

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

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**WILLIAMS #1 BRIDGE
STATE ROUTE 156 OVER WILLIAMS BROOK
FARMINGTON, MAINE**

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Franklin County
WIN 21690.00

Soils Report 2019-24
Bridge No. 2980

Fed. No. STP-2169(000)
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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	FILL.....	3
5.2	STREAM ALLUVIUM	3
5.3	GLACIOMARINE DEPOSIT	3
5.4	GLACIAL TILL	4
5.5	GROUNDWATER	5
6.0	FOUNDATION 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.....	7
7.1.4	BEARING RESISTANCE	7
7.1.5	MODULUS OF SUBGRADE REACTION	7
7.2	SETTLEMENT	7
7.3	SUBGRADE PREPARATION	8
7.4	FROST PROTECTION.....	8
7.5	SCOUR AND RIPRAP.....	8
7.6	SEISMIC DESIGN CONSIDERATIONS	9
7.7	CONSTRUCTION CONSIDERATIONS	9
8.0	CLOSURE	10

Tables

Table 1 – Atterberg Limits Test Results

Table 2 – D₅₀ of Representative Samples for Scour

Sheets

Sheet 1 – Location Map

Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile

Sheet 3 – 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 Williams #1 Bridge, which carries State Route 156 over Williams 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 1954 and consists of 20-foot span corrugated steel plate arch approximately 64 feet in length set on timber grillage. The previous structure (1931) was a timber bridge on granite stone crib abutments. According to the 2017 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culvert is rated 4 (poor condition). The steel arch has corroded and shows scaling 1 foot above and below the waterline. There is minor section loss at the edges at the upstream ends. The structure has a NBI Sufficiency Rating of 70.0. The structure is due for full replacement in the 17/18/19 Workplan.

The proposed replacement structure will be a 24-foot span by 11-foot rise precast concrete box. The concrete box culvert will have 1-foot tall precast headwalls and toe walls. The upstream and downstream ends of the culvert will be slope-tapered to match the 1.75H:V (horizontal:vertical) sideslopes. The box culvert will be installed approximately 3 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be installed on a 2-foot-thick layer of crushed stone that is reinforced with geogrid and wrapped in stabilization/reinforcement geotextile placed on the native soil subgrade.

The new box culvert will be located on approximately the same horizontal and vertical alignment as the existing plate arch. The bridge replacement project will occur during a short bridge closure of approximately 30 days with a detour on state aid roads.

2.0 GEOLOGIC SETTING

Williams #1 Bridge carries State Route 156 over Williams Brook approximately .19 mile southeast of the Wilton town line, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Reconnaissance Surficial Geology of the Farmington Quadrangle, Open File No. 86-29 (1986) indicates the surficial soil unit in the vicinity of the bridge is glacial stream deposits, which consist of sand and gravel, and are commonly buried by glaciomarine deposits. The glacial stream deposits have contacts to glacial till. Glacial till includes two varieties: basal till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage.

The MGS Reconnaissance Bedrock Geology of the Farmington Quadrangle, Open-File No. 78-16 (1978) cites the bedrock at the project site as the Anasagunticook Member of the Sangerville Formation, consisting of thinly interlaminated grey metasiltstone and dark grey metapelite and to a lesser extent, light grey, calcareous greywacke metasandstone bedded with grey metasiltstone and dark grey metapelite.

3.0 SUBSURFACE INVESTIGATION

Two borings were drilled at the site: BB-FWB-101 was drilled west of the existing steel arch and boring BB-FWB-102 was drilled east of the structure, both in the westbound lane. The boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

Northern Test Boring (NTB) of Gorham, Maine, drilled the borings on February 15 and 16, 2018. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were performed using solid stem auger and cased wash boring 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 drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” prior to the borings in June 2017. 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.

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 four standard grain size analyses with natural water contents, six grain size analyses with hydrometer, and six 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, Stream Alluvium, Glaciomarine Clayey Silt, and Glacial Till. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of fill was encountered in both borings. The thickness of the fill unit encountered was approximately 10 to 14 feet at the boring locations. The unit encountered generally consisted of:

- Brown, damp, sand, some gravel, trace to little silt;
- Brown, dry, fine to medium sand, trace silt, trace gravel;
- Brown, moist, sandy silt, trace gravel.

SPT N_{60} -values in the layer ranged from 7 to 55 blows per foot (bpf), indicating the layer is medium stiff to very dense in consistency. Grain size analyses conducted on samples of the fill indicate the soil is classified as A-1-b and A-4 under the AASHTO Soil Classification System and SW-SM and CL under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 7 to 24 percent.

5.2 Stream Alluvium

A deposit of stream alluvium was encountered beneath the fill in boring BB-FWB-102. The thickness of the alluvial deposit encountered was approximately 5 feet at the boring location. The alluvial deposit encountered generally consisted of:

- Brown, wet, silty fine sand, trace organics.

A SPT N_{60} -value in the alluvial deposit was 5 bpf indicating the soil is loose in consistency.

5.3 Glaciomarine Deposit

A glaciomarine deposit of clayey silt was encountered below the fill unit in BB-FWB-101 and below the stream alluvium in BB-FWB-102. The thickness of the deposit encountered was approximately 3 to 6 feet at the boring locations. The glaciomarine deposit encountered generally consisted of:

- Grey, wet, clayey silt, trace sand;
- Grey, wet, silt, some clay, trace sand, trace gravel.

SPT N_{60} -values in the layer ranged from 2 to 7 bpf, indicating the layer is soft to medium stiff in consistency. The remainder of SPT tests were conducted through soils disturbed by in-situ vane shear tests and therefore are invalid.

Grain size analyses with hydrometer conducted on samples from the clayey silt deposit resulted in the material being classified as A-4 under the AASHTO Soil Classification System and ML and CL under the USCS. The natural water contents of the samples tested ranged from approximately 33 to 38 percent.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the clayey silt deposit.

A 55 x 110 mm vane was used. Occasionally vane shear tests could not be completed because the vane was unable to be pushed by hand to test depth; this is noted on the boring logs. Three of six successful vane shear tests conducted within the clayey silt layers measured undisturbed undrained shear strengths ranging from approximately 491 to 737, indicating the soil is soft to medium stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 89 to 156 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the clayey silt has a sensitivity of approximately 5 and is classified as medium sensitive.

Atterberg limits tests were conducted on six samples of the silty clay. Four of the tests resulted in the samples being classified as non-plastic. The results of the other two tests are summarized in Table 1:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-FWB-101, 3D	Clayey SILT	38.0	33	24	9	1.6
BB-FWB-102, 4D	Clayey SILT	38.0	32	23	9	1.7

Table 1 – Atterberg Limits Test Results

The plasticity indices of the samples indicate that the soil has medium plasticity (Burmister, 1949). The liquidity indices were 1.6 and 1.7. Soils with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 also have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding and has the potential to convert this type of soil deposit into a liquid. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as “quick.”

5.4 Glacial Till

Glacial till was encountered beneath the glaciomarine layer in both borings. The glacial till deposit encountered generally consisted of:

- Grey, moist to wet, silt, little to some sand, trace to little gravel, trace to little clay;
- Grey, wet, silty fine to medium sand, trace to little gravel;
- Grey, moist, fine sandy silt;
- Grey, wet, sandy silt, little gravel, wood fibers.

SPT N₆₀-values in the layer ranged from 7 to 116 bpf, indicating the layer is loose to very dense in consistency. Grain size analyses conducted on samples from the glacial till indicated the material 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 11 to 15 percent. Both borings were terminated in the glacial till at a depth of 52 feet bgs without encountering bedrock.

5.5 Groundwater

Groundwater elevations were recorded at 10.0 feet and 11.6 feet bgs in the boreholes at the completion of the borings. Water was introduced into the boreholes during drilling operations. Therefore, water levels may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 FOUNDATION 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 24-foot span precast concrete box;
- A 55-foot single span integral abutment bridge on H-piles driven to bedrock.

A 24-foot span precast concrete box culvert has the advantages of efficiency of installation, low maintenance and low lifecycle costs. The precast concrete box culvert was chosen as the preferred alternative.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 24-foot span by 11-foot rise precast concrete box culvert with slope-tapered inlet and outlet walls. The box culvert will have 1 foot tall precast headwalls. To prevent undermining, the box culvert will have inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 2 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. The riprap apron should be embedded into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

The box culvert will be constructed on an 18-inch thick layer of crushed stone reinforced with geogrid and wrapped in stabilization/reinforcement geotextile. An additional 6-inches of compacted bedding stone shall be placed on top of the geotextile-wrapped stone to facilitate sliding and fitting of precast box segments. Once the soil subgrade is prepared for the crushed stone mat construction, the stabilization/reinforcement geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. Crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor (method of compaction approximating 97 percent of AASHTO T-108 maximum dry density).

The geotextile shall meet Class 1 Stabilization/Reinforcement Geotextile meeting MaineDOT Standard Specification 722.01. Adjoining sections of the stabilization geotextile should be

overlapped by a minimum of 1 foot.

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 1.75H: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 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 1.75H:1V should be designed using the Rankine active earth pressure coefficient of 0.56. 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 installed on an 18-inch-thick layer of crushed stone that is reinforced with geogrid and wrapped in stabilization/reinforcement geotextile placed on the native soil subgrade. To facilitate sliding and fitting of adjacent box segments, the wrapped crushed stone mat shall be topped with 6 inches of compacted crushed stone. The bearing elevation of the crushed stone mat will be approximately Elev. 337 at the inlet and 336 at the outlet. The subgrade soils at this elevation are expected to be soft to stiff marine clayey silt.

For a precast concrete box culvert with a base width of 26 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 2.0 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 1.5 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 28 pounds per cubic inch (pci).

7.2 Settlement

An approximately 3 to 6-foot thick deposit of soft marine clay was encountered at the proposed bottom slab elevation of box culvert. The soft deposit will undergo immediate and consolidation settlement in response to a net applied increase of vertical overburden pressure. Based on an estimated service limit state pressure of 1,500 psf for a 23-foot wide precast concrete box, and a net pressure increase of 500 psf, we estimate settlements on the order of 1-inch during the first year after construction and an additional 0.4-inch of long term settlement (i.e. secondary) over 50 years. Should the net pressure increase approach 1500 psf due to the proposed box culvert, 1-year consolidation settlement could potentially be on the order of 3-inches with an additional 0.4-inch of long term, secondary settlement.

See Appendix C – Calculations for supporting documentation.

7.3 Subgrade Preparation

The marine clay layer encountered at the bearing elevation 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 the bearing pad construction in the dry;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade;
- Construct the box on an 18-inch thick layer of crushed stone layer reinforced with geogrid and wrapped in a heavy non-woven stabilization/ reinforcement geotextile, and topped with 6 inches of compacted crushed stone;
- Hand-deploy the geotextile on the prepared clay subgrade prior to installing the geogrid-reinforced stone mat.

The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor.

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 30% was used for fine grained soils. These components correlate to a frost depth of 4.2 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 30% was used. These components correlate to a frost depth of approximately 3.9 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on soil be designed with an embedment of approximately 4.2 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.5 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and wingwalls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1 foot below the maximum depth of scour. Inlet and outlet toe walls shall also be protected with riprap aprons. The riprap shall be

underlain by a Class 1 erosion control geotextile and a 1-foot-thick layer of bedding material conforming to MaineDOT Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot beneath the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

Grain size analyses were performed on samples from the marine clay deposit to generate grain size curves for determining parameters to be used in scour analyses. The selected samples were judged to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameter can be used in scour analyses:

Soil Unit	Sample	Sample Depth (ft)	D ₅₀ (mm)	Representative D ₅₀ (mm)	Classification
Marine Clay	BB-FWB-101/3D	15-17	0.003	0.003	Silt
	BB-FWB-102/4D	15-17	0.003		
	BB-FWB-102/5D	20-22	0.005		

Table 2 - D₅₀ of Representative Samples for Scour

7.6 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

7.7 Construction Considerations

The box culvert will be constructed on a 2-foot thick layer of crushed stone reinforced with geogrid; the lower 18-inches of the crush stone mat shall be wrapped in stabilization/reinforcement geotextile. The geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor. The geotextile shall meet Class 1 Stabilization/Reinforcement Geotextile meeting MaineDOT Standard Specification 722.01. Adjoining sections of the stabilization geotextile should be overlapped by a minimum of 1 foot.

The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to limit disturbance and rutting of the clay and 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 not operate heavy equipment over the excavated subgrade to minimize subgrade disturbance.
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the clay subgrade.
- The stabilization/reinforcement geotextile shall be hand-deployed on the prepared clay subgrade prior to installing the geogrid-reinforced stone mat.

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 sensitive, 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.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT for specific application to the proposed replacement of Williams #1 Bridge in Farmington, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

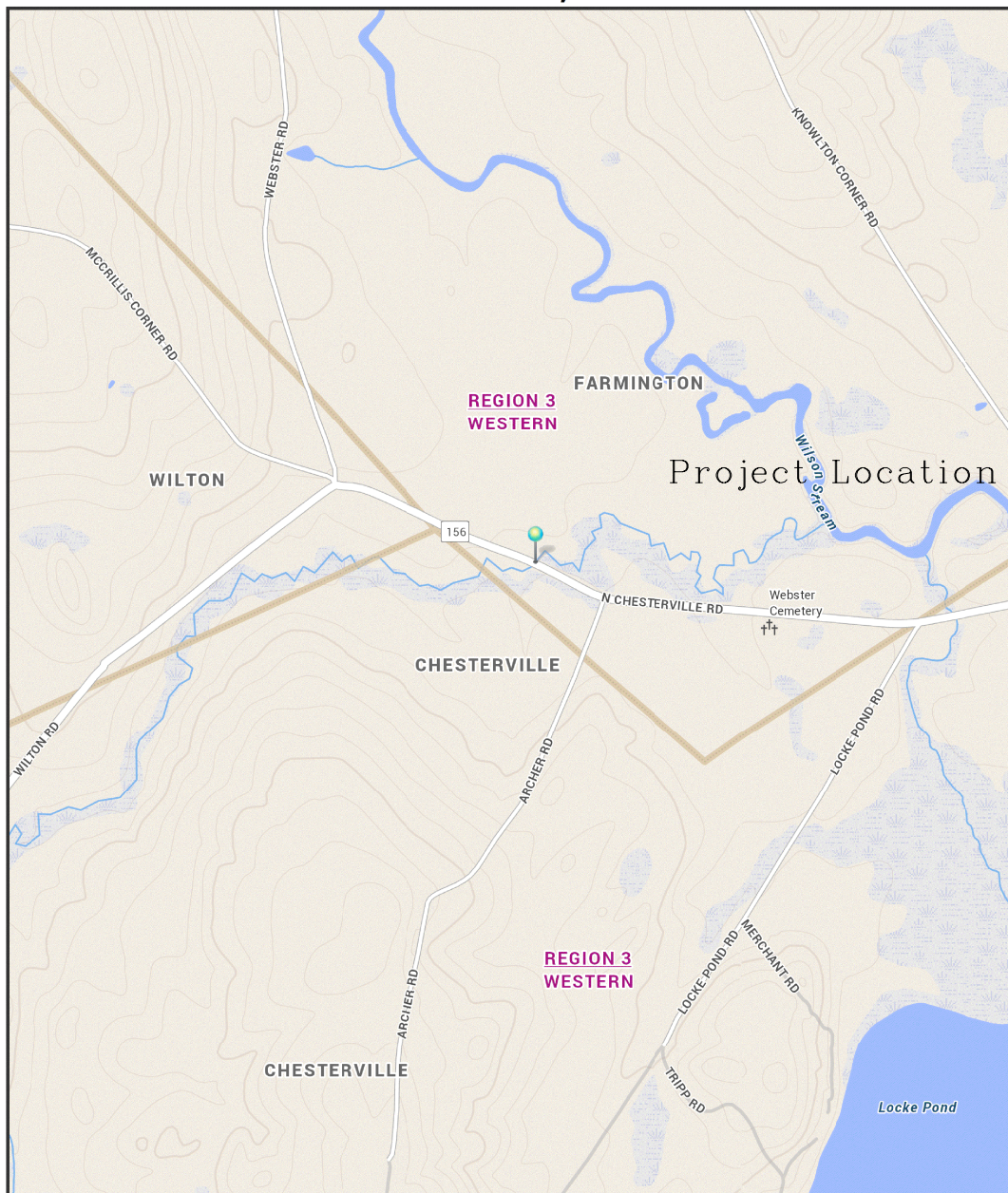
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

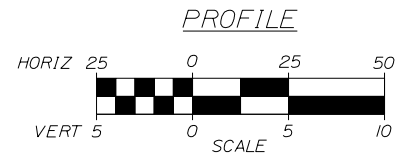
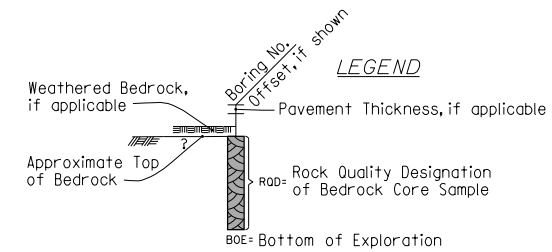
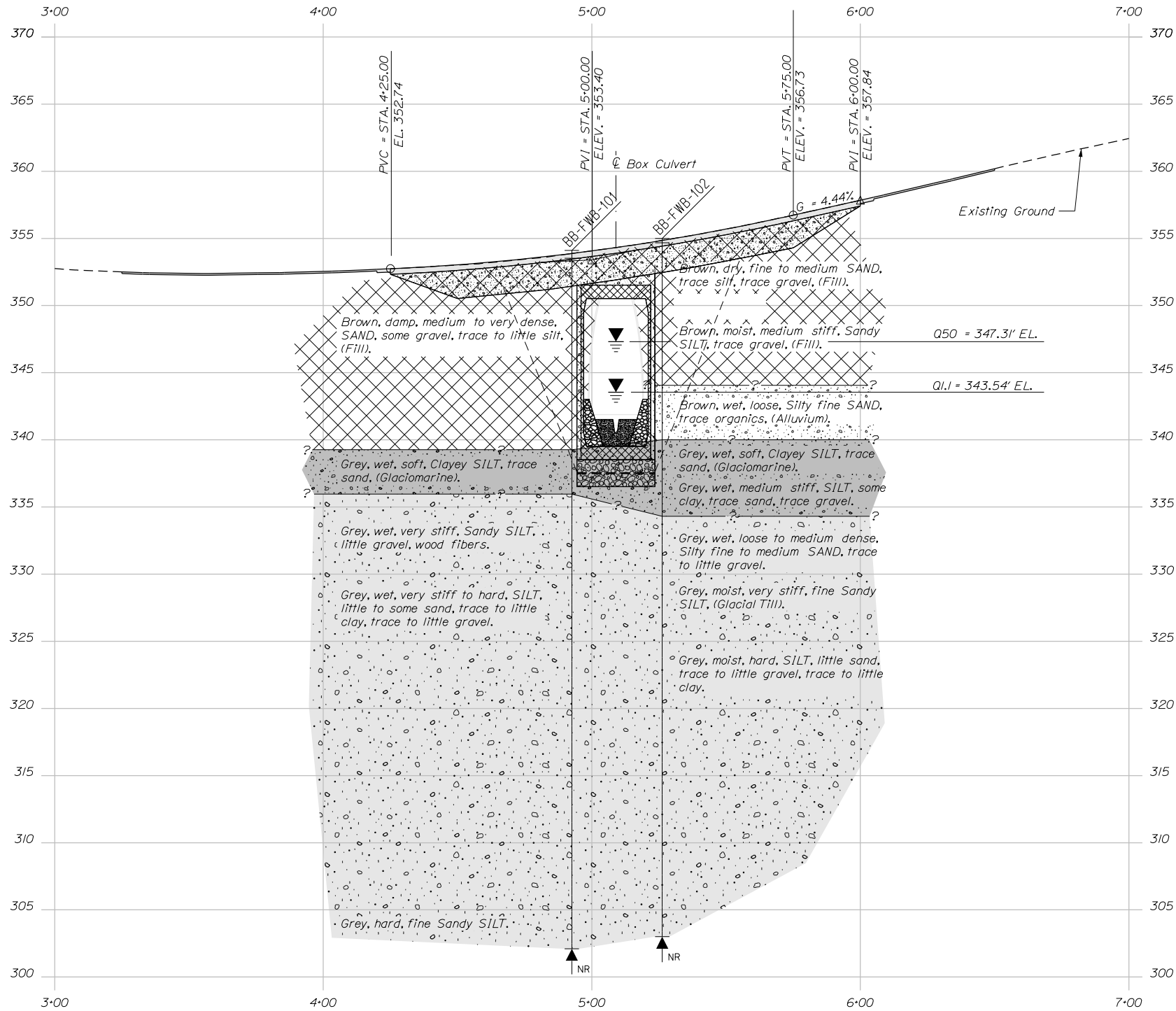
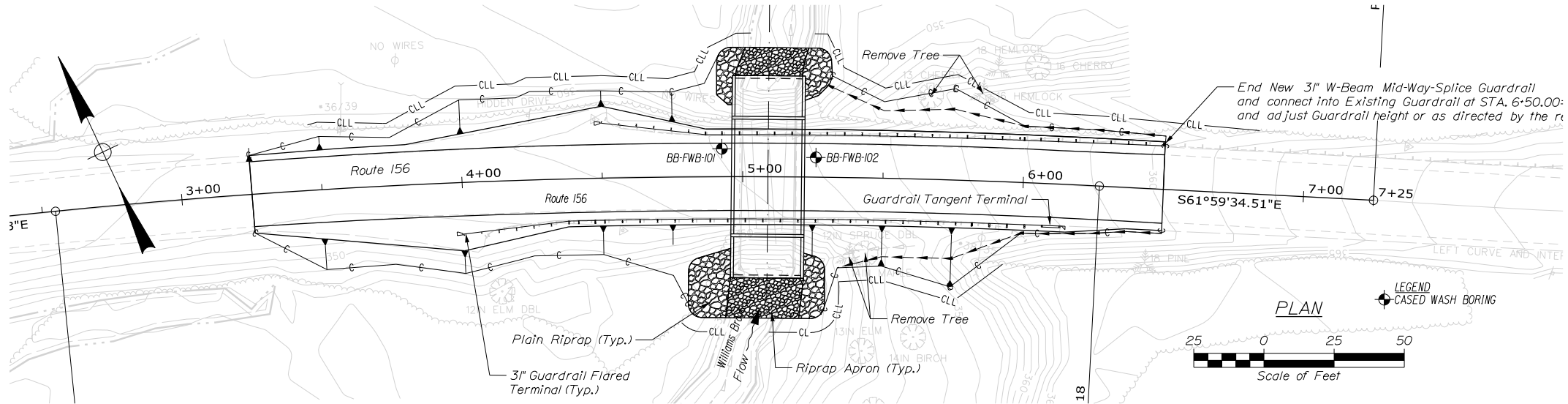


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0.25 Miles
1 inch = 0.28 miles

Date: 1/9/2019
Time: 12:27:00 PM

SHEET NUMBER 1 OF 3	WILLIAMS #1 BRIDGE WILLIAMS BROOK FARMINGTON FRANKLIN COUNTY	STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
		STP-2169(000)	
	LOCATION MAP	WIN BRIDGE NO. 2980 21690.00 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	
STP-2169(000)		BRIDGE NO. 2990	
WIN		21690.00	
BRIDGE PLANS			
WILLIAMS #1 BRIDGE		DATE	
WILLIAMS BROOK		BY	
FRANKLIN COUNTY		D. EATON	
FARMINGTON		J. LEAVITT	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE		12-10-2018	
SHEET NUMBER		SIGNATURE	
2		T. WHITE	
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		DATE	
		FEB 2018	
		REVISIONS 1	
		REVISIONS 2	
		REVISIONS 3	
		REVISIONS 4	
		FIELD CHANGES	

Maine Department of Transportation										Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine										Boring No.: BB-FWB-101																			
Self/Book Estimation Log										US CUSTOMARY UNITS										WIN: 21690.00																			
Drillers: Northern Test Boring										Elevation (ft.): 354.1										Auger ID/OD: 2.25-6.25"																			
Operator: NAD088										Datum										Sampler: Standard Split Spoon																			
Logged By: A. Van Buskirk										Rig Type: Dierffon D-50										Home Wt./Fall: 140#/30"																			
Date Start/Finish: 2/15/2018-2/16/2018										Drilling Method: HSA/Coated Wash Boring										Core Barrel: N/A																			
Boring Location: 4+92.6, 10.0 ft L.										Coating ID/OD: H&4"										Water Level: 11.6 ft bgs.																			
Home Efficiency Factor: 0.377										Home Type: Automatic @ Hydraulic @ Rope & Cathead @										Rope & Cathead @																			
Drift/Notes: 2 - Split Spoon Sample										K = Max Core Sample										K = Max Core Sample										K = Max Core Sample									
M = Unconsolidated Split Spoon Sample Average										L = Lab Vane Undrained Shear Strength (psi)										L = Lab Vane Undrained Shear Strength (psi)										L = Lab Vane Undrained Shear Strength (psi)									
N = In Situ Vane Shear										H = Moisture Content										H = Moisture Content										H = Moisture Content									
W = Unconsolidated In Situ Vane Sample Average										W = Weight of Hole, number										W = Weight of Hole, number										W = Weight of Hole, number									
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N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test									
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N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test									
M = Unconsolidated Fluid Vane Shear Test										M = Unconsolidated Fluid Vane Shear Test										M = Unconsolidated Fluid Vane Shear Test										M = Unconsolidated Fluid Vane Shear Test									
N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test										N = Unconsolidated Fluid Vane Shear Test									
M = Unconsolidated Fluid Vane Shear Test										M = Unconsolidated Fluid Vane Shear Test										M = Unconsolidated Fluid Vane Shear Test										M									

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.	<u>Coarse-grained soils</u> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).			
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		Standard Penetration Resistance N-Value (blows per foot)		
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Very loose	0 - 4			
				Loose	5 - 10			
				Medium Dense	11 - 30			
	SILTS AND CLAYS (liquid limit greater than 50)			Dense	31 - 50			
				Very Dense	> 50			
		OH	SM	Silty sands, sand-silt mixtures	<u>Fine-grained soils</u> (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.			
			SC	Clayey sands, sand-clay mixtures.				
	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)		
<u>Desired Soil Observations (in this order, if applicable):</u> 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					Very Soft WOH, WOR, WOP, <2 0 - 250 Fist easily penetrates			
					Soft 2 - 4 250 - 500 Thumb easily penetrates			
<u>Desired Rock Observations (in this order, if applicable):</u> 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))					Medium Stiff 5 - 8 500 - 1000 Thumb penetrates with moderate effort			
					Stiff 9 - 15 1000 - 2000 Indented by thumb with great effort			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Very Stiff 16 - 30 2000 - 4000 Indented by thumbnail			
					Hard >30 over 4000 Indented by thumbnail with difficulty			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					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			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Rock Mass Quality RQD (%)			
					Very Poor ≤25			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Poor 26 - 50			
					Fair 51 - 75			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Good 76 - 90			
					Excellent 91 - 100			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					<u>Desired Rock Observations (in this order, if applicable):</u>			
					Color (Munsell color chart)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Texture (aphanitic, fine-grained, etc.)			
					Rock Type (granite, schist, sandstone, etc.)			
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					-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-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)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-infilling (grain size, color, etc.)			
					Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					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			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Recovery (inch/inch and percentage)			
					Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					<u>Desired Rock Observations (in this order, if applicable):</u>			
					Color (Munsell color chart)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Texture (aphanitic, fine-grained, etc.)			
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<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					<u>Desired Rock Observations (in this order, if applicable):</u>			
					Color (Munsell color chart)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Texture (aphanitic, fine-grained, etc.)			
					Rock Type (granite, schist, sandstone, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Hardness (very hard, hard, mod. hard, etc.)			
					Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					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.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-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)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-infilling (grain size, color, etc.)			
					Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					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			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Recovery (inch/inch and percentage)			
					Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					<u>Desired Rock Observations (in this order, if applicable):</u>			
					Color (Munsell color chart)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Texture (aphanitic, fine-grained, etc.)			
					Rock Type (granite, schist, sandstone, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					Hardness (very hard, hard, mod. hard, etc.)			
					Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					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.)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-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)			
<u>Sample Container Labeling Requirements:</u> WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth					-infilling (grain size, color, etc.)			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-101 WIN: 21690.00	
Driller: Northern Test Boring				Elevation (ft.) 354.1		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		Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 2/15/2018-2/16/2018				Drilling Method: HSA/Cased Wash Boring		Core Barrel: N/A			
Boring Location: 4+92.6, 10.0 ft Lt.				Casing ID/OD: HW-4"		Water Level*: 11.6 ft bgs.			
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	353.5		7" HMA.	G#271134 A-1-b, SW-SM WC=7.1%	
5	1D	24/11	5.00 - 7.00	37/22/12/6	34	55				Brown, damp, very dense, SAND, some gravel, trace silt, (Fill).		
10	2D	24/11	10.00 - 12.00	4/7/7/6	14	23	43			Brown, damp, medium dense, SAND, some gravel, little silt, (Fill).		G#271135 A-4, ML WC=38.0% LL=33 PL=24 PI=9
							40					
							40					
							62					
15	3D	24/24	15.00 - 17.00	WOH/WOH/1/2	1	2		339.1		a14 blows for 0.6 ft. Hydraulic Push ahead from 14.5-19.5 ft bgs.		
										Grey, wet, soft, Clayey SILT, trace sand, (Glaciomarine).		
	MV		18.00 - 18.10	Would not Push				336.1		Failed 55x110 mm vane attempt, would not push.		
20	MV/4D	24/5	20.00 - 22.00	7/7/7/9	14	23			b30 blows for 0.5 ft. c31 blows for 0.5 ft. Failed 55x110 mm vane attempt, would not push.			
									Grey, wet, very stiff, Sandy SILT, little gravel, wood fibers.			
25												

Remarks:
 Auto Hammer #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-FWB-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-101 WIN: 21690.00			
Driller: Northern Test Boring				Elevation (ft.): 354.1				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				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 2/15/2018-2/16/2018				Drilling Method: HSA/Cased Wash Boring				Core Barrel: N/A			
Boring Location: 4+92.6, 10.0 ft Lt.				Casing ID/OD: HW-4"				Water Level*: 11.6 ft bgs.			
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 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				
25	5D	24/6	25.00 - 27.00	5/7/7/7	14	23				Grey, wet, very stiff, SILT, some sand, little clay, trace gravel, (Glacial Till).	G#271136 A-4, CL WC=13.7% Non-Plastic
30	6D	24/11	30.00 - 32.00	11/11/10/12	21	34				Grey, wet, hard, SILT, little sand, little gravel, little clay, (Glacial Till).	G#271137 A-4, CL WC=13.3% Non-Plastic
35	7D	24/9	35.00 - 37.00	10/8/11/14	19	31				Similar to 6D.	
40	8D	16.8/9	40.00 - 41.40	17/43/50(4.8")	---					Grey, wet, SILT, some sand, little gravel, (Glacial Till).	G#271138 A-4, CL WC=11.3%
45	9D	22.8/18	45.00 - 46.90	17/26/34/50(4.8)	60	98				Grey, wet, hard, SILT, little sand, little clay, trace gravel, (Glacial Till).	
50											
Remarks: Auto Hammer #377											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 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-FWB-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-101 WIN: 21690.00			
Driller: Northern Test Boring				Elevation (ft.): 354.1				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				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 2/15/2018-2/16/2018				Drilling Method: HSA/Cased Wash Boring				Core Barrel: N/A			
Boring Location: 4+92.6, 10.0 ft Lt.				Casing ID/OD: HW-4"				Water Level*: 11.6 ft bgs.			
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				
50	10D	24/19	50.00 - 52.00	23/33/38/75	71	116		302.1		Grey, hard, fine Sandy Silt, (Glacial Till).	
										Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL	
55											
60											
65											
70											
75											
Remarks: Auto Hammer #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-FWB-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-102 WIN: 21690.00	
Driller: Northern Test Boring			Elevation (ft.): 355.0			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			Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 2/15/2018-2/16/2018			Drilling Method: HSA/Cased Wash Boring			Core Barrel: N/A			
Boring Location: 5+26.2, 7.1 ft Lt.			Casing ID/OD: HW-4"			Water Level*: 10.0 ft bgs.			
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	354.7		4" HMA.	0.3
	1D	2.4/1	2.50 - 2.70	50(2.4")	---					Brown, dry, fine to medium SAND, trace silt, trace gravel, (Fill).	
5	2D	24/22	5.00 - 7.00	14/2/2/1	4	7				Brown, moist, medium stiff, Sandy SILT, trace gravel, (Fill).	G#271139 A-4, CL WC=24.2%
10	3D	24/22	10.00 - 12.00	2/1/2/1	3	5		344.7		Brown, wet, loose, Silty fine SAND, trace organics, (Stream Alluvium).	10.3
15	4D	24/24	15.00 - 17.00	1/WOR/1/WOR	1	2	HYD PUSH	340.0		Grey, wet, soft, Clayey SILT, trace sand, (Glaciomarine).	G#271140 A-4, CL WC=38.0% LL=32 PL=23 PI=9
	V1		17.00 - 17.37	Su=491/89 psf						55x110 mm raw torque readings: V1: 11.0/2.0 ft-lbs V2: 11.0/2.0 ft-lbs	
	V2		18.00 - 18.37	Su=491/89 psf							
20	V3 5D/A MV	24/24	20.00 - 20.37 20.00 - 22.00 21.00 - 21.20	Su=737/156 psf WOH/WOH/4/4 Would not Push	4	7	OPEN HOLE	334.0		55x110 mm vane raw torque readings: V3: 16.5/3.5 ft-lbs 5D (20.0-21.0 ft bgs) Grey, wet, medium stiff, SILT, some clay, trace sand, trace gravel.	G#271141 A-4, CL WC=33.1% Non-Plastic
										5D/A 21.0-22.0 ft bgs) Grey, wet, loose, Silty fine to medium SAND, trace gravel. Failed 55x110 mm vane attempt, would not push.	21.0
25											

Remarks:
 Auto Hammer #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-FWB-102

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-102 WIN: 21690.00			
Driller: Northern Test Boring			Elevation (ft.): 355.0			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			Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 2/15/2018-2/16/2018			Drilling Method: HSA/Cased Wash Boring			Core Barrel: N/A					
Boring Location: 5+26.2, 7.1 ft Lt.			Casing ID/OD: HW-4"			Water Level*: 10.0 ft bgs.					
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 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				
25	6D	24/7	25.00 - 27.00	4/5/6/6	11	18				Similar to 5D/A, except medium dense with little gravel.	
30	7D	24/5	30.00 - 32.00	5/7/8/11	15	24				Grey, moist, very stiff, fine Sandy SILT, (Glacial Till).	
35	8D	24/10	35.00 - 37.00	10/18/23/25	41	67				Grey, moist, hard, SILT, little sand, little gravel, little clay, (Glacial Till).	G#271142 A-4, CL WC=12.9% Non-Plastic
40	9D	24/11	40.00 - 42.00	8/13/15/13	28	46				Similar to 8D, (Glacial Till).	
45	10D	24/20	45.00 - 47.00	19/26/30/35	56	91				Grey, moist, hard, SILT, little sand, trace gravel, (Glacial Till).	G#271143 A-4, CL WC=14.9%
50											
Remarks: Auto Hammer #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 2 of 3 Boring No.: BB-FWB-102	

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Williams #1 Bridge #2980 carries Route 156 over Williams Brook Location: Farmington, Maine				Boring No.: BB-FWB-102 WIN: 21690.00																																																																																																																																																																		
Driller: Northern Test Boring				Elevation (ft.): 355.0				Auger ID/OD: 2.25-6.25"																																																																																																																																																																		
Operator: Nadeau/Taylor				Datum: NAVD88				Sampler: Standard Split Spoon																																																																																																																																																																		
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Date Start/Finish: 2/15/2018-2/16/2018				Drilling Method: HSA/Cased Wash Boring				Core Barrel: N/A																																																																																																																																																																		
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Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected																																																																																																																																																																		
T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																										
<table border="1"><thead><tr><th rowspan="2">Depth (ft.)</th><th colspan="7">Sample Information</th><th rowspan="2">Elevation (ft.)</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></tr></thead><tbody><tr><td>50</td><td>11D</td><td>24/20</td><td>50.00 - 52.00</td><td>21/24/28/37</td><td>52</td><td>85</td><td></td><td rowspan="16">303.0</td><td rowspan="16"></td><td rowspan="16">Grey, moist, hard, SILT, little sand, trace gravel, (Glacial Till). Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL</td><td rowspan="16"></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><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><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>75</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></tbody></table>												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	50	11D	24/20	50.00 - 52.00	21/24/28/37	52	85		303.0		Grey, moist, hard, SILT, little sand, trace gravel, (Glacial Till). Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL																																																																																																																										75							
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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-FWB-102																																																																																																																																																																

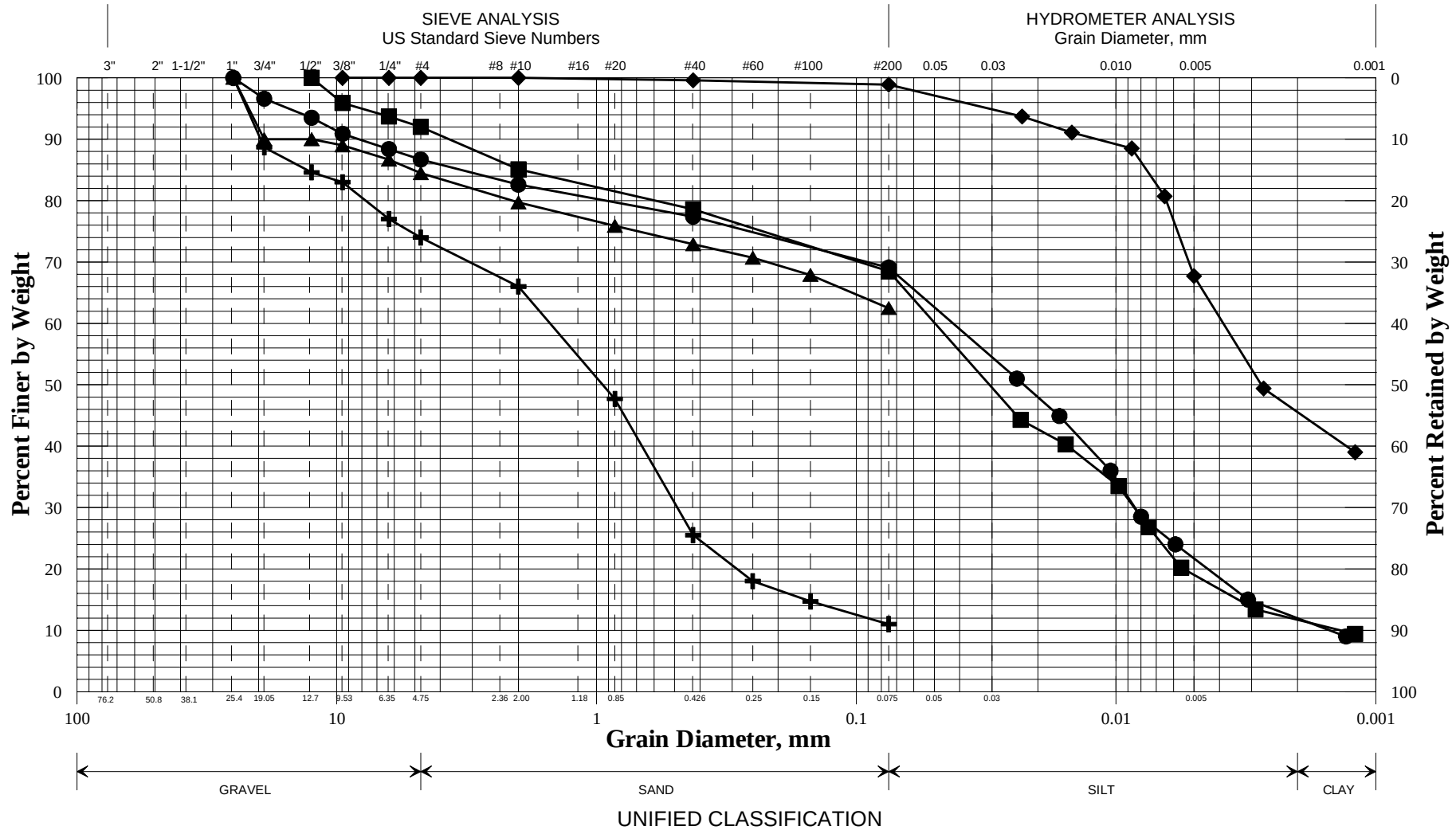
Appendix B

Laboratory Test Results

Work Number: 21690.00

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

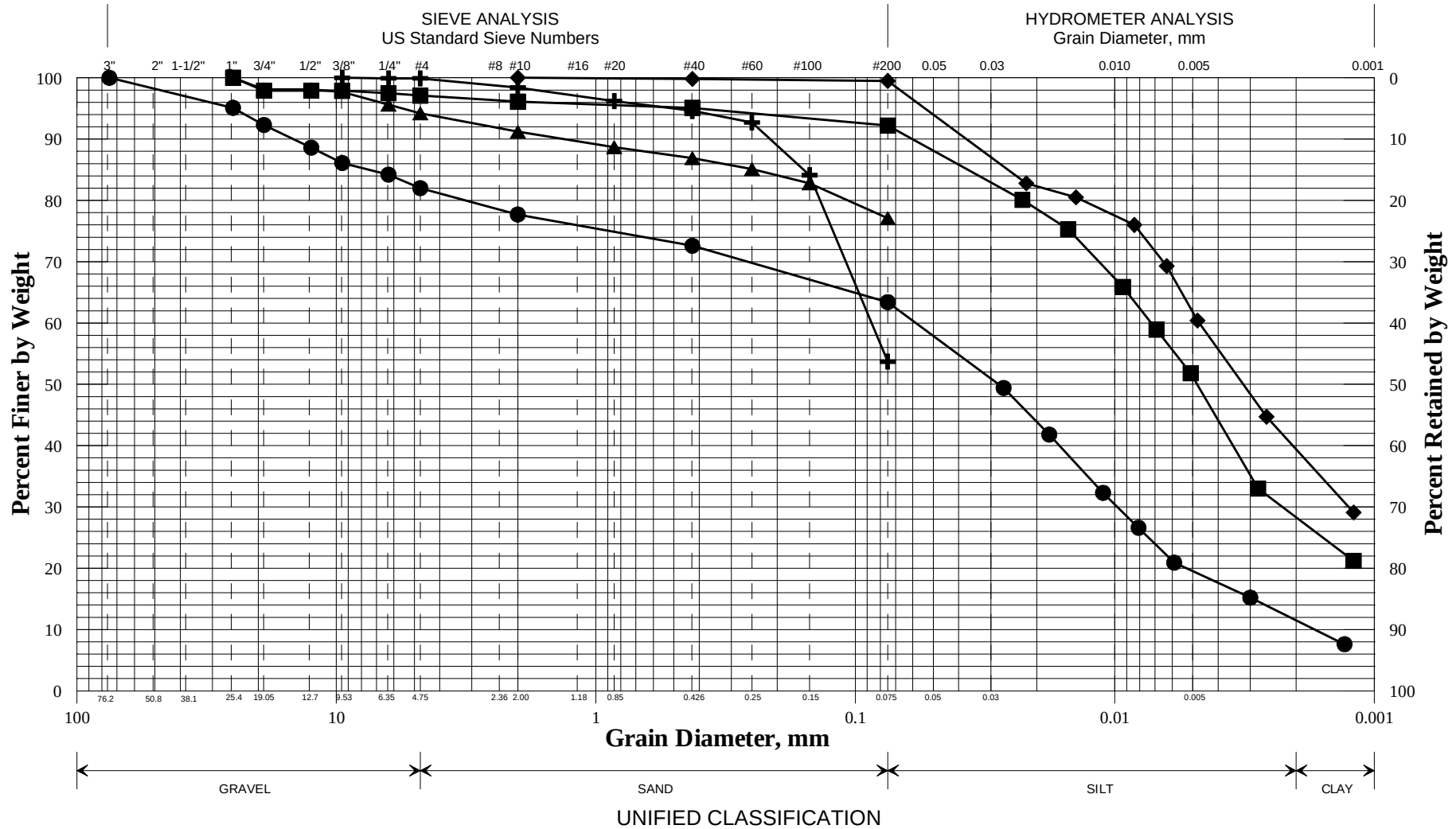
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-FWB-101/2D	4+92.6	10.0 LT	10.0-12.0	SAND, some gravel, little silt.	7.1			
◆	BB-FWB-101/3D	4+92.6	10.0 LT	15.0-17.0	Clayey SILT, trace sand.	38.0	33	24	9
■	BB-FWB-101/5D	4+92.6	10.0 LT	25.0-27.0	SILT, some sand, little clay, trace gravel.	13.7			NP
●	BB-FWB-101/6D	4+92.6	10.0 LT	30.0-32.0	SILT, little sand, little gravel, little clay.	13.3			NP
▲	BB-FWB-101/8D	4+92.6	10.0 LT	40.0-41.4	SILT, some sand, little gravel.	11.3			
×									

WIN	
021690.00	
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-FWB-102/2D	5+26.2	7.1 LT	5.0-7.0	Sandy SILT, trace gravel.	24.2			
◆	BB-FWB-102/4D	5+26.2	7.1 LT	15.0-17.0	Clayey SILT, trace sand.	38.0	32	23	9
■	BB-FWB-102/5DA	5+26.2	7.1 LT	20.0-21.0	SILT, some clay, trace sand, trace gravel.	33.1			NP
●	BB-FWB-102/8D	5+26.2	7.1 LT	35.0-37.0	SILT, little sand, little gravel, little clay.	12.9			NP
▲	BB-FWB-102/10D	5+26.2	7.1 LT	45.0-47.0	SILT, little sand, trace gravel.	14.9			
×									

WIN	
021690.00	
Town	
Farmington	
Reported by/Date	
WHITE, TERRY A	3/28/2018



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271135** Boring No./Sample No. **BB-FWB-101/3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **4+92.6** Offset, ft: **10.0** LT Dbfg, ft: **15.0-17.0**

WIN/Town **021690.00 - 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]	100.0
¼ in. [6.3 mm]	100.0
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	98.9
[0.0230 mm]	93.7
[0.0148 mm]	91.1
[0.0087 mm]	88.5
[0.0065 mm]	80.7
[0.0050 mm]	67.7
[0.0027 mm]	49.4
[0.0012 mm]	39.0

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	33
Plastic Limit (T 90), %	24
Plasticity Index (T 90), %	9
Specific Gravity, Corrected to 20°C (T 100)	2.64
Loss on Ignition, % (T 267)	
Water Content (T 265), %	38.0

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:

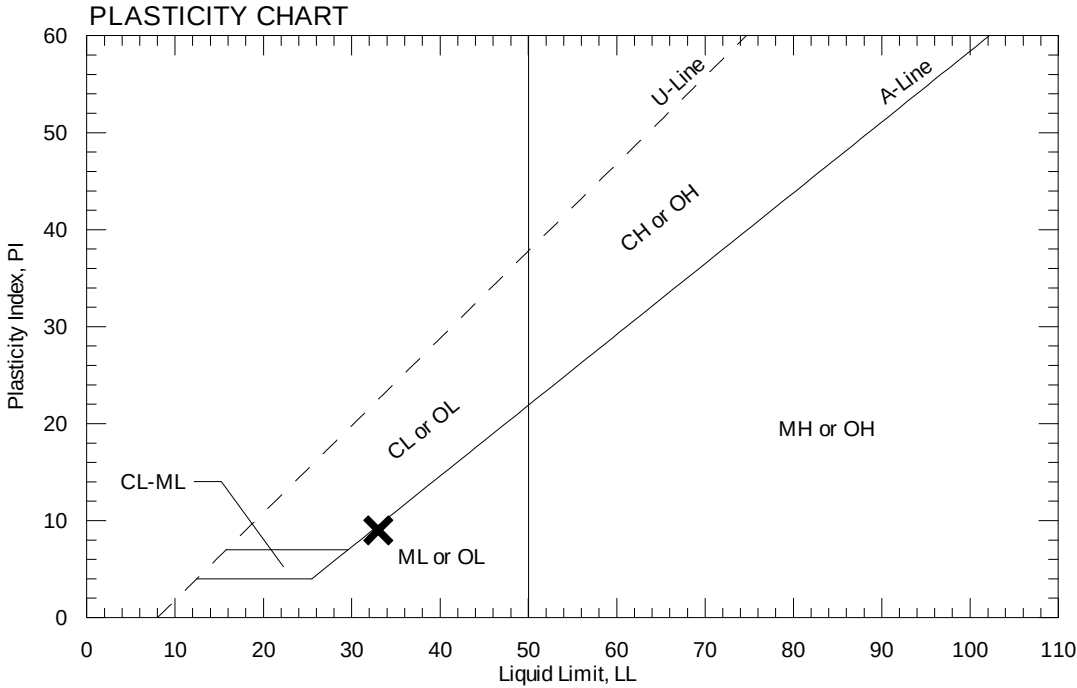
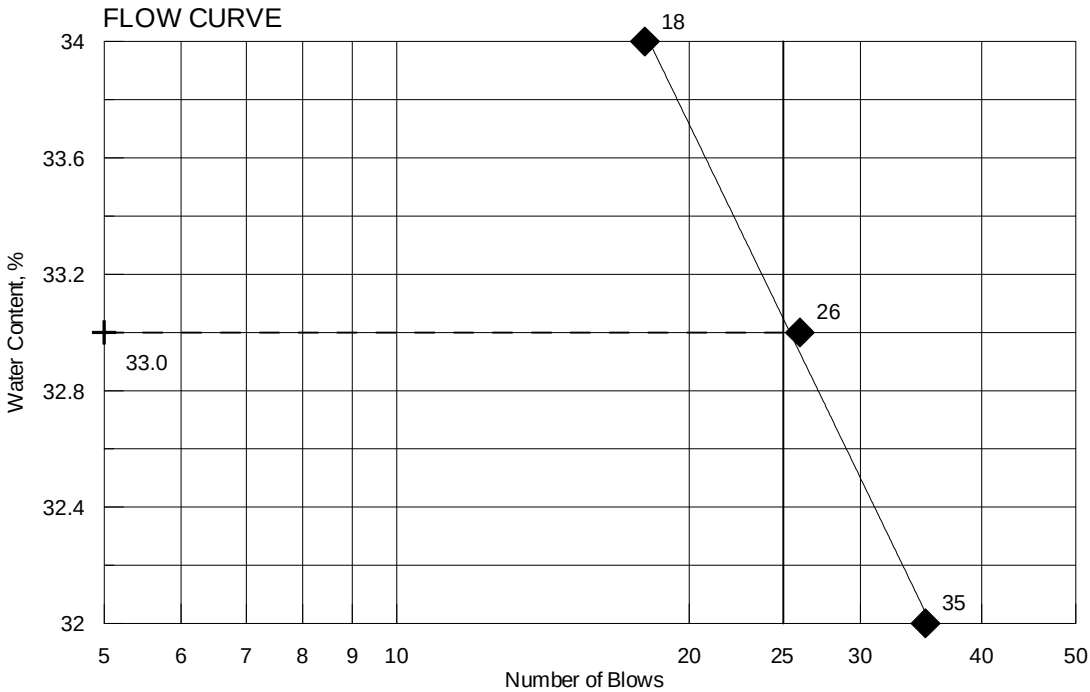
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

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

Paper Copy: Lab File; Project File; Geotech File

TOWN	Farmington	Reference No.	271135
WIN	021690.00	Water Content, %	38
Sampled	2/12/2018	Liquid Limit @ 25 blows (T 89), %	33
Boring No./Sample No.	BB-FWB-101/3D	Plastic Limit (T 90), %	24
Station	4+92.6	Plasticity Index (T 90), %	9
Depth	15.0-17.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271136** Boring No./Sample No. **BB-FWB-101/5D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **4+92.6** Offset, ft: **10.0** LT Dbfg, ft: **25.0-27.0**

WIN/Town **021690.00 - 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]	100.0
⅜ in. [9.5 mm]	95.9
¼ in. [6.3 mm]	93.7
No. 4 [4.75 mm]	92.0
No. 10 [2.00 mm]	85.1
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	78.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	68.5
[0.0233 mm]	44.3
[0.0156 mm]	40.3
[0.0098 mm]	33.5
[0.0075 mm]	26.8
[0.0056 mm]	20.2
[0.0029 mm]	13.4
[0.0012 mm]	9.4

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)	2.79
Loss on Ignition, % (T 267)	
Water Content (T 265), %	13.7

Consolidation (T 216)

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

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

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271137** Boring No./Sample No. **BB-FWB-101/6D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **4+92.6** Offset, ft: **10.0** LT Dbfg, ft: **30.0-32.0**

WIN/Town **021690.00 - 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]	100.0
¾ in. [19.0 mm]	96.6
½ in. [12.5 mm]	93.5
⅜ in. [9.5 mm]	90.9
¼ in. [6.3 mm]	88.4
No. 4 [4.75 mm]	86.7
No. 10 [2.00 mm]	82.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	77.4
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	69.1
[0.0241 mm]	51.0
[0.0165 mm]	44.9
[0.0105 mm]	36.0
[0.0080 mm]	28.5
[0.0059 mm]	24.0
[0.0031 mm]	15.0
[0.0013 mm]	9.0

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)	2.62
Loss on Ignition, % (T 267)	
Water Content (T 265), %	13.3

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:

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271140** Boring No./Sample No. **BB-FWB-102/4D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **5+26.2** Offset, ft: **7.1** LT Dbfg, ft: **15.0-17.0**

WIN/Town **021690.00 - 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]	99.5
[0.0219 mm]	82.8
[0.0141 mm]	80.5
[0.0084 mm]	76.0
[0.0063 mm]	69.3
[0.0048 mm]	60.4
[0.0026 mm]	44.7
[0.0012 mm]	29.1

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	32
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	9
Specific Gravity, Corrected to 20°C (T 100)	2.77
Loss on Ignition, % (T 267)	
Water Content (T 265), %	38.0

Consolidation (T 216)

Trimming, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

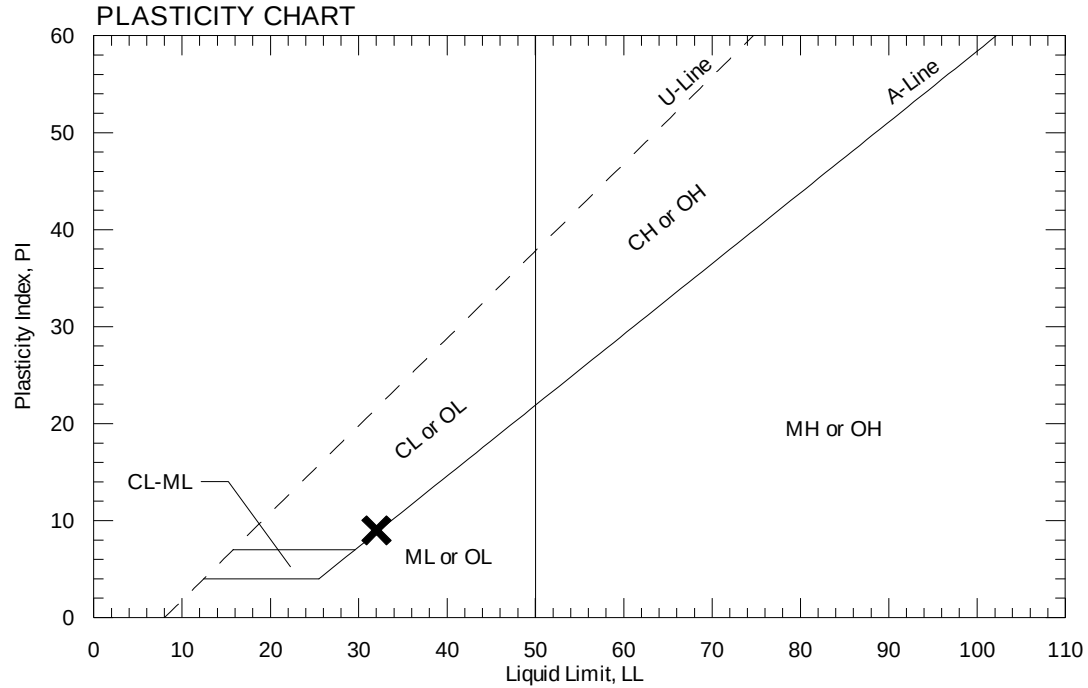
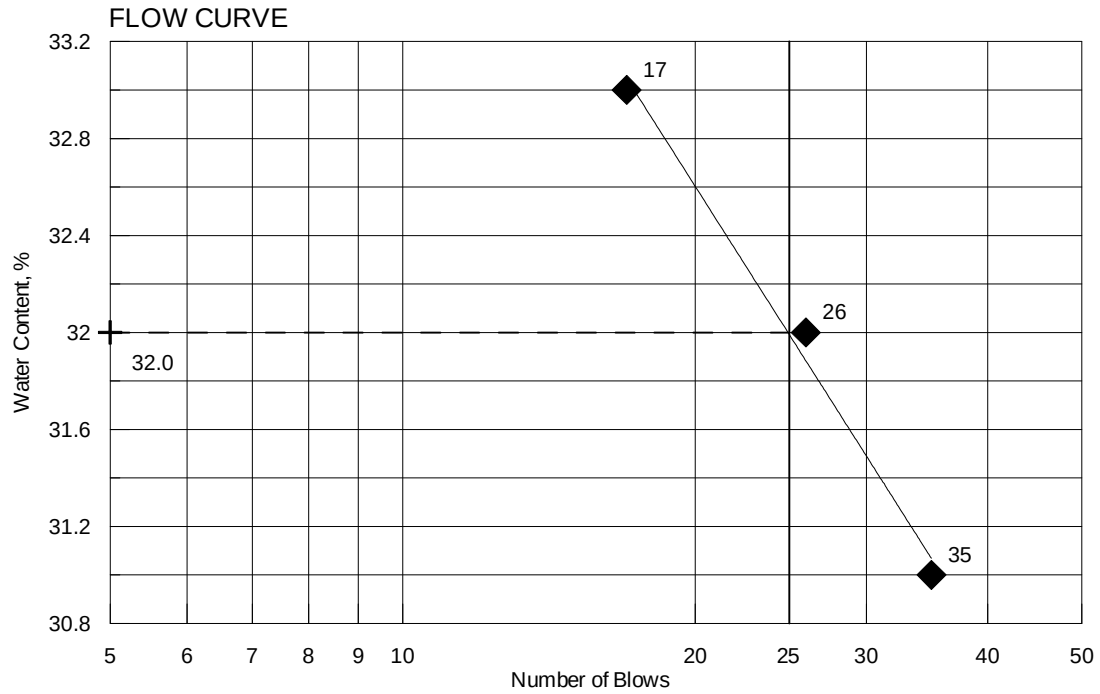
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Reported by: **GREGORY LIDSTONE**

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

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TOWN	Farmington	Reference No.	271140
WIN	021690.00	Water Content, %	38
Sampled	2/12/2018	Liquid Limit @ 25 blows (T 89), %	32
Boring No./Sample No.	BB-FWB-102/4D	Plastic Limit (T 90), %	23
Station	5+26.2	Plasticity Index (T 90), %	9
Depth	15.0-17.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271141** Boring No./Sample No. **BB-FWB-102/5D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **5+26.2** Offset, ft: **7.1** LT Dbfg, ft: **20.0-22.0**

WIN/Town **021690.00 - 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]	100.0
¾ in. [19.0 mm]	97.9
½ in. [12.5 mm]	97.9
⅜ in. [9.5 mm]	97.9
¼ in. [6.3 mm]	97.5
No. 4 [4.75 mm]	97.1
No. 10 [2.00 mm]	96.1
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	95.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	92.2
[0.0227 mm]	80.1
[0.0151 mm]	75.3
[0.0093 mm]	65.9
[0.0069 mm]	58.9
[0.0051 mm]	51.8
[0.0028 mm]	33.0
[0.0012 mm]	21.2

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.78
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 tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

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

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **271142** Boring No./Sample No. **BB-FWB-102/8D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **2/12/2018** Received **2/28/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **5+26.2** Offset, ft: **7.1** LT Dbfg, ft: **35.0-37.0**

WIN/Town **021690.00 - FARMINGTON** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	95.1
¾ in. [19.0 mm]	92.3
½ in. [12.5 mm]	88.6
⅜ in. [9.5 mm]	86.1
¼ in. [6.3 mm]	84.2
No. 4 [4.75 mm]	82.0
No. 10 [2.00 mm]	77.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	72.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	63.4
[0.0268 mm]	49.4
[0.0179 mm]	41.8
[0.0111 mm]	32.3
[0.0081 mm]	26.6
[0.0059 mm]	20.9
[0.0030 mm]	15.2
[0.0013 mm]	7.6

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)	2.73
Loss on Ignition, % (T 267)	
Water Content (T 265), %	12.9

Consolidation (T 216)

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **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 cantilever walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 1.75H:1V backfill

β = Angle of fill slope to the horizontal $\beta := 29.74 \cdot \text{deg}$

$$K_{ar_slope} := \cos(\beta) \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \quad K_{ar_slope} = 0.56$$

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

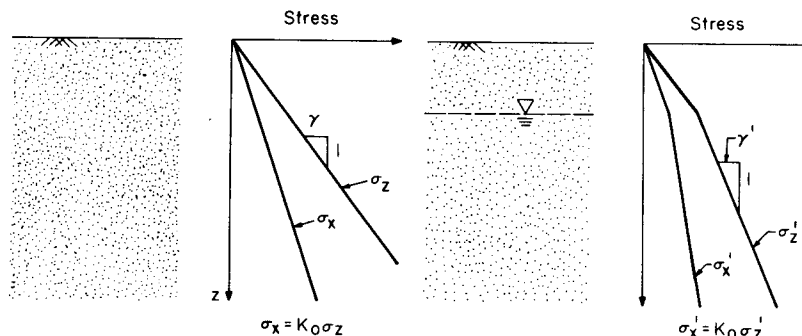


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

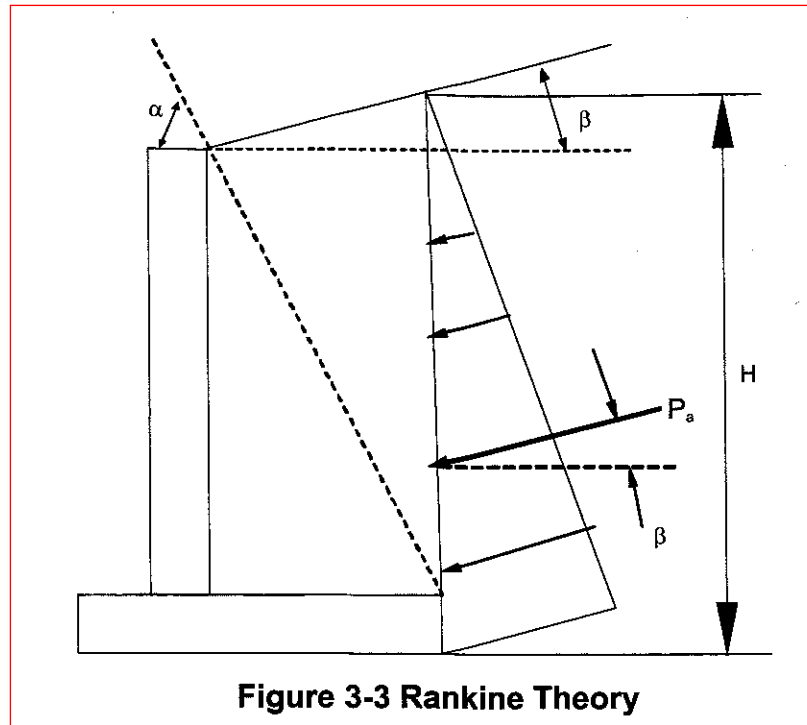


Figure 3-3 Rankine Theory

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 strength properties based on correlations to SPT N-values.
3. Undrained shear strength from vane shear tests or correlations to N-values.

Assumptions:

1. The box culvert's embedment into the streambed is conservatively assumed as 2 feet. Include 2 foot crushed stone mat as Foundation Footing, therefore Foundation Depth is 4.0 feet
2. Crushed stone considered part of base slab
3. The proposed bearing elevation slopes from El. 339 at inlet to 338 at outlet .
4. Bearing elevation of crushed stone mat is El. 337 at inlet and 336 at outlet.
5. Proposed finish roadway grade elevation is approximately 354 feet.
6. Proposed precast concrete box has a span of 24 feet and a base of 26 feet wide. Use dimensions of geogrid reinforced crushed stone mat as the Foundation Width, $26 + 3 = 29$ feet
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
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Soft	1-2	1.5

The Marine Clay (ML) deposit is soft to medium stiff in consistency. Recommend 1.5 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Assumed Foundation Width, Depth, and Water Surface

$$B := 29\text{ft}$$

$$D_f := 4.0\text{ft}$$

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

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

Foundation soils:

$$\gamma_{1d} := 83 \cdot \text{pcf}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2
Soft Clay

$$w_{\text{sat}} := .38$$

Moisture contents of saturated samples BB-FWB-101 3D and
BB-FWB-102 4D

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

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

$$\gamma_{1\text{sat}} = 114.5 \cdot \text{pcf}$$

$$\begin{aligned} \phi &:= 0 \cdot \text{deg} \\ \text{Cohesion } c_1 &:= 737 \cdot \text{psf} \end{aligned}$$

Assume excavation for stone mud mat to Elev 336 to
337 use undrained shear strength from BB-FWB-102
V3 = 737 psf

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing

$$s_\gamma := 1.0$$

$$s_c := 1.0$$

Bowles 5th Ed., p. 220 Table 4-1

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for Marine Clay $\phi = 0$

$$N_c := 5.14$$

$$N_q := 1$$

$$N_\gamma := 0$$

Nominal Bearing Resistance per Terzaghi equation

$$q := D_f \cdot (\gamma_{1\text{sat}} - \gamma_w) \quad q = 0.209 \cdot \text{ksf}$$

Das Principles of Foundation Engineering 7th Ed. p. 142:
Eq. 3.16 Water table modification

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{1\text{sat}} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

Bowles Foundation Analysis and Design 5th Ed. p. 220:
Table 4-1 Bearing-capacity Equations

$$q_n = 4 \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.50$$

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

$$q_r = 2 \cdot \text{ksf}$$

Nominal Bearing Resistance for Strength Limit States

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

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

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

$$N_c := 5.14$$

$$N_q := 1$$

$$N_\gamma := 0$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume:

$$L := 80\text{ft}$$

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

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

$$s_\gamma = 0.855$$

$$s_q = 1$$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

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

Where the depth of water is less than the depth of the footing, all water coefficients are 0.5.

$$C_{wq} := .5$$

$$C_{w\gamma} := 0.5$$

Load Inclination factors

No knowledge of vertical and horizontal loads at this time. Use 1.0

$$i_c := 1.0$$

$$i_\gamma := 1.0$$

$$i_q := 1.0$$

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

$$\frac{D_f}{B} = 0.138$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$$N_{cm} = 5.14$$

$$N_{\gamma m} = 0$$

$$N_{qm} = 1$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 4 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.50$$

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

$$q_r = 2 \cdot \text{ksf}$$

Recommend a factored bearing resistance of 2 ksf for precast box culvert constructed on Glaciomarine Deposit.

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

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.

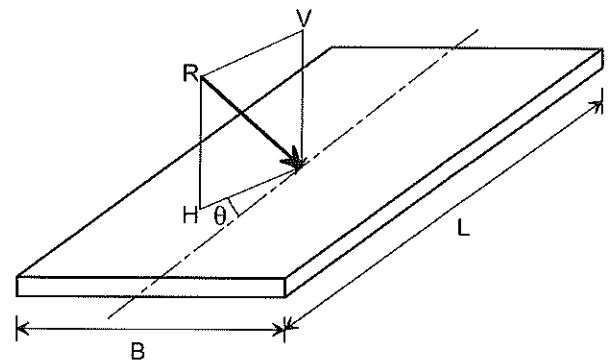


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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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
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>
		Semi-empirical methods (Meyerhof, 1957), all soils
		Footings on rock
		Plate Load Test
Sliding	ϕ_τ	Precast concrete placed on sand
		Cast-in-Place Concrete on sand
		Cast-in-Place or precast Concrete on Clay
		Soil on soil
	ϕ_{ep}	Passive earth pressure component of sliding resistance

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. Boring logs, SPT N-values, S_u from vane shear tests, lab data

Assumptions:

1. The box culvert embedded 2 feet into the streambed.
2. The two foot layer of crushed stone bedding is not considered.
3. The proposed bearing elevation of base slab slopes from Elev. 339 at inlet to 338 at outlet.
4. Proposed finished grade matches existing
5. Proposed precast concrete box is 26 feet wide and 43 feet long (excluding slab connecting slope tapered outlet and inlet sections).
6. Model subsurface conditions based on Marine Clay deposit, $S_u = 491$ to 737 psf.
7. 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 submerged, loose, sand and soft clay:

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

Range of modulus of subgrade reaction

Subgrade of Clayey soil, $q_a < 4000$ psf or 4 ksf: $k_s = 44$ pci

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

Range of modulus of subgrade reaction

Clay subgrade (min. value of curve) K_{v1} , 23 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:

Typical subgrade reaction values for 0.3 m x 0.3 m plate

Stiff clay, 37-92 pci (no value for soft clay): $k_{0.3}$ (k_1) = 30 pci

Adjust Published values for dimensions of base slab

Published range for soft, clay subgrade is 23 - 44 pci.

Assume a subgrade modulus of 32 pci for clay.

Value of $k_{s1} = 32$ pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base.

(Width B - 26 ft, Length L = 43 ft)

Square to rectangle base adjustment:

$$k_{s1} := 32 \text{ pci} \quad B := 26 \text{ ft} \quad \text{L} := 43 \text{ ft}$$

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

$$k = 27.8 \cdot \text{pci}$$

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

Recommend a subgrade modulus of 28 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 (with d_i factors dropped) to give

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

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

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

assumes 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

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

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

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

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

even as a "good" estimate.

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

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

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

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

compute

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

TABLE 9-1

Range of modulus of subgrade

reaction k_s

Use values as guide and for comparison when $\frac{kN}{M^3} \rightarrow \frac{lb}{in^3} : \frac{224.8 lb}{1 kN} * \frac{1 M^3}{61023.7 in^3} = .003684 \frac{kN}{M^3} = 1 \frac{lb}{in^3}$
using approximate equations

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

44 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

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

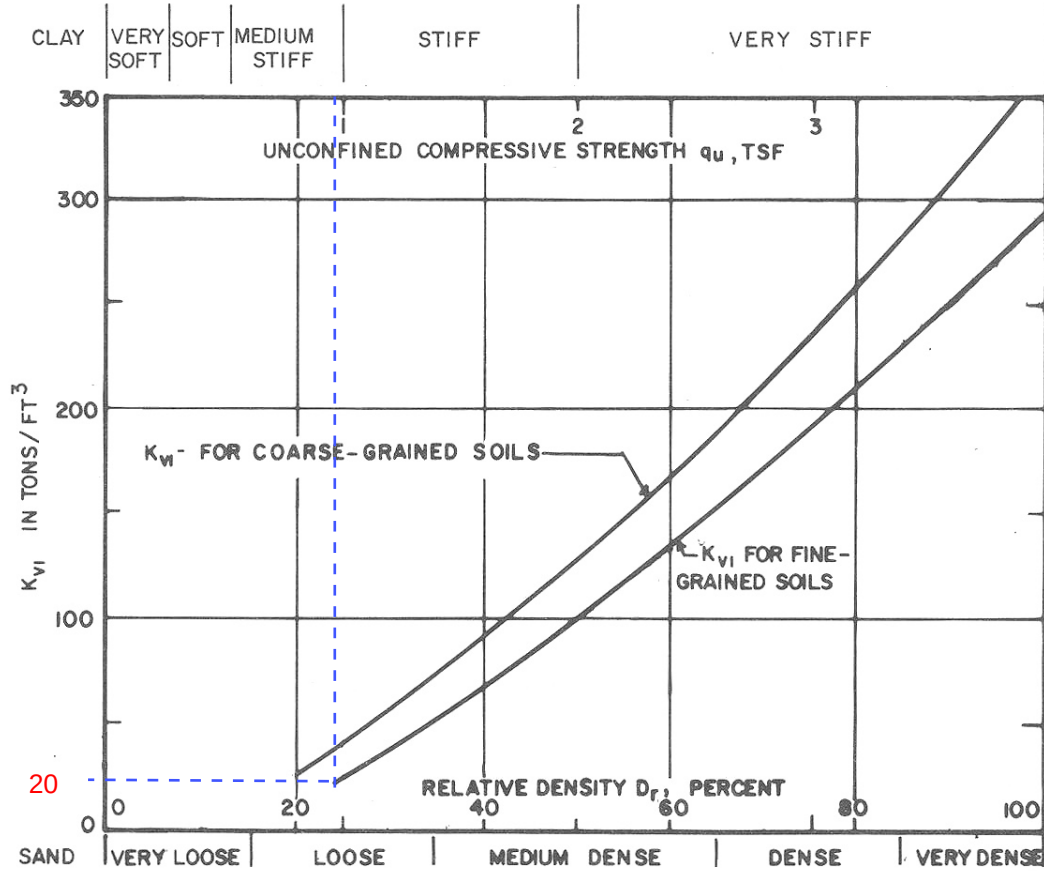
You should note that k_s does not depend on the contact pressure of the base q_o .

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)]

Clay Subgrade - lowest value per chart is 20 tcf = 23 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_{v1} = 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:

$$\Delta H_i = \frac{4 q B^2}{K_{v1} (B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2 q B^2}{K_{v1} (B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2 q B^2}{K_{v1} (B+1)^2}$$

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

2. VALUES OF K_{v1} 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 $K_{v1}/2$ 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

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

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 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 $B \times B$

Settlement

Settle3D Analysis Information

21690.00 Farmington Williams Bridge

Project Settings

Document Name	21690 Farmington Settlement_q=500_run1.s3z
Project Title	21690.00 Farmington Williams Bridge
Analysis	settlement analysis q=500
Author	A. Van Buskirk
Date Created	11/27/2018, 11:24:08 AM
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name	Time [years]
1	Immediate	0
2	Consolidated	1
3	Long Term	50

Results

Time taken to compute: 0 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.834623
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.834623
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	4.09809
Total Stress [ksf]	0	6.75094
Total Strain	0	0.00318171
Pore Water Pressure [ksf]	0	2.68047
Excess Pore Water Pressure [ksf]	0	0.5
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.00106119	4.09678
Over-consolidation Ratio	1	1.7
Void Ratio	0	0.9
Permeability [ft/y]	0	0.0116245
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.819617

Stage: Consolidated = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.917377
Consolidation Settlement [in]	0	0.0967114
Immediate Settlement [in]	0	0.834623
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	4.25624
Total Stress [ksf]	0	6.75094
Total Strain	-2.02573e-005	0.00318171
Pore Water Pressure [ksf]	0	2.52021
Excess Pore Water Pressure [ksf]	0	0.0213723
Degree of Consolidation [%]	0	99.9985
Pre-consolidation Stress [ksf]	0.00106119	4.25507
Over-consolidation Ratio	1	1.70503
Void Ratio	0	0.900038
Permeability [ft/y]	0	0.0116245
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.824753

Stage: Long Term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.29993
Consolidation Settlement [in]	0	0.0988447
Immediate Settlement [in]	0	0.834623
Secondary Settlement [in]	0	0.481771
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	4.2586
Total Stress [ksf]	0	6.75094
Total Strain	0	0.0110022
Pore Water Pressure [ksf]	0	2.50195
Excess Pore Water Pressure [ksf]	-2.89575e-018	4.20406e-018
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.00106119	4.25743
Over-consolidation Ratio	1	1.7
Void Ratio	0	0.9
Permeability [ft/y]	0	0.0116245
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.824753

Loads

1. Rectangular Load

Length	73 ft
Width	26 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1898 ft ²
Load	0.5 ksf
Depth	15.6 ft
Installation Stage	Immediate = 0 y

Coordinates

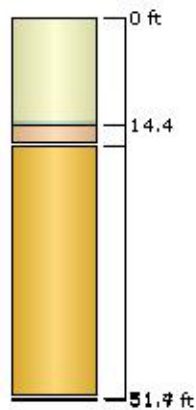
X [ft]	Y [ft]
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40.638	4.512
40.638	30.512
-32.362	30.512

Soil Layers

Ground Surface Drained: Yes

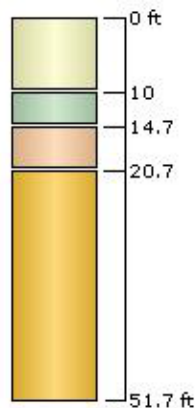
BB-FWB-101: (-6.2, 0.75)

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Silty Sand	14.4	0	No
2	Saturated Alluvium	0	14.4	No
3	Soft Clayey Silt	3	14.4	No
4	Very Stiff Sandy Silt	34	17.4	No



BB-FWB-102: (-3.7, 34.3)

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Silty Sand	10	0	No
2	Saturated Alluvium	4.7	10	No
3	Soft Clayey Silt	6	14.7	No
4	Very Stiff Sandy Silt	31	20.7	No



Soil Properties

Property	Loose Silty Sand	Saturated Alluvium	Soft Clayey Silt	Very Stiff Sandy Silt
Color				
Unit Weight [kips/ft ³]	0.102	0.102	0.083	0.121
Saturated Unit Weight [kips/ft ³]	0.117	0.117	0.115	0.139
Immediate Settlement	Disabled	Disabled	Disabled	Enabled
Es [ksf]				156
Esur [ksf]				624
Primary Consolidation	Disabled	Disabled	Enabled	Disabled
Material Type			Non-Linear	
Cc			0.3	
Cr			0.03	
e0			0.9	
OCR	1	1	1.7	1
Cv [ft ² /y]			36.5	
B-bar			1	
Secondary Consolidation	Disabled	Disabled	Standard	Disabled
Cae/Ca			0.01	
Car/Care			0.01	
Undrained Su A [kips/ft ²]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method Piezometric Lines
 Water Unit Weight 0.0624 kips/ft³

Piezometric Line Entities

ID	Depth (ft)
1	11.6 ft

Query Points

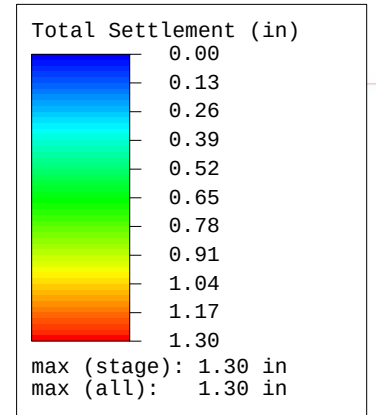
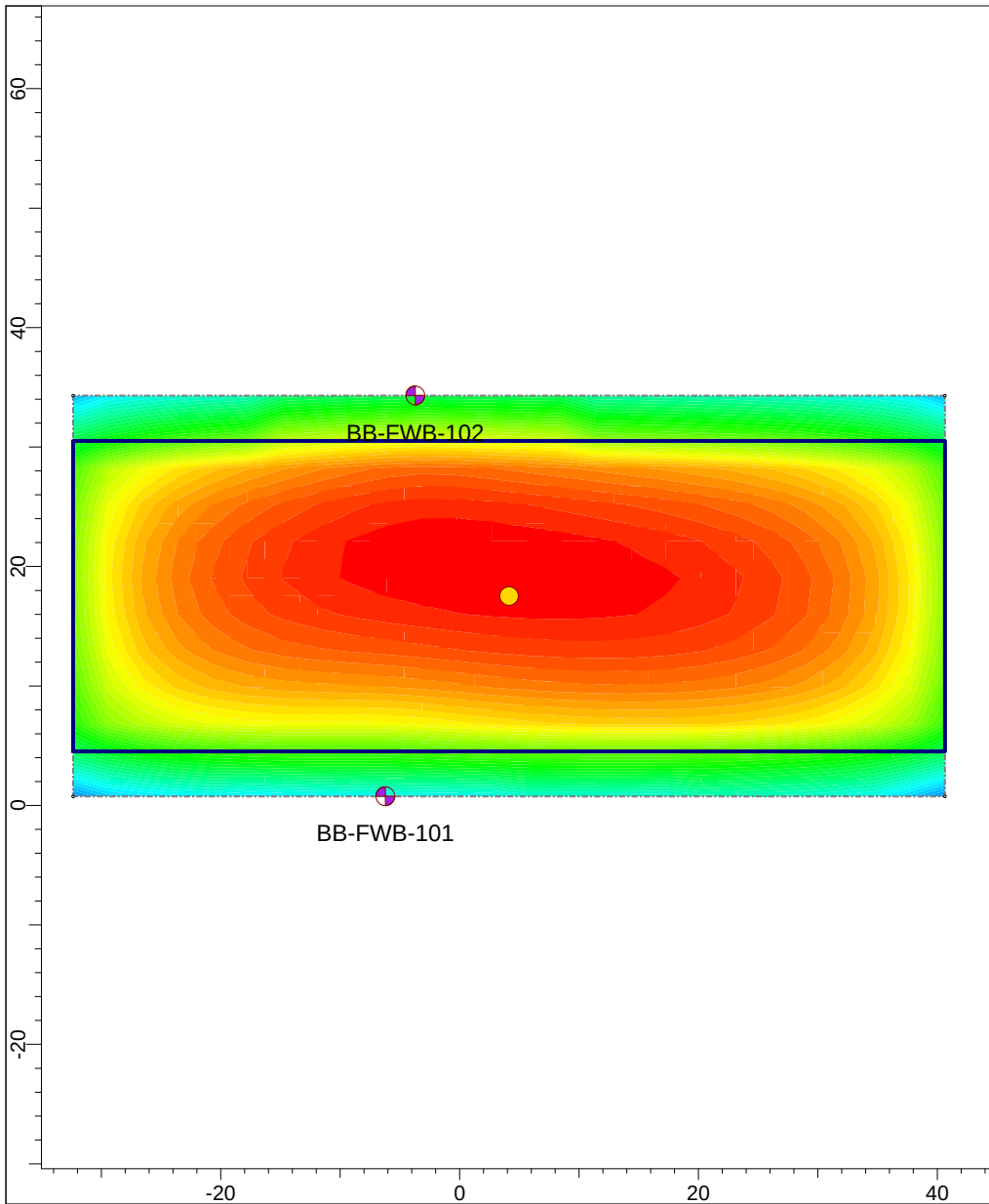
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Field Point Grid

Number of points 312
 Expansion Factor 1

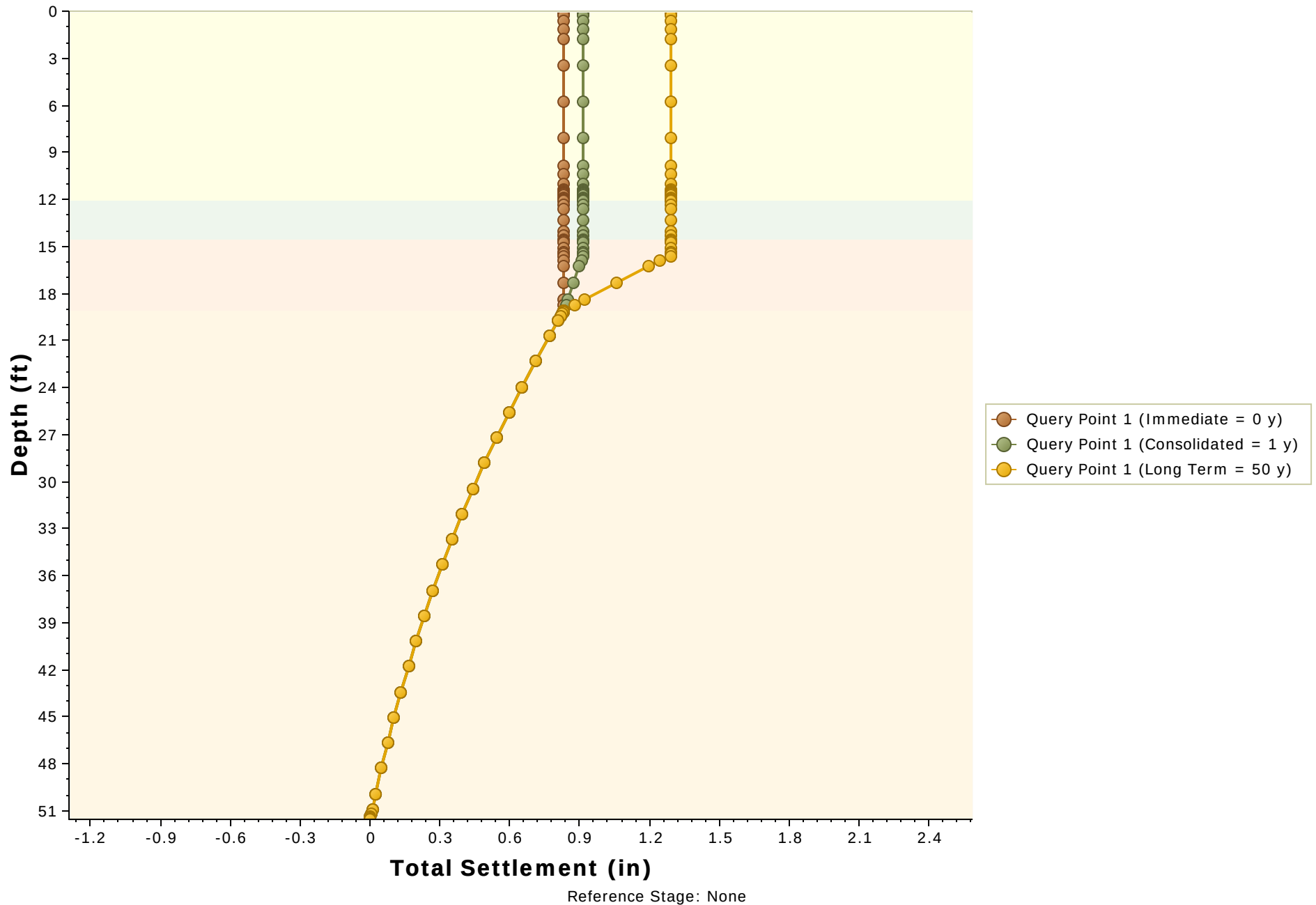
Grid Coordinates

X [ft]	Y [ft]
77.138	70.8
77.138	-35.75
-68.862	-35.75
-68.862	70.8



	Project		21690.00 Farmington Williams Bridge
	Analysis Description		settlement analysis q=500
	Drawn By	A. Van Buskirk	Company
	Date	11/27/2018, 11:24:08 AM	File Name 21690 Farmington Settlement_q=500_run1.s3z

Total Settlement vs. Depth



Settle3D Analysis Information

21690.00 Farmington Williams Bridge

Project Settings

Document Name	21690 Farmington Settlement_q=1500_run2.s3z
Project Title	21690.00 Farmington Williams Bridge
Analysis	settlement analysis q = 1500
Author	A. Van Buskirk
Date Created	11/27/2018, 11:24:08 AM
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name	Time [years]
1	Immediate	0
2	Consolidated	1
3	Long Term	50

Results

Time taken to compute: 0 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.50387
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	2.50387
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	4.09809
Total Stress [ksf]	0	7.1247
Total Strain	0	0.00954514
Pore Water Pressure [ksf]	0	3.05508
Excess Pore Water Pressure [ksf]	0	1.5
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.00106119	4.09678
Over-consolidation Ratio	1	1.7
Void Ratio	0	0.9
Permeability [ft/y]	0	0.110743
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.819617

Stage: Consolidated = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.11853
Consolidation Settlement [in]	0	0.665512
Immediate Settlement [in]	0	2.50387
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	4.61825
Total Stress [ksf]	0	7.1247
Total Strain	-6.09554e-005	0.0168589
Pore Water Pressure [ksf]	0	2.56296
Excess Pore Water Pressure [ksf]	0	0.0641168
Degree of Consolidation [%]	0	99.9985
Pre-consolidation Stress [ksf]	0.00106119	4.61729
Over-consolidation Ratio	1	1.71518
Void Ratio	0	0.900116
Permeability [ft/y]	0	0.0116245
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.83608

Stage: Long Term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.51741
Consolidation Settlement [in]	0	0.708358
Immediate Settlement [in]	0	2.50387
Secondary Settlement [in]	0	0.48628
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	4.63196
Total Stress [ksf]	0	7.1247
Total Strain	0	0.0259289
Pore Water Pressure [ksf]	0	2.50195
Excess Pore Water Pressure [ksf]	-6.14839e-018	9.39415e-018
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.00106119	4.631
Over-consolidation Ratio	1	1.7
Void Ratio	0	0.9
Permeability [ft/y]	0	0.110743
Coefficient of Consolidation [ft ² /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.836183

Loads

1. Rectangular Load

Length	73 ft
Width	26 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1898 ft ²
Load	1.5 ksf
Depth	15.6 ft
Installation Stage	Immediate = 0 y

Coordinates

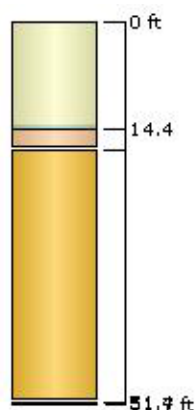
X [ft]	Y [ft]
-32.362	4.512
40.638	4.512
40.638	30.512
-32.362	30.512

Soil Layers

Ground Surface Drained: Yes

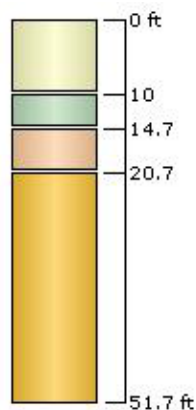
Borehole 1: (-6.2, 0.75)

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Silty Sand	14.4	0	No
2	Saturated Alluvium	0	14.4	No
3	Soft Clayey Silt	3	14.4	No
4	Very Stiff Sandy Silt	34	17.4	No



Borehole 2: (-3.7, 34.3)

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Silty Sand	10	0	No
2	Saturated Alluvium	4.7	10	No
3	Soft Clayey Silt	6	14.7	No
4	Very Stiff Sandy Silt	31	20.7	No



Soil Properties

Property	Loose Silty Sand	Saturated Alluvium	Soft Clayey Silt	Very Stiff Sandy Silt
Color				
Unit Weight [kips/ft ³]	0.102	0.102	0.083	0.121
Saturated Unit Weight [kips/ft ³]	0.117	0.117	0.115	0.139
Immediate Settlement	Disabled	Disabled	Disabled	Enabled
Es [ksf]				156
Esur [ksf]				624
Primary Consolidation	Disabled	Disabled	Enabled	Disabled
Material Type			Non-Linear	
Cc			0.3	
Cr			0.03	
e0			0.9	
OCR	1	1	1.7	1
Cv [ft ² /y]			36.5	
B-bar			1	
Secondary Consolidation	Disabled	Disabled	Standard	Disabled
Cae/Ca			0.01	
Car/Care			0.01	
Undrained Su A [kips/ft ²]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Groundwater

Groundwater method Piezometric Lines
 Water Unit Weight 0.0624 kips/ft³

Piezometric Line Entities

ID	Depth (ft)
1	11.6 ft

Query Points

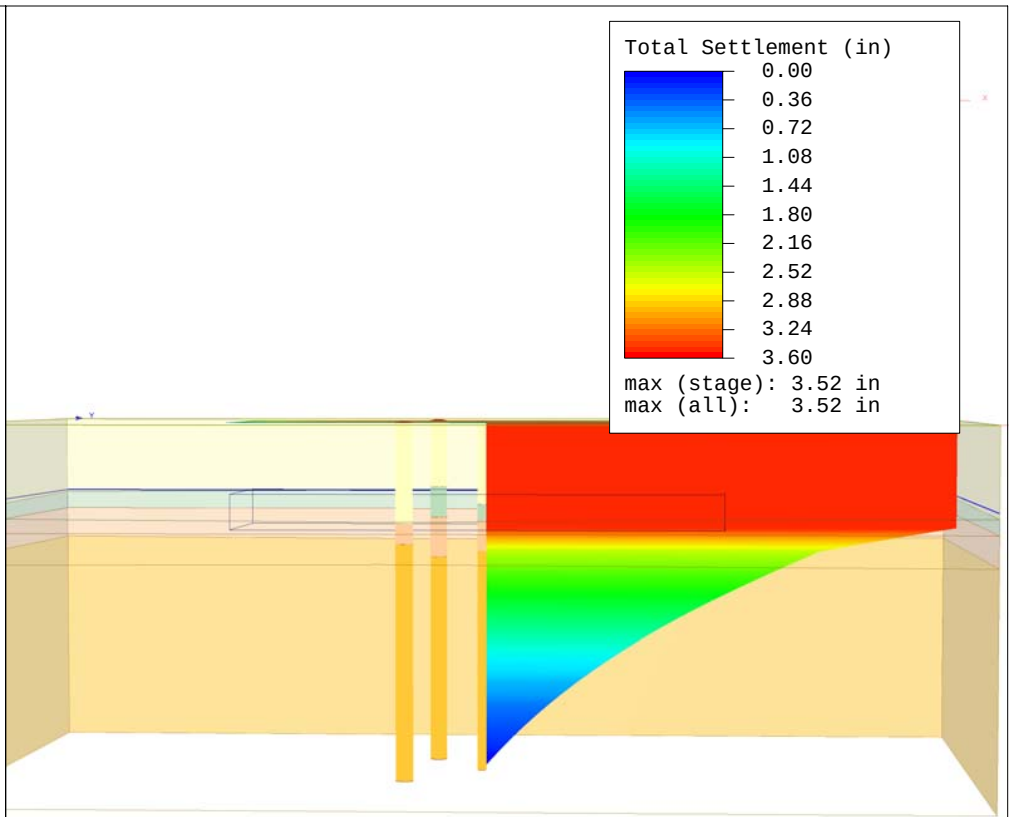
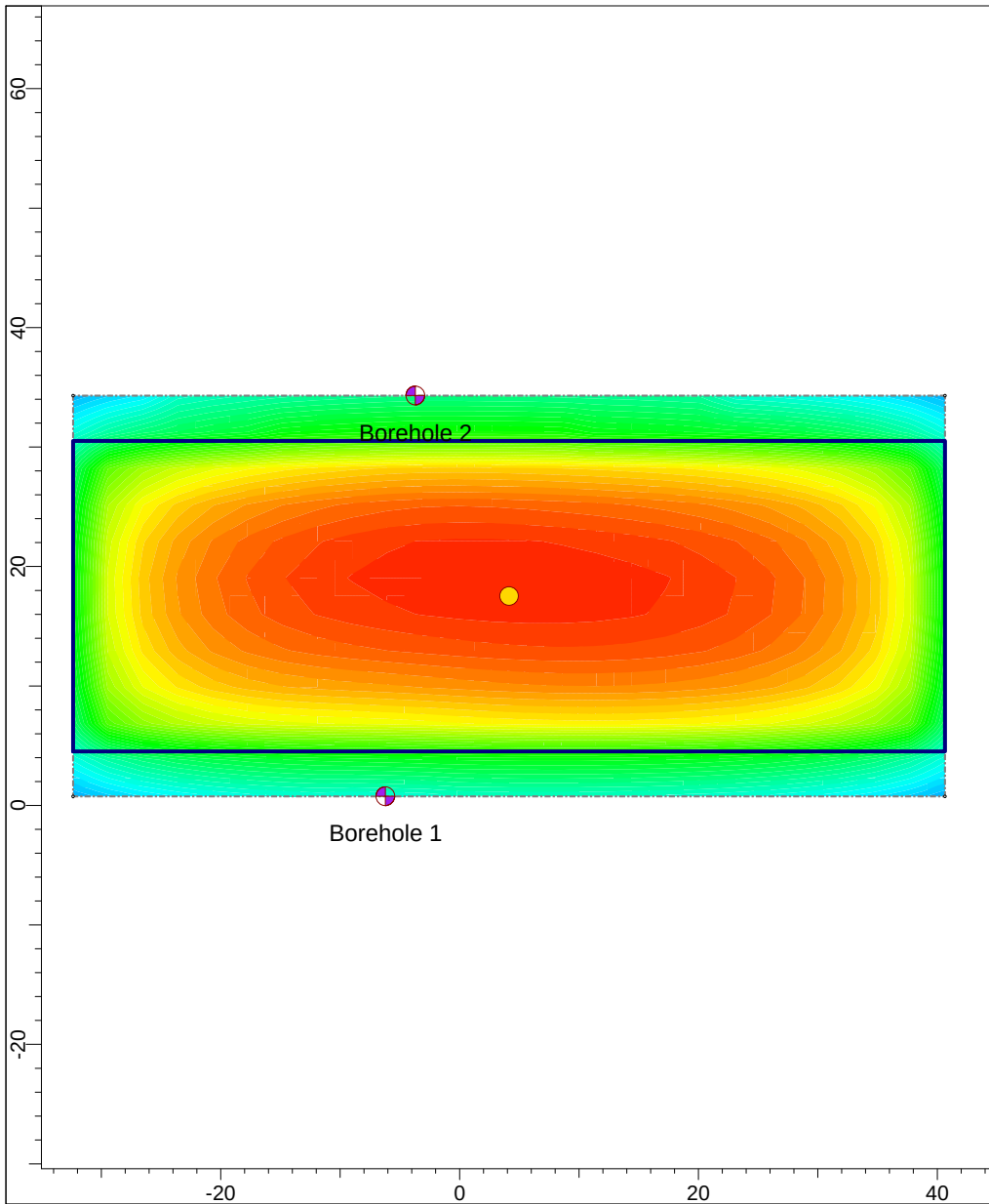
Point #	(X,Y) Location	Number of Divisions
1	4.138, 17.512	Auto: 73

Field Point Grid

Number of points 312
 Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
77.138	70.8
77.138	-35.75
-68.862	-35.75
-68.862	70.8



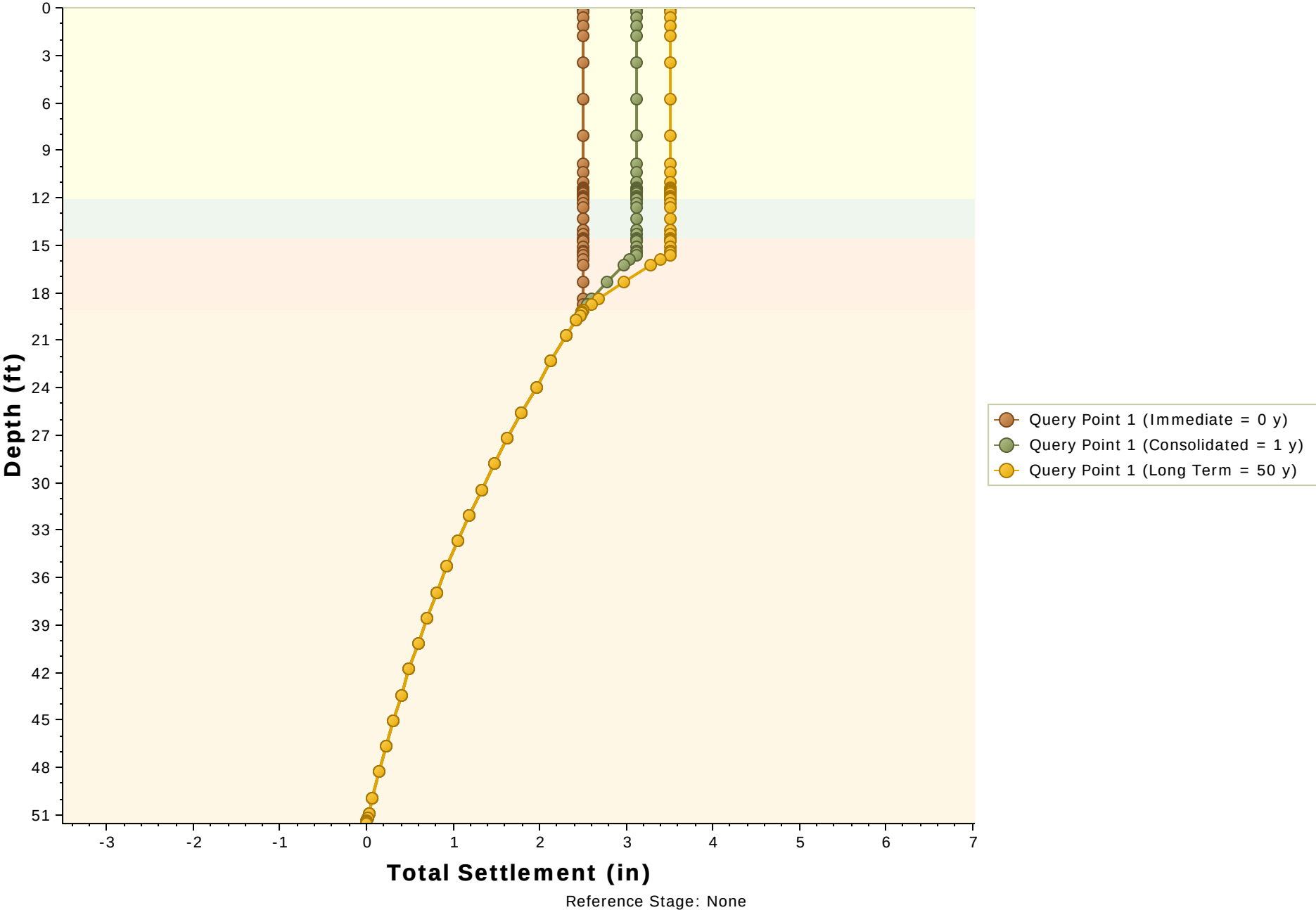
Total Settlement (in)

0.00
0.36
0.72
1.08
1.44
1.80
2.16
2.52
2.88
3.24
3.60

max (stage): 3.52 in
max (all): 3.52 in

	Project		21690.00 Farmington Williams Bridge
	Analysis Description		settlement analysis q = 1500
	Drawn By	A. Van Buskirk	Company
	Date	11/27/2018, 11:24:08 AM	File Name 21690 Farmington Settlement_q=1500_run2.s3z

Total Settlement vs. Depth



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 - fine grained granular fill soils W=30% .

$$DFI_1 := 1800 \quad d_1 := 49.8 \cdot \text{in}$$

$$DFI_2 := 1900 \quad d_2 := 51.1 \text{ in}$$

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

$$DFI_3 := 1870$$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1) \cdot (d_2 - d_1)}{(DFI_2 - DFI_1)} \quad d_3 = 50.7 \cdot \text{in}$$

$$\text{Depth of Frost Penetration} \quad d_3 = 4.2 \cdot \text{ft}$$

--- ModBerg Results ---

Project Location: Farmington, Maine

Air Design Freezing Index = 2023 F-days

N-Factor = 0.70

Surface Design Freezing Index = 1416 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-Fine 46.4 30.0 83.0 27 39 1.0 .6 3,586

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).

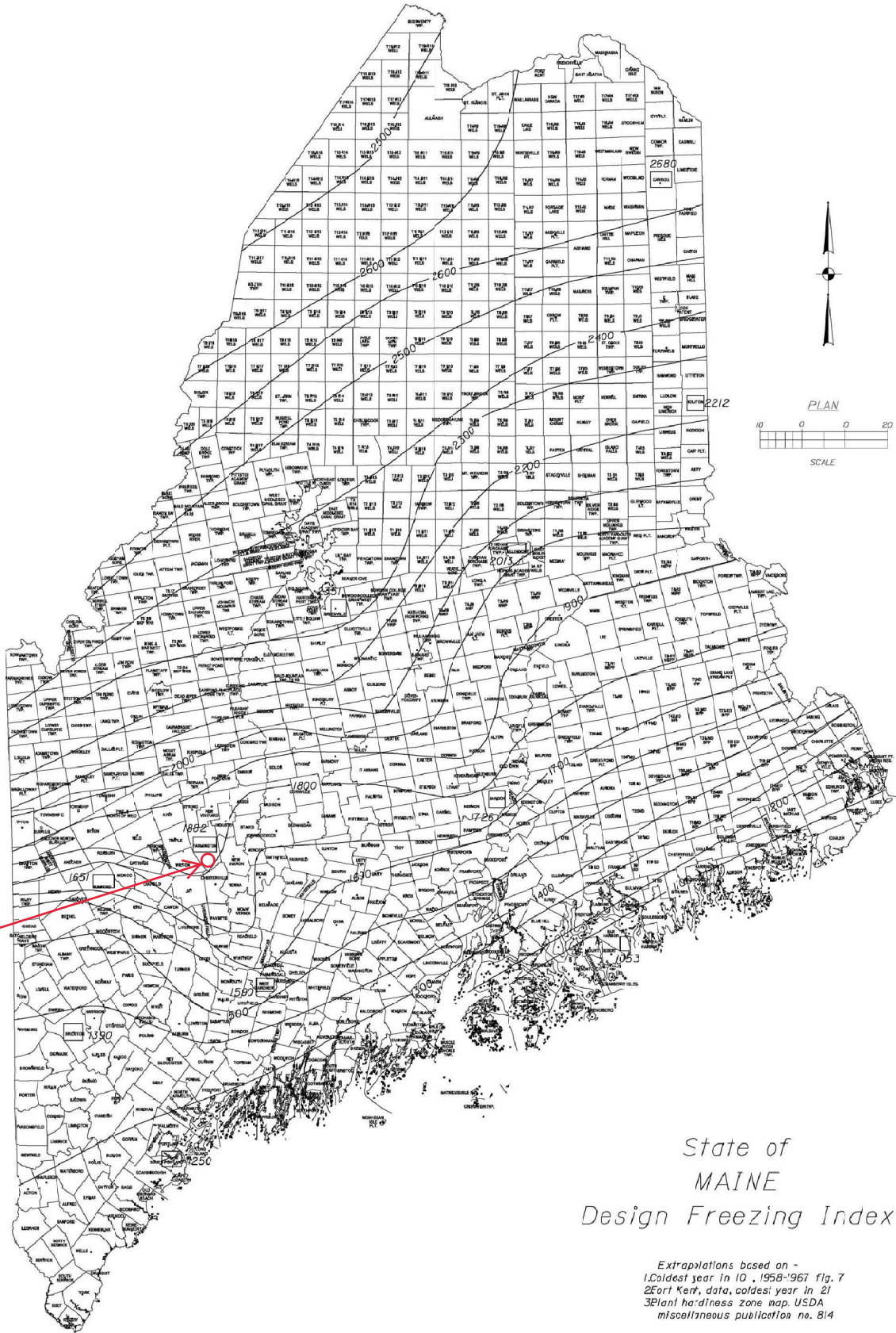
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

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

Total Depth of Frost Penetration = 3.87 ft = 46.4 in.

Recommendation: 4.2 feet for design of foundations constructed on fine grained soil.

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