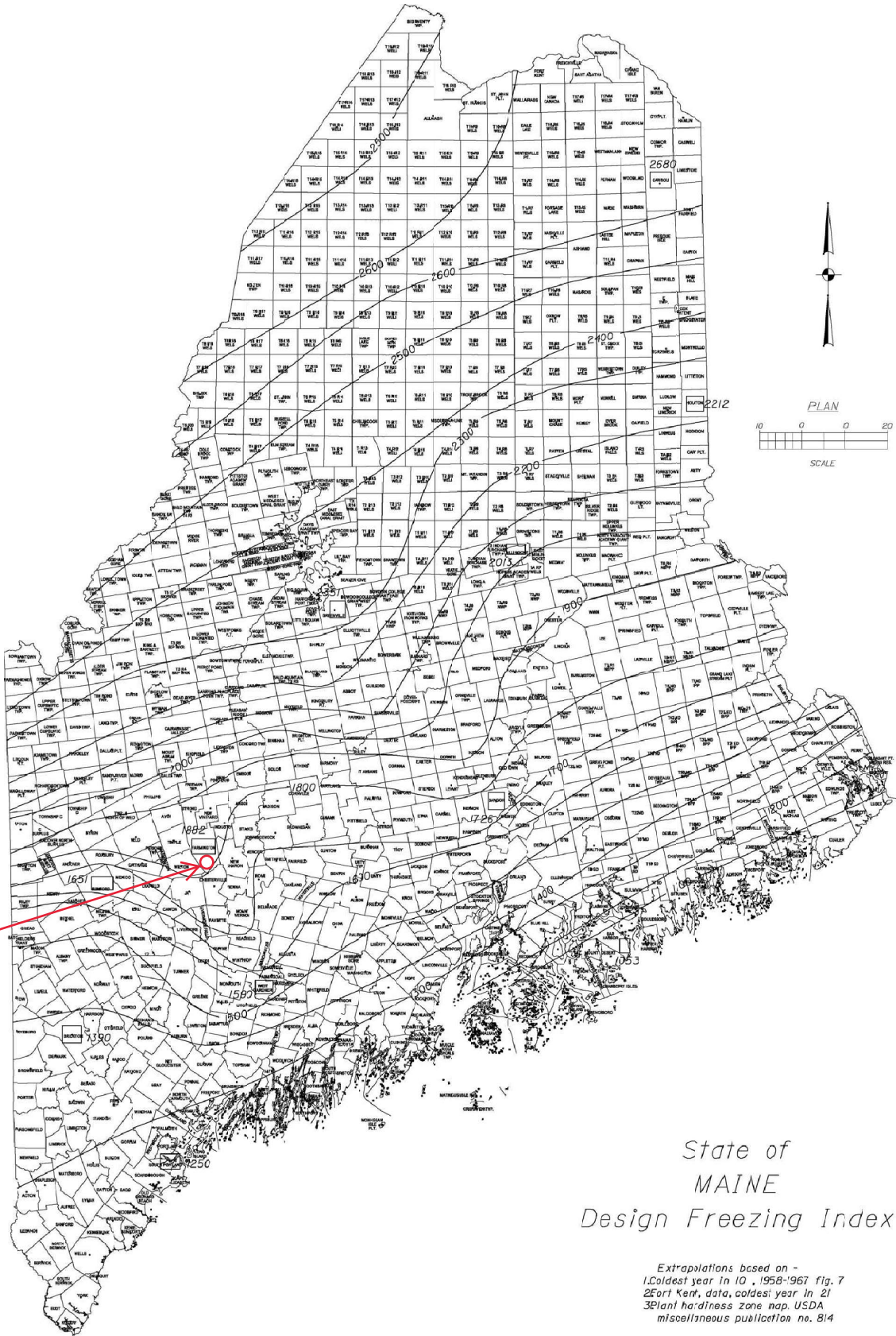
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

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

Total Depth of Frost Penetration = 6.05 ft = 72.7 in.

Recommendation: 6.3 feet for design of foundations constructed on coarse grained soils

Figure 5-1 Maine Design Freezing Index Map



Project Location

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
1870