MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CREEK BRIDGE
HARBOR ROAD OVER CREEK BROOK
LOVELL, MAINE

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Oxford County
WIN 22248.00

Soils Report 2019-44
Bridge No. 3582

Project No. 22248.00
October 23, 2019

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Creek Bridge, which carries Harbor Road over Creek Brook in Lovell, 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 14-foot 11-inch span by 10-foot 2-inch rise steel plate pipe arch culvert was built in 1983, replacing a smaller steel pipe culvert. The 2018 Maine Department of Transportation (MaineDOT) Bridge Inspection Report rates the culvert as 4 (poor condition). The report cites a large hole on the east corner near the outlet and large rust nodules at or below the waterline throughout the culvert. The structure has a FHWA Sufficiency Rating of 66.2. The culvert is scoped for replacement due to its poor condition.

The proposed replacement structure will be a 15-foot span by 12-foot rise precast concrete box culvert with a total length of 90 feet. The concrete box culvert will have 2-foot deep precast toe walls to prevent undermining of the box. The inlet and outlet ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) sideslopes. The box culvert will be embedded approximately 3 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box culvert will be installed on an 18-inch thick layer of crushed stone reinforced wrapped in stabilization/reinforcement geotextile.

The proposed horizontal alignment of the proposed box culvert will approximately match the existing horizontal alignment. The proposed vertical alignment will raise the roadway profile approximately 3 inches over the length of the new pavement section. The bridge will be closed during replacement and traffic will be maintained on an offsite detour. The bridge replacement project will last approximately one construction season.

2.0 GEOLOGIC SETTING

Creek Bridge carries Harbor Road over the Creek Brook, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Center Lovell Quadrangle (2014), Open-File Map 14-28, indicates the surficial soil at the project site is stream alluvium. The stream alluvium is mapped with contacts to glacial till and glaciolacustrine deposits of the Glacial Lake Pigwacket formation. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Glaciolacustrine sediments of the Glacial Lake Pigwacket formation consist of varved layers of clay and silt.

The MGS Bedrock Geologic Map of Maine (1985) cites the bedrock at the project site as an intrusion consisting of carboniferous muscovite-biotite granite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings. Boring BB-LCB-101 was outside the southwest corner of the existing culvert and boring BB-LCB-102 was drilled outside the northeast corner of the culvert. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

Test borings BB-LCB-101 and BB-LCB-102 were drilled between July 31 and August 2, 2018 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were performed using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" prior to the test borings in July and August 2018. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 for both borings. The hammer efficiency factor (0.928) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

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

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of 12 standard grain size analyses with natural water content, five grain size analyses with hydrometer and natural water content, and five 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 Granular Fill, Stream Alluvium, Glaciolacustrine Silt and Clay, and Glacial Till, underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of fill was encountered in the borings. The thickness of the fill unit encountered was approximately 9.5 feet at the boring locations. The unit encountered generally consisted of:

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

SPT N_{60}-values in the layer ranged from 22 to 45 blows per foot (bpf), indicating the layer is medium dense to dense in consistency. Based on grain size analyses conducted on samples of the Fill, the soil is classified as A-2-4 under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 5 to 7 percent.

5.2 Stream Alluvium

Stream Alluvium was encountered in the borings beneath the Fill unit. The thickness of the deposit encountered was approximately 15 feet at the boring locations. The deposit generally consisted of:

- Brown, wet, sand, some silt, trace gravel;
- Brown or grey, wet, silty sand, trace gravel trace wood fibers, trace organics;
- Grey, wet, fine sand, little silt, trace gravel.
- Grey or grey-brown, wet, sandy silt, trace gravel;

SPT N_{60}-values in the Alluvium ranged from 3 to 25 bpf indicating the consistency of the silty layers are soft to stiff and the coarse-grained layers are loose to dense. Based on grain size analyses conducted on samples the soils are classified as A-2-4 and A-4 under the AASHTO Soil Classification System and CL and SM under the USCS. The natural water contents of the samples ranged from approximately 28 to 39 percent.

5.3 Glaciolacustrine Deposit

A deposit of Glaciolacustrine silt, clay and fine sand was encountered in the borings beneath the Stream Alluvium. The thickness of the deposit encountered ranged from approximately 6 to 14 feet at the boring locations. The layered deposit generally consisted of:

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

SPT N_{60}-values in the Glaciolacustrine deposit ranged from less than 1 to 32 bpf indicating the deposit is very soft to hard or dense in consistency.

Six grain size analysis conducted on samples of the Glaciolacustrine deposit classified the soils as A-4 under the AASHTO Soil Classification System and CL, ML, and CL-ML under the USCS. The natural water contents of the samples tested ranged from approximately 32 to 40 percent.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the Glaciolacustrine deposit. A 55 x 110 vane was used. One vane shear test could not be completed because the vane was unable to be pushed by hand to test depth. Five of six successful vane shear tests conducted within the silty clay layers measured undisturbed undrained shear strengths ranging from approximately 290 to 759 psf, indicating that the layered silt and clay deposit is soft to medium stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 22 to 80 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the silt and clay deposit has sensitivities ranging from 6 to 17 and is classified as medium sensitive to quick.

Atterberg limits tests were conducted on five samples of the Glaciolacustrine deposit, and are summarized in Table 1:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-LCB-101, 6D/B	SILT	39.1	25	24	1	15.1
BB-LCB-101, 7D	Clayey SILT	39.7	30	22	8	2.2
BB-LCB-101, 8D/A	Silty CLAY	32.9	26	24	2	4.5
BB-LCB-102, 6D	Clayey SILT	35.5	27	20	7	2.2
BB-LCB-102, 7A	SILT	37.7	28	24	4	3.4

Table 1 – Summary of Atterberg Limits Tests

The plasticity indices of the soil samples indicate the soils range from slightly plastic to exhibiting low plasticity (Burmister, 1949). The natural water contents of all the tested samples exceeded the liquid limits and the liquidity indices ranged from approximately 2 to 15. 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 has the potential to convert this type of deposit into a viscous liquid. Soils with liquidity index values greater than or equal to 1 are also commonly referred to as "quick."

5.4 Glacial Till

Glacial till was encountered beneath the Silt and Clay (Glaciolacustrine) deposit. Borings BB-LCB-101 and BB-LCB-102 were terminated within the glacial till layer at approximately 52 feet below the roadway surface (brs). The deposit encountered generally consisted of:

- Grey, gravel, little sand, trace silt;
- Grey, gravelly sand;
- Grey, sand, little to some silt, trace to little gravel;
- Grey, sandy gravel, trace silt;
- Brown, silty sand.

SPT N_{60}-values in the glacial till ranged from 48 to greater than 50 bpf indicating glacial till is dense to very dense in consistency. Four grain size analyses conducted on samples from the glacial till resulted in the soils being classified as A-2-4 and A-4 under the AASHTO Soil Classification System and SM under the USCS. The moisture content of the tested samples ranged from approximately 10 to 22 percent.

5.5 Groundwater

Groundwater was measured in the borings at depths ranging from 5.4 to 7.5 feet bgs. Water was introduced into the boreholes during drilling operations. Therefore, water levels may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report (PDR) dated April 2019 investigated one replacement alternative: a precast concrete box culvert. A precast concrete box culvert has the advantages of efficiency of installation and low maintenance and lifecycle costs. The bottom slab of the precast concrete box culvert will be embedded into the streambed to accommodate 2 feet of engineered streambed material creating a natural streambed. Refer to the project PDR for a more detailed analysis and discussion of the replacement alternative.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 15-foot span by 12-foot rise precast concrete box culvert with slope-tapered inlet and outlet walls. The box culvert will have 1-foot tall precast headwalls. To prevent undermining, the box culvert will have inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 3 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. The riprap apron should be embedded 6 inches into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

The box culvert should be installed on a minimum 18-inch thick layer of crushed stone wrapped in stabilization/reinforcement geotextile. To facilitate setting and sliding adjacent precast box segments, the 18-inch thick crushed stone mat can be topped with 6-inches of granular borrow bedding, or steel rollers or steel plates can be utilized.

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

The loading specified for the design of the box culvert shall be Modified HL-93 Strength I, which increases the HS-20 design truck wheel loads by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ 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 stone slope treatment and prevent stones from dropping or eroding into the waterway. Nominal 1-foot by 1- foot concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert's outlet and inlet walls will be slope-tapered to match the 2H:1V sideslopes of the roadway embankment. The left and right outlet walls will share the same base slab. The sloped walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5, and 11.6. The inlet and outlet walls shall be designed to resist lateral earth pressures and forces resulting from creep, temperature, and shrinkage deformations of the concrete box culvert. Passive pressure resulting from the embedment of the box culvert and walls with engineered streambed, or any other material shall not contribute to resisting forces.

Inlet and outlet walls that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient, K_o, of 0.47. Wingwall sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope. Wingwall sections that are independent of the box culvert and have a backslope of 2H:1V should be designed using the Rankine active earth pressure coefficient of 0.46. See Appendix C – Calculations for supporting documentation.

7.1.3 Precast Concrete Toe Walls

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

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on an 18-inch-thick layer of crushed stone that is wrapped in stabilization/reinforcement. The bearing elevation of the crushed stone mat will be approximately Elev. 364 at the inlet and Elevation 363 at the outlet. The Alluvial soils at this elevation is expected to consist of loose fine sand and soft, sandy silt..

For a precast concrete box culvert with a base width of 17 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 3.0 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 bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$. See Appendix C – Calculations for supporting calculations.

7.1.5 Modulus of Subgrade Reaction

Large span precast box culverts can be viewed similarly to a mat foundation where the volume of soil displaced by the foundation will result in a lower net applied stress. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 27 pounds per cubic inch (pci).

7.2 Settlement

The bottom slab of the proposed box culvert will bear on a layer of Stream Alluvium underlain by soft to medium stiff Glaciolacustrine silt and clay. An estimated Service Limit State pressure of 1500 psf results in a zero net applied pressure increase. Therefore, for the Service Limit State, we do not anticipate any immediate or long-term settlement. A net pressure increase of 880 psf is estimated based on an assumed Strength Limit State bearing pressure of 2500 psf. For this load case we anticipate immediate (elastic) and consolidation settlement on the order of 3 inches within the first year of construction and an additional 2.5 inches of long-term consolidation settlement over the life of the structure. The estimated elastic settlement (1.6 inches) will occur relatively quickly and during construction. See Appendix C – Calculations for supporting documentation.

7.3 Subgrade Preparation

The saturated, loose fine sands and soft silt (Stream Alluvium) encountered at the box culvert 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;
- Limit vibration-induced disturbance to the subgrade, to limit the risk of excavation bottom heave;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade;
- Avoid overexcavation. It will be important to maintain as much of the Alluvium over the lower, weaker soils, and the underlying Glaciolacustrine Clays and Silt.
- The box culvert shall be installed on an 18-inch thick layer of crushed stone; the crushed stone mat shall be wrapped in stabilization/reinforcement geotextile.
- Hand-deploy the geotextile on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat.
- In lieu of the crushed stone mat, a flowable fill (mud) mat may be substituted to provide a stable subgrade for installation of the box culvert;
- For either a crushed stone mat or mud mat, top the mat with 6-inches of granular borrow bedding to facilitate setting and sliding adjacent precast box segments. Alternatively, steel rollers or steel plates can be utilized.

The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type C 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 plate 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, Lovell has a design freezing index (DFI) of approximately 1400 F-degree days. A water content of 20% was used for saturated, fine sands. These components correlate to a frost depth of 5.5 feet. It is recommended that foundations bearing on soil be designed with an embedment of approximately 5.5 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 2H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

7.6 Seismic Design Considerations

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

7.7 Construction Considerations

The box culvert will be installed on an 18-inch thick layer of crushed stone reinforced wrapped in stabilization/reinforcement geotextile. The geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. To facilitate setting and sliding adjacent precast box segments, the 18-inch thick crushed stone mat can be topped with 6-inches of granular borrow bedding, or steel rollers or steel plates can be utilized.

The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type C 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 loose sands and soft silt (Alluvium) 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.
- Limit vibration-induced disturbance to limit the risk of excavation bottom heave;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade.

- Avoid overexcavation. It will be important to maintain as much of the Alluvium over the lower, weaker soils, and the underlying Glaciolacustrine Clay and Silt.
- Hand-deploy the stabilization/reinforcement geotextile on the prepared soil subgrade prior to installing the crushed stone mat.
- In lieu of the crushed stone mat, a flowable fill (mud) mat may be substituted to provide a stable subgrade for installation of the box culvert;

For either a crushed stone mat or mud mat, top the mat with 6-inches of granular borrow bedding to facilitate setting and sliding adjacent precast box segments. Alternatively, steel rollers or steel plates can be utilized to move/slide precast box units.

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.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT for specific application to the proposed replacement of Creek Bridge in Lovell, 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.

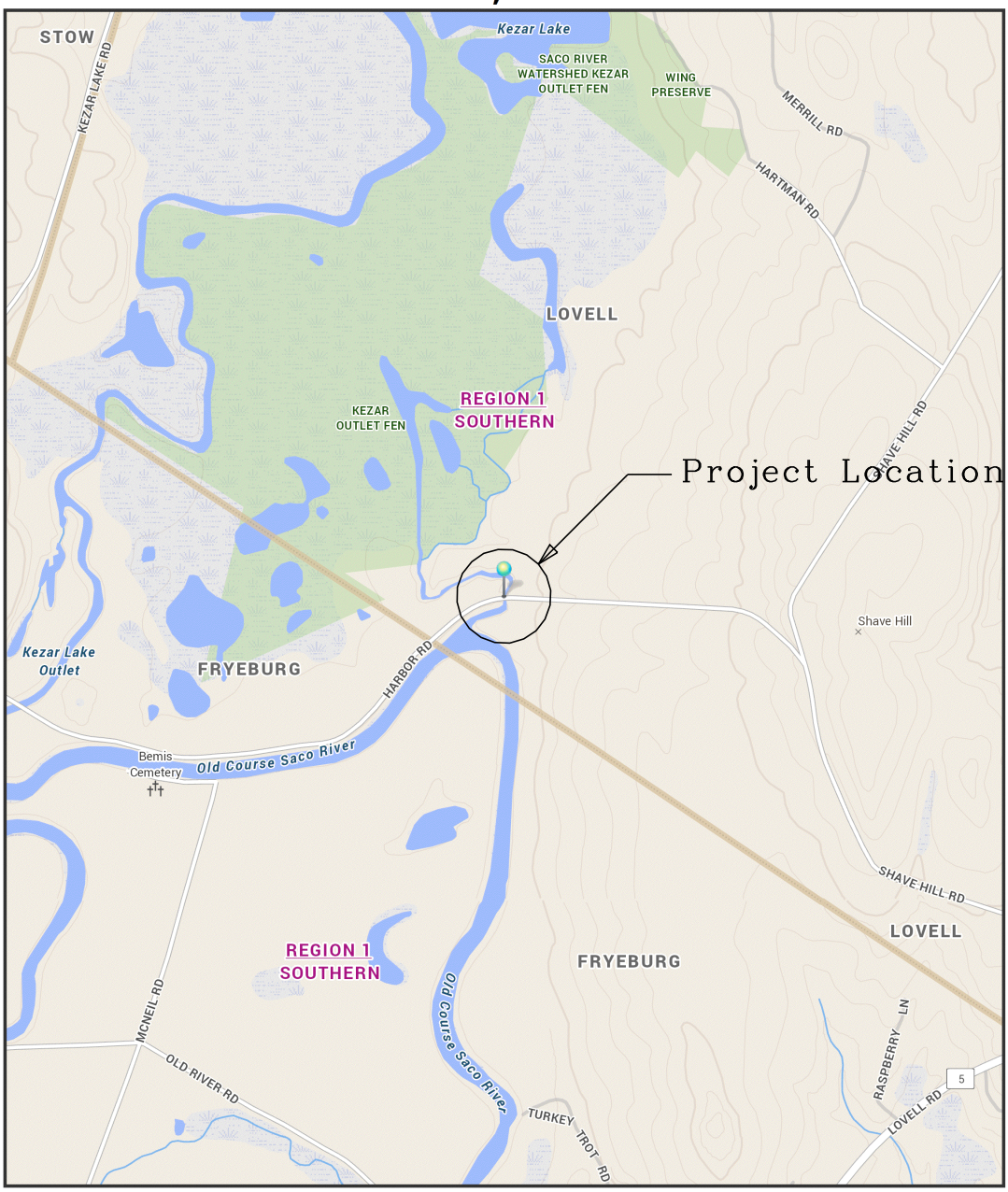
Sheets

Date:1/28/2019

Division: GEOTECH

Username: Terry.White

LOVELL, MAINE



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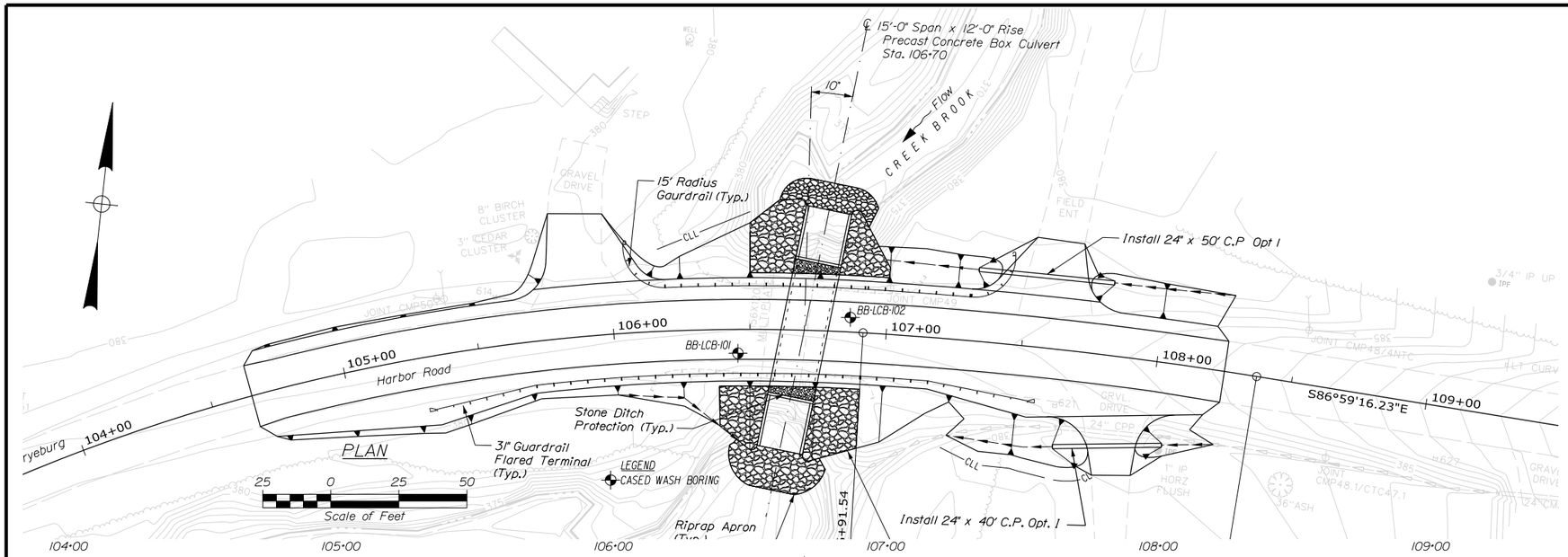
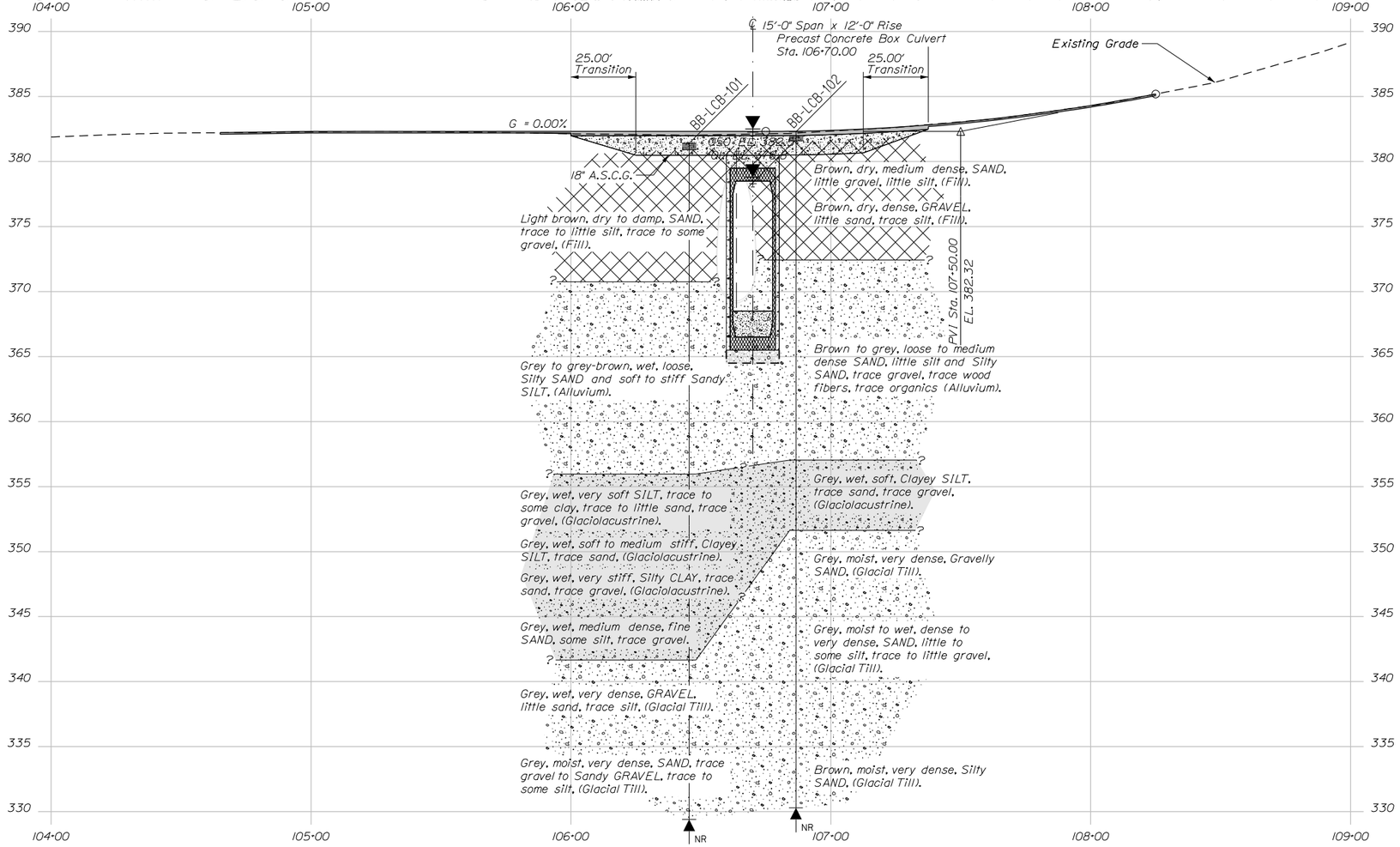
0.25 Miles

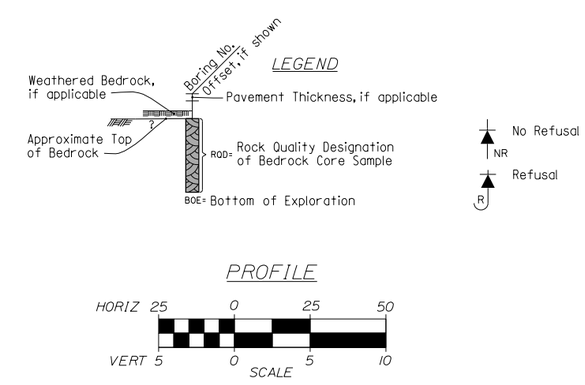
1 inch = 0.28 miles

Date: 1/28/2019
Time: 7:35:12 AM

SHEET NUMBER		
1	CREEK BRIDGE CREEK BROOK	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
	LOVELL OXFORD COUNTY	22248.00
OF 3	LOCATION MAP	WIN BRIDGE NO. 3582 22248.00 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

22248.00

WIN 22248.00

BRIDGE NO. 3582

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. Kersbergen		
DESIGN-DETAILED	K. Nash	M. Critchlow	June 2018
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	K.KRUSINSKI	T.WHITE	OCT 2019
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

CREEK BRIDGE
CREEK BROOK
OXFORD COUNTY

LOVELL

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\GEOTECH\MSTA\006_BLP&SP1.dgn Division: GEOTECH Username: terry.white Date:10/18/2019

Filename: ...\MSTA\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:10/18/2019

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook	Location: Lovell, Maine	Boring No.: BB-LCB-101	WIN: 22248.00
Driller: MaineDOT	Elevation (ft.) 381.4	Auger ID/OD: 5" Solid Stem	
Operator: Daggett/Sullivan	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 7/31-8/1/2018	Drilling Method: Cased Wash Boring	Core Barrel: N/A	
Boring Location: 106+45.5, 8.8 ft Rt.	Casing ID/OD: NW-3"	Water Level*: 5.4 ft bgs.	
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	380.9	6" HMA. 0.5	
	1D	24/13	1.00 - 3.00	6/7/7/4	14	22			Light brown, dry, medium dense, fine to medium SAND, little silt, little gravel, (Fill).	G#296576 A-2-4, SM WC=4.9%
5	2D	24/7	5.00 - 7.00	4/9/8/5	17	26			Light brown, damp, medium dense, SAND, some gravel, trace silt, (Fill).	
10	3D	24/4	10.00 - 12.00	1/2/2/2	4	6	3	371.4	10.0 Brown, wet, loose, Silty SAND, trace gravel, (Alluvium).	
							5			
							6			
							9			
							12			
15	4D	24/10	15.00 - 17.00	3/3/3/2	6	9	17		Grey, wet, stiff, Sandy SILT, trace gravel, (Alluvium).	G#296577 A-4, CL WC=28.1%
							17			
							21			
							22			
							21			
20	5D	24/15	20.00 - 22.00	1/1/1/1	2	3	25		Grey brown, wet, soft, fine Sandy SILT, (Alluvium).	G#296578 A-4, CL WC=38.1%
							24			
							22			
							29			
							30			
25	6D	24/24	25.00 - 27.00	WOH/WOH/WOR/WOR	---		14	356.4	25.0 6D/A (25.0-26.0 ft bgs) Grey, wet, very soft, SILT, little sand, trace gravel. 6D/B (26.0-27.0 ft bgs) Grey, wet, very soft, SILT, some clay, trace sand, (Glaciolacustrine).	G#296579 A-4, CL WC=31.7% G#296580 A-4, ML WC=39.1% LL=25 PL=24 PI=1
							26			
							28			
							30			
	V1		29.00 - 29.37	Su=290/45 psf			32		55x110 mm raw vane readings: V1: 6.5/1.0 ft-lbs	
30	V2		30.00 - 30.37	Su=558/45 psf			24		V2: 12.5/1.0 ft-lbs	
	V3 7D	24/24	31.00 - 31.37 31.00 - 33.00	Su=603/67 psf WOR/WOR/WOR/WOR	---		35		V3: 13.5/1.5 ft-lbs Grey, wet, soft to medium stiff, Clayey SILT, trace sand, (Glaciolacustrine).	G#296581 A-4, CL WC=39.7% LL=30 PL=22 PI=8
	V4		32.00 - 32.37	Su=759/80 psf			35		V4: 17.0/1.8 ft-lbs	
							45			
							43			
35	MV 8D	24/12	35.00 - 37.00	Would not Push 2/4/17/15	21	32	25		Failed 55x110 mm vane attempt. 8D/A (35.0-35.7 ft bgs.) Grey, wet, Silty CLAY, trace sand, trace gravel, (Glaciolacustrine). 8D/B (35.7-37.0 ft bgs.) Grey, wet, dense, fine SAND, some silt, trace gravel.	G#296582 A-4, ML WC=32.9% LL=26 PL=24 PI=2
							30			
							56			
							60			
							167	342.4	39.0 Change of density at 39.0 ft bgs.	
40	9D	24/8	40.00 - 42.00	38/19/20/21	39	60	175		Grey, wet, very dense, GRAVEL, little sand, trace silt, (Glacial Till).	
							170			
							171			
							190			
							232			
45	10D		45.00 - 47.00	34/50/60/74	110	170	OPEN HOLE		Grey, moist, very dense, SAND, some silt, trace gravel, (Glacial Till).	G#296583 A-2-4, SM WC=10.3%
50	11D	24/19	50.00 - 52.00	2/49/64/71	113	175			Grey, moist, very dense, Sandy GRAVEL, trace silt, (Glacial Till).	
								329.4	52.0 Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL	
55										
60										
65										
70										
75										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-LCB-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook	Location: Lovell, Maine	Boring No.: BB-LCB-102	WIN: 22248.00
Driller: MaineDOT	Elevation (ft.) 382.3	Auger ID/OD: 5" Solid Stem	
Operator: Daggett/Sullivan	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: A. Van Buskirk	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 8/1/2018-8/2/2018	Drilling Method: Cased Wash Boring	Core Barrel: N/A	
Boring Location: 106+86.6, 5.4 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 7.5 ft bgs.	
Hammer Efficiency Factor: 0.928	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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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
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	381.6	8.5" HMA. 0.7	
	1D	24/20	1.00 - 3.00	7/7/10/8	17	26			Brown, dry, medium dense, SAND, little gravel, little silt, (Fill).	G#296584 A-2-4, SM WC=7.4%
5	2D	24/3	5.00 - 7.00	12/14/15/5	29	45	6		Brown, dry, dense, GRAVEL, little sand, trace silt, (Fill).	
							20			
							16			
							17			
							9			
10	3D	24/9	10.00 - 12.00	4/4/2/2	6	9	4	372.3	10.0 Brown, wet, loose, SAND, some silt, trace gravel, (Alluvium).	G#296585 A-2-4, SM WC=39.0%
							15			
							12			
							13			
							12			
15	4D	24/10	15.00 - 17.00	5/9/7/6	16	25	16		Grey, wet, medium dense, Silty SAND, trace gravel, trace wood fibers, organics, (Alluvium).	G#296586 A-4, SM WC=28.7%
							13			
							16			
							26			
							16			
20	5D	24/14	20.00 - 22.00	2/2/1/WOH	3	5	15		Grey, wet, loose, fine SAND, little silt, trace gravel.	G#296587 A-2-4, SM WC=25.1%
							26			
							17			
							15			
							14			
25	6D V1	24/24	25.00 - 27.00 25.63 - 26.00	Su=379/22 psf			17	357.3	25.0 Grey, wet, soft Clayey SILT, trace sand, trace gravel, (Glaciolacustrine). 55x110 mm raw torque readings: V1: 8.5/0.5 ft-lbs.	G#296588 A-4, CL-ML WC=35.5% LL=27 PL=20 PI=7
	V2		26.63 - 27.00	Su=335/45 psf			18		V2: 7.5/1.0 ft-lbs	
							21			
							25			
							30			
30	7D	24/11	30.00 - 32.00	WOH/67/26/27	93	144	23	351.8	7D/A (30.0-30.5 ft bgs.) Grey, wet, hard, SILT, little clay, trace sand. Open hole from 30.0-35.0 ft bgs, then drove casing. 30.5 7D/B (30.5-32.0 ft bgs.) Grey, moist, very dense, Gravelly SAND (Glacial Till).	G#296589 A-4, ML WC=37.7% LL=28 PL=24 PI=4
							32			
							30			
							31			
							40			
35	8D	24/13	35.00 - 37.00	7/16/15/14	31	48	29		Grey, moist, dense, SAND, little gravel, little silt, (Glacial Till).	G#296591 A-2-4, SM WC=12.1%
							50			
							54			
							86			
							214			
40	9D	24/23	40.00 - 42.00	27/44/63/46	107	165	23		Grey, wet, very dense, SAND, some silt, little gravel, (Glacial Till).	
							17			
							24			
							a122		a122 blows for 0.7 ft.	
									Roller Coned ahead from 43.7-45.0 ft bgs.	
45	10D	19/14	45.00 - 46.58	47/77/104/18(1")	181	280	OPEN HOLE		Grey, moist, very dense, SAND, some silt, trace gravel, (Glacial Till).	G#296592 A-2-4, SM WC=9.9%
50	11D	24/24	50.00 - 52.00	28/37/40/54	77	119			Brown, moist, very dense, Silty SAND, (Glacial Till).	G#296593 A-4, SM WC=22.4%
								330.3	52.0 Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL	
55										
60										
65										
70										
75										

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Page 1 of 1

Boring No.: BB-LCB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

22248.00

BRIDGE NO. 3582 WIN 22248.00 BRIDGE PLANS

PROJ. MANAGER	M. Kersbergen	BY	DATE
DESIGN-DETAILED	K. Nash	M. Critchlow	June 2018
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	L.KRUSINSKI	T.WHITE	OCT 2019
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE

P.E. NUMBER

DATE

CREEK BRIDGE
CREEK BROOK
LOVELL OXFORD COUNTY

BORING LOGS

SHEET NUMBER 3 OF 3

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook
Location: Lovell, Maine

Boring No.: BB-LCB-101

WIN: 22248.00

Driller:	MaineDOT	**Elevation (ft.)**	381.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Sullivan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/31-8/1/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	106+45.5, 8.8 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	5.4 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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WOR/C = Weight of Rods or Casing
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	380.9		6" HMA. 0.5	
	1D	24/13	1.00 - 3.00	6/7/7/4	14	22				Light brown, dry, medium dense, fine to medium SAND, little silt, little gravel, (Fill).	G#296576 A-2-4, SM WC=4.9%
5	2D	24/7	5.00 - 7.00	4/9/8/5	17	26				Light brown, damp, medium dense, SAND, some gravel, trace silt, (Fill).	
10								371.4		10.0	
	3D	24/4	10.00 - 12.00	1/2/2/2	4	6	3			Brown, wet, loose, Silty SAND, trace gravel, (Alluvium).	
							5				
							6				
							9				
							12				
15	4D	24/10	15.00 - 17.00	3/3/3/2	6	9	17			Grey, wet, stiff, Sandy SILT, trace gravel, (Alluvium).	G#296577 A-4, CL WC=28.1%
							17				
							21				
							22				
							21				
20	5D	24/15	20.00 - 22.00	1/1/1/1	2	3	25			Grey brown, wet, soft, fine Sandy SILT, (Alluvium).	G#296578 A-4, CL WC=38.1%
							24				
							22				
							29				
25							30				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook
Location: Lovell, Maine

Boring No.: BB-LCB-101
WIN: 22248.00

Driller:	MaineDOT	**Elevation (ft.)**	381.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Sullivan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/31-8/1/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	106+45.5, 8.8 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	5.4 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	WOH/WOH/WOR/ WOR	---		14	356.4	25.0 6D/A (25.0-26.0 ft bgs) Grey, wet, very soft, SILT, little sand, trace gravel. 6D/B (26.0-27.0 ft bgs) Grey, wet, very soft, SILT, some clay, trace sand, (Glaciolacustrine).	G#296579 A-4, CL WC=31.7% G#296580 A-4, ML WC=39.1% LL=25 PL=24 PI=1
							26			
							28			
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	V1		29.00 - 29.37	Su=290/45 psf			32		55x110 mm raw vane readings: V1: 6.5/1.0 ft-lbs	
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	V3	24/24	31.00 - 31.37	Su=603/67 psf	---		35		V3: 13.5/1.5 ft-lbs Grey, wet, soft to medium stiff, Clayey SILT, trace sand, (Glaciolacustrine). V4: 17.0/1.8 ft-lbs	G#296581 A-4, CL WC=39.7% LL=30 PL=22 PI=8
	7D V4		31.00 - 33.00 32.00 - 32.37	WOR/WOR/WOR/ WOR Su=759/80 psf			35			
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35	MV 8D	24/12	35.00 - 37.00	Would not Push 2/4/17/15	21	32	25		Failed 55x110 mm vane attempt. 8D/A (35.0-35.7 ft bgs.) Grey, wet, Silty CLAY, trace sand, trace gravel, (Glaciolacustrine). 8D/B (35.7-37.0 ft bgs.) Grey, wet, dense, fine SAND, some silt, trace gravel.	G#296582 A-4, ML WC=32.9% LL=26 PL=24 PI=2
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							170			
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							190			
							232			
45	10D		45.00 - 47.00	34/50/60/74	110	170	OPEN HOLE		Grey, moist, very dense, SAND, some silt, trace gravel, (Glacial Till).	G#296583 A-2-4, SM WC=10.3%
50										

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								329.4		52.0	
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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

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Location: Lovell, Maine

Boring No.: BB-LCB-102

WIN: 22248.00

Driller:	MaineDOT	**Elevation (ft.)**	382.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Sullivan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/1/2018-8/2/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	106+86.6, 5.4 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	7.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	381.6		8.5" HMA. 0.7	
	1D	24/20	1.00 - 3.00	7/7/10/8	17	26				Brown, dry, medium dense, SAND, little gravel, little silt, (Fill).	G#296584 A-2-4, SM WC=7.4%
5	2D	24/3	5.00 - 7.00	12/14/15/5	29	45	6			Brown, dry, dense, GRAVEL, little sand, trace silt, (Fill).	
							20				
							16				
							17				
							9				
10	3D	24/9	10.00 - 12.00	4/4/2/2	6	9	4	372.3		10.0 Brown, wet, loose, SAND, some silt, trace gravel, (Alluvium).	G#296585 A-2-4, SM WC=39.0%
							15				
							12				
							13				
							12				
15	4D	24/10	15.00 - 17.00	5/9/7/6	16	25	16			Grey, wet, medium dense, Silty SAND, trace gravel, trace wood fibers, organics, (Alluvium).	G#296586 A-4, SM WC=28.7%
							13				
							16				
							26				
							16				
20	5D	24/14	20.00 - 22.00	2/2/1/WOH	3	5	15			Grey, wet, loose, fine SAND, little silt, trace gravel.	G#296587 A-2-4, SM WC=25.1%
							26				
							17				
							15				
25							14				

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook
Location: Lovell, Maine

Boring No.: BB-LCB-102
WIN: 22248.00

Driller:	MaineDOT	**Elevation (ft.)**	382.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Sullivan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/1/2018-8/2/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	106+86.6, 5.4 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	7.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	Su=379/22 psf			17	357.3	Grey, wet, soft Clayey SILT, trace sand, trace gravel, (Glaciolacustrine). 55x110 mm raw torque readings: V1: 8.5/0.5 tf-lbs. V2: 7.5/1.0 ft-lbs	25.0 G#296588 A-4, CL-ML WC=35.5% LL=27 PL=20 PI=7
	V1		25.63 - 26.00							
	V2		26.63 - 27.00	Su=335/45 psf			18			
							21			
							25			
							30			
30	7D	24/11	30.00 - 32.00	WOH/67/26/27	93	144	23	351.8	7D/A (30.0-30.5 ft bgs.) Grey, wet, hard, SILT, little clay, trace sand. Open hole from 30.0-35.0 ft bgs, then drove casing.	G#296589 A-4, ML WC=37.7% LL=28 PL=24 PI=4
							32		30.5 7D/B (30.5-32.0 ft bgs.) Grey, moist, very dense, Gravelly SAND (Glacial Till).	
							30			
							31			
							40			
35	8D	24/13	35.00 - 37.00	7/16/15/14	31	48	29		Grey, moist, dense, SAND, little gravel, little silt, (Glacial Till).	G#296591 A-2-4, SM WC=12.1%
							50			
							54			
							86			
							214			
40	9D	24/23	40.00 - 42.00	27/44/63/46	107	165	23		Grey, wet, very dense, SAND, some silt, little gravel, (Glacial Till).	
							17			
							24			
							[a]122		[a]122 blows for 0.7 ft. Roller Coned ahead from 43.7-45.0 ft bgs.	
45	10D	19/14	45.00 - 46.58	47/77/104/18(1")	181	280	OPEN HOLE		Grey, moist, very dense, SAND, some silt, trace gravel, (Glacial Till).	G#296592 A-2-4, SM WC=9.9%
50										

Remarks:

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

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

Boring No.: BB-LCB-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creek Bridge #3582 carries Harbor Road over Creek Brook
Location: Lovell, Maine

Boring No.: BB-LCB-102
WIN: 22248.00

Driller:	MaineDOT	**Elevation (ft.)**	382.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Sullivan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/1/2018-8/2/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	106+86.6, 5.4 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	7.5 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/24	50.00 - 52.00	28/37/40/54	77	119				Brown, moist, very dense, Silty SAND, (Glacial Till).	G#296593 A-4, SM WC=22.4%
								330.3		52.0	
										Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL	
55											
60											
65											
70											
75											

Remarks:

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

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

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. sandy, clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lovell Work Number: 22248.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-LCB-101, 1D	106+45.5	8.8 Rt.	1.0-3.0	296576	1	4.9			SM	A-2-4	II
BB-LCB-101, 4D	106+45.5	8.8 Rt.	15.0-17.0	296577	1	28.1			CL	A-4	IV
BB-LCB-101, 5D	106+45.5	8.8 Rt.	20.0-22.0	296578	1	38.1			CL	A-4	IV
BB-LCB-101, 6D/A	106+45.5	8.8 Rt.	25.0-26.0	296579	1	31.7			CL	A-4	IC
BB-LCB-101, 6D/B	106+45.5	8.8 Rt.	26.0-27.0	296580	1	39.1	25	1	ML	A-4	IV
BB-LCB-101, 7D	106+45.5	8.8 Rt.	31.0-33.0	296581	2	39.7	30	8	CL	A-4	IV
BB-LCB-101, 8D/A	106+45.5	8.8 Rt.	35.0-35.7	296582	2	32.9	26	2	ML	A-4	IV
BB-LCB-101, 10D	106+45.5	8.8 Rt.	45.0-47.0	296583	2	10.3			SM	A-2-4	II
BB-LCB-102, 1D	106+86.6	5.4 Lt.	1.0-3.0	296584	3	7.4			SM	A-2-4	II
BB-LCB-102, 3D	106+86.6	5.4 Lt.	10.0-12.0	296585	3	39.0			SM	A-2-4	II
BB-LCB-102, 4D	106+86.6	5.4 Lt.	15.0-17.0	296586	3	28.7			SM	A-4	III
BB-LCB-102, 5D	106+86.6	5.4 Lt.	20.0-22.0	296587	3	25.1			SM	A-2-4	II
BB-LCB-102, 6D	106+86.6	5.4 Lt.	25.0-27.0	296588	4	35.5	27	7	CL-ML	A-4	IV
BB-LCB-102, 7D/A	106+86.6	5.4 Lt.	30.0-30.5	296589	4	37.7	28	4	ML	A-4	IV
BB-LCB-102, 8D	106+86.6	5.4 Lt.	35.0-37.0	296591	4	12.1			SM	A-2-4	II
BB-LCB-102, 10D	106+86.6	5.4 Lt.	45.0-46.6	296592	4	9.9			SM	A-2-4	II
BB-LCB-102, 11D	106+86.6	5.4 Lt.	50.0-52.0	296593	4	22.4			SM	A-4	III

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

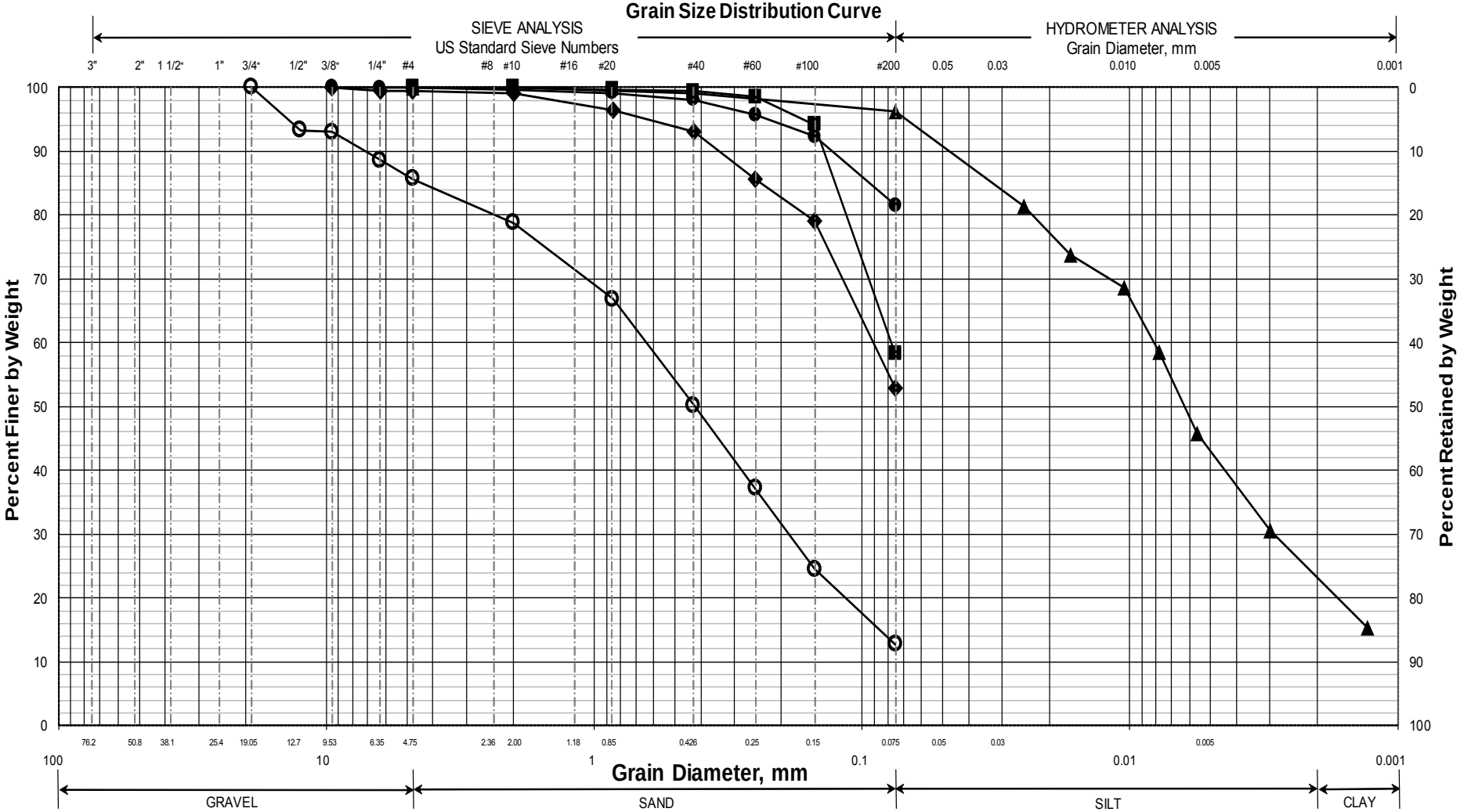
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98 NP = Non Plastic

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

Maine Department of Transportation
Grain Size Distribution Curve



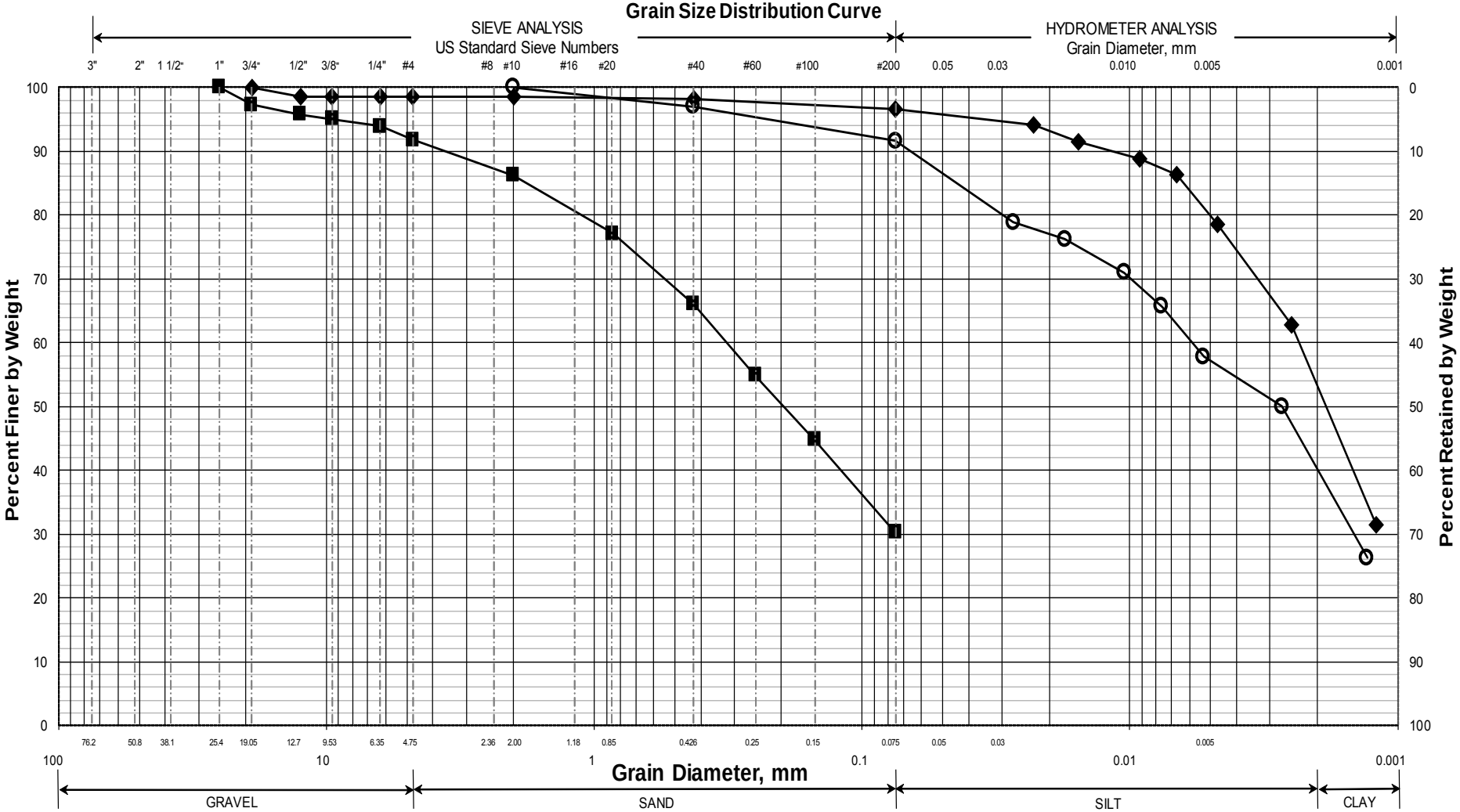
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LCB-101/1D	106+45.5	8.8 RT	1.0-3.0	SAND, little gravel, little silt.	4.9			
◆	BB-LCB-101/4D	106+45.5	8.8 RT	15.0-17.0	Sandy SILT, trace gravel.	28.1			
■	BB-LCB-101/5D	106+45.5	8.8 RT	20.0-22.0	Sandy SILT.	38.1			
●	BB-LCB-101/6DA	106+45.5	8.8 RT	25.0-26.0	SILT, little sand, trace gravel.	31.7			
▲	BB-LCB-101/6DB	106+45.5	8.8 RT	26.0-27.0	SILT, some clay, trace sand.	39.1	25	24	1
×									

WIN
022248.00
Town
Lovell
Reported by/Date
WHITE, TERRY A 9/1/2018

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



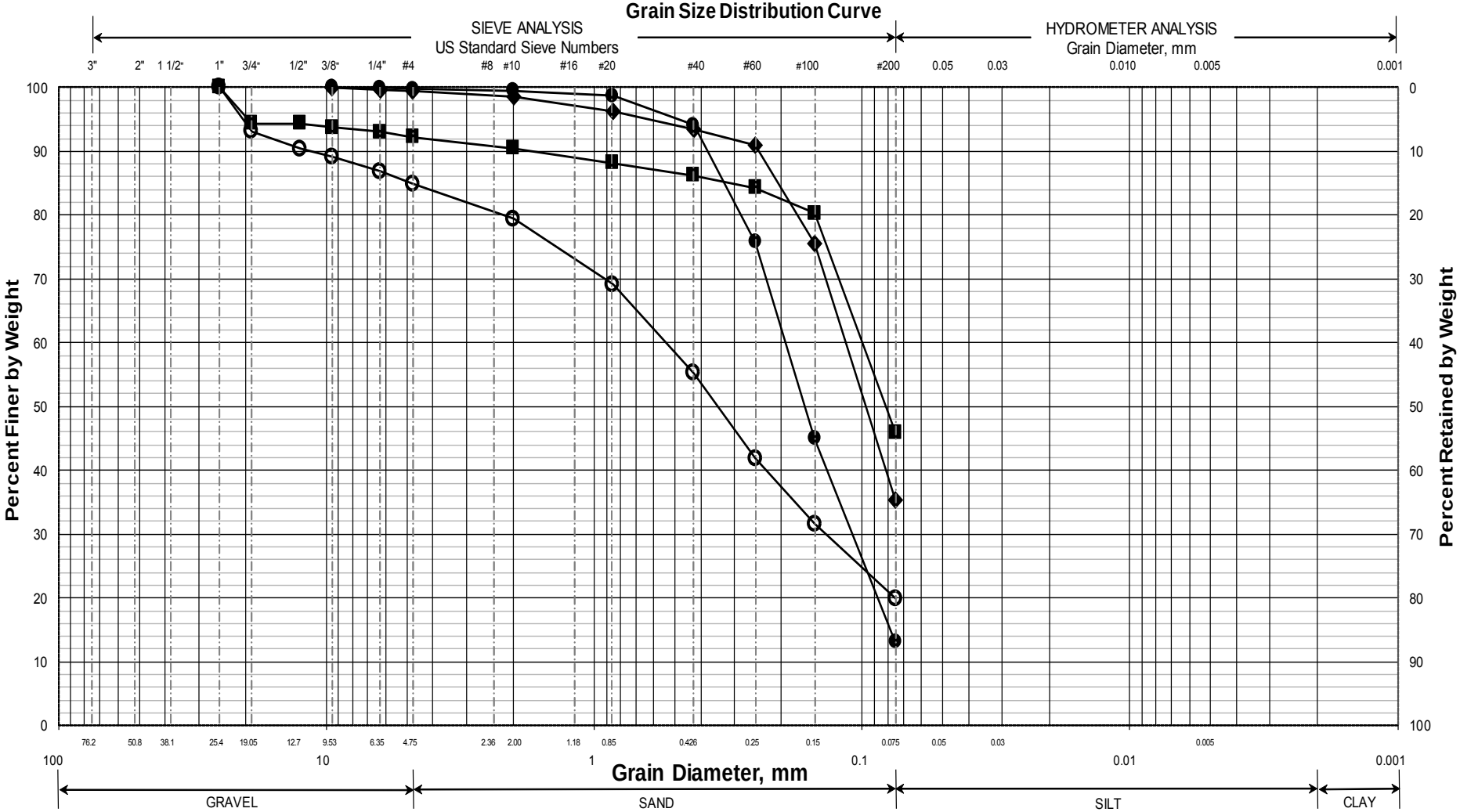
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LCB-101/7D	106+45.5	8.8 RT	31.0-33.0	Clayey SILT, trace sand.	39.7	30	22	8
◆	BB-LCB-101/8DA	106+45.5	8.8 RT	35.0-35.7	Silty CLAY, trace sand, trace gravel.	32.9	26	24	2
■	BB-LCB-101/10D	106+45.5	8.8 RT	45.0-47.0	SAND, some silt, trace gravel.	10.3			
●									
▲									
×									

WIN
022248.00
Town
Lovell
Reported by/Date
WHITE, TERRY A 9/1/2018

SHEET 2

Maine Department of Transportation
Grain Size Distribution Curve



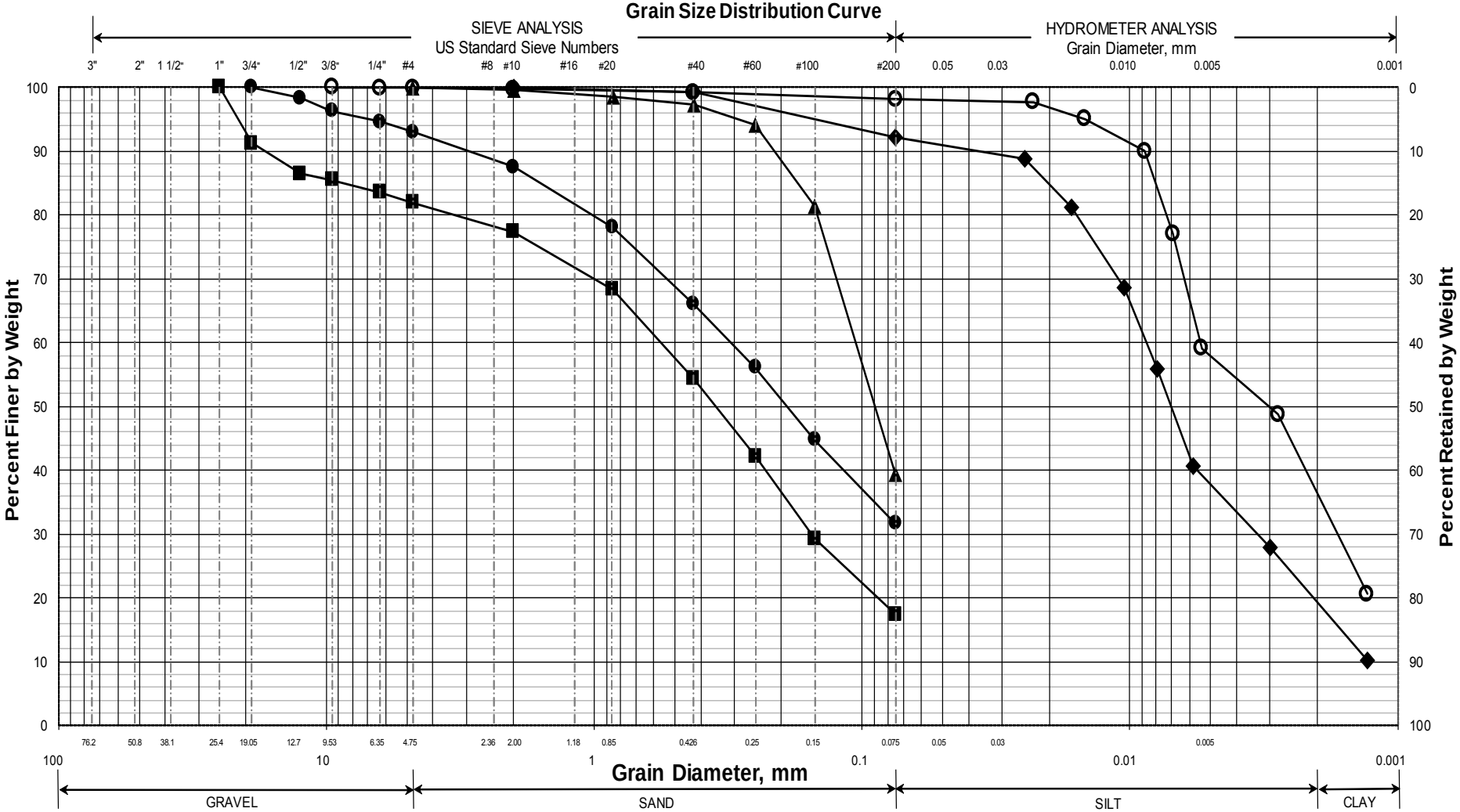
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LCB-102/1D	106+86.6	5.4 LT	1.0-3.0	SAND, little silt, little gravel.	7.4			
◆	BB-LCB-102/3D	106+86.6	5.4 LT	10.0-12.0	SAND, some silt, trace gravel.	39			
■	BB-LCB-102/4D	106+86.6	5.4 LT	15.0-17.0	Silty SAND, trace gravel.	28.7			
●	BB-LCB-102/5D	106+86.6	5.4 LT	20.0-22.0	SAND, little silt, trace gravel.	25.1			
▲									
×									

WIN
022248.00
Town
Lovell
Reported by/Date
WHITE, TERRY A 9/1/2018

SHEET 3

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LCB-102/6D	106+86.6	5.4 LT	25.0-27.0	Clayey SILT, trace sand, trace gravel.	35.5	27	20	7
◆	BB-LCB-102/7DA	106+86.6	5.4 LT	30.0-30.5	SILT, little clay, trace sand.	37.7	28	24	4
■	BB-LCB-102/8D	106+86.6	5.4 LT	35.0-37.0	SAND, little gravel, little silt.	12.1			
●	BB-LCB-102/10D	106+86.6	5.4 LT	45.0-46.6	SAND, some silt, trace gravel.	9.9			
▲	BB-LCB-102/11D	106+86.6	5.4 LT	50.0-52.0	Silty SAND.	22.4			
×									

WIN
022248.00
Town
Lovell
Reported by/Date
WHITE, TERRY A 9/1/2018

SHEET 4

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296580	**BB-LCB-101/6DB**	**GEOTECHNICAL (DISTURBED)**	**7/31/2018**	**8/16/2018**

Sample Type: **GEOTECHNICAL** Location:

Station: **106+45.5** Offset, ft: **8.8** **RT** Dbfg, ft: **26.0-27.0**

WIN/Town **022248.00 - LOVELL**

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]	**100.0**
No. 10 [2.00 mm]	**100.0**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**96.2**
[0.0248 mm]	**81.4**
[0.0166 mm]	**73.8**
[0.0105 mm]	**68.7**
[0.0078 mm]	**58.5**
[0.0056 mm]	**45.8**
[0.0030 mm]	**30.5**
[0.0013 mm]	**15.3**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**25**
Plastic Limit (T 90), %	**24**
Plasticity Index (T 90), %	**1**
Specific Gravity, Corrected to 20°C (T 100)	**2.69**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**39.1**

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		

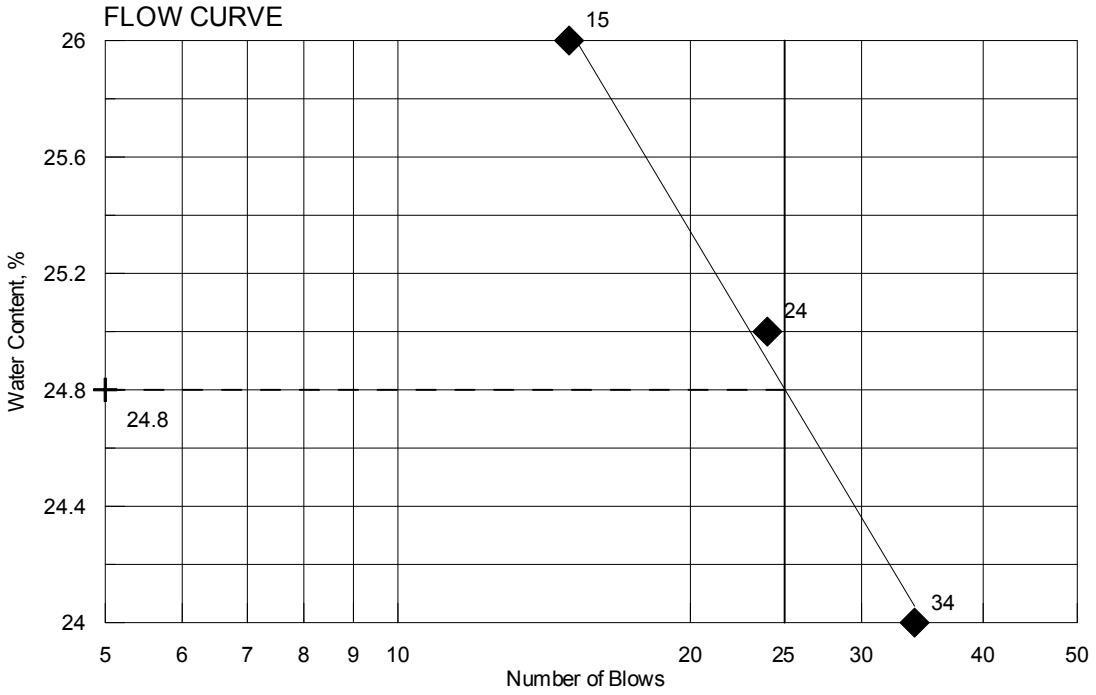
Comments:

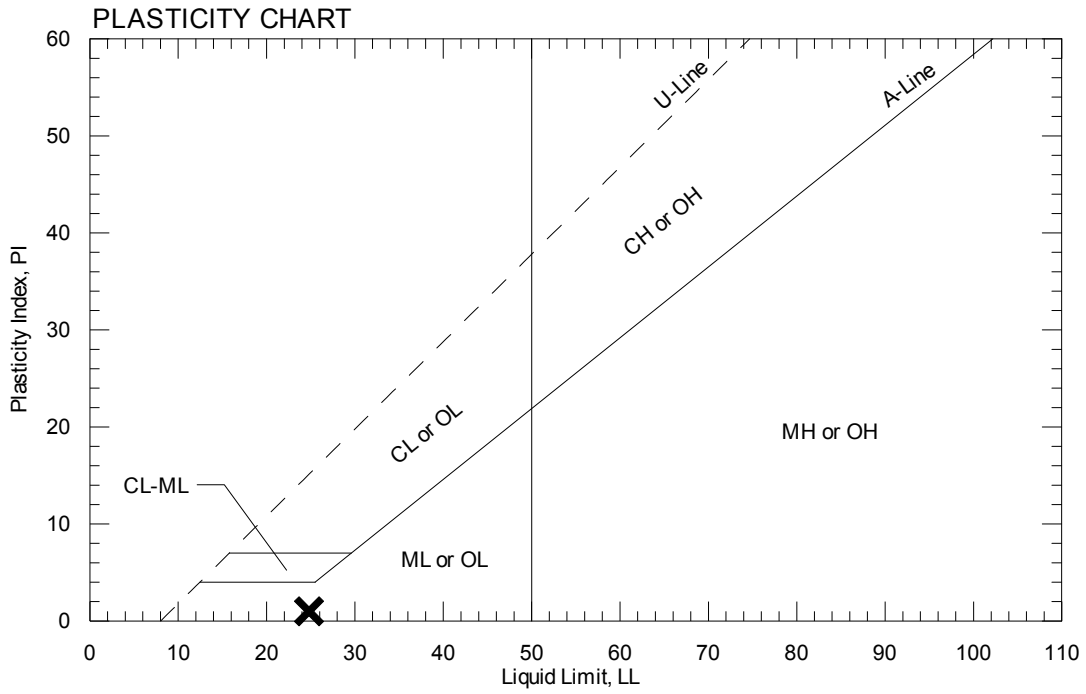
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/27/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Lovell	Reference No.	296580
WIN	022248.00	Water Content, %	39.1
Sampled	7/31/2018	Liquid Limit @ 25 blows (T 89), %	25
Boring No./Sample No.	BB-LCB-101/6DB	Plastic Limit (T 90), %	24
Station	106+45.5	Plasticity Index (T 90), %	1
Depth	26.0-27.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296581	**BB-LCB-101/7D**	**GEOTECHNICAL (DISTURBED)**	**7/31/2018**	**8/16/2018**

Sample Type: **GEOTECHNICAL** Location:

Station: **106+45.5** Offset, ft: **8.8** **RT** Dbfg, ft: **31.0-33.0**

WIN/Town **022248.00 - LOVELL**

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]	**97.0**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**91.6**
[0.0272 mm]	**78.9**
[0.0175 mm]	**76.2**
[0.0105 mm]	**71.0**
[0.0076 mm]	**65.7**
[0.0053 mm]	**57.8**
[0.0027 mm]	**50.0**
[0.0013 mm]	**26.3**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**30**
Plastic Limit (T 90), %	**22**
Plasticity Index (T 90), %	**8**
Specific Gravity, Corrected to 20°C (T 100)	**2.70**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**39.7**

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

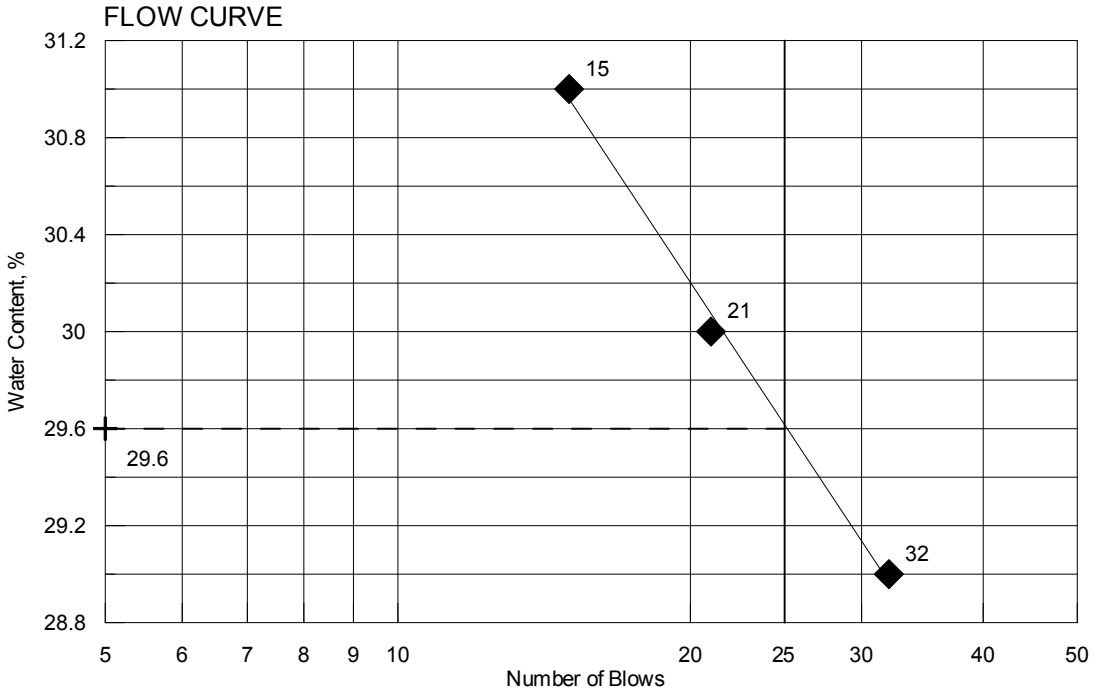
Comments:

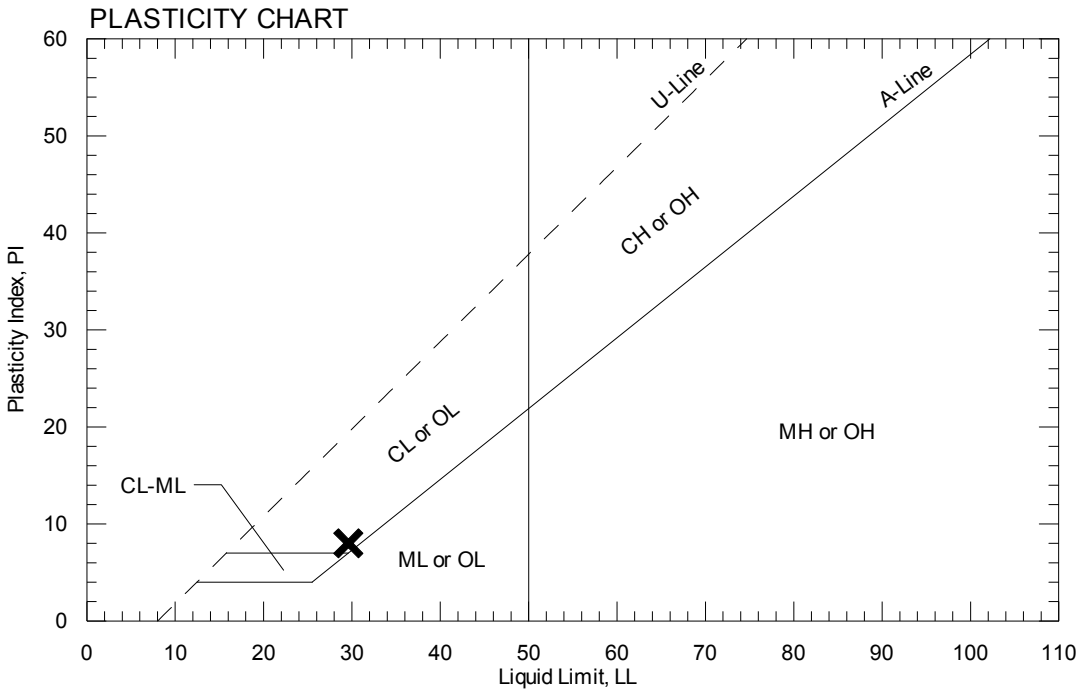
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/27/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Lovell	Reference No.	296581
WIN	022248.00	Water Content, %	39.7
Sampled	7/31/2018	Liquid Limit @ 25 blows (T 89), %	30
Boring No./Sample No.	BB-LCB-101/7D	Plastic Limit (T 90), %	22
Station	106+45.5	Plasticity Index (T 90), %	8
Depth	31.0-33.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296582	**BB-LCB-101/8DA**	**GEOTECHNICAL (DISTURBED)**	**7/31/2018**	**8/16/2018**

Sample Type: **GEOTECHNICAL** Location:

Station: **106+45.5** Offset, ft: **8.8** **RT** Dbfg, ft: **35.0-35.7**

WIN/Town **022248.00 - LOVELL**

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]	**100.0**
½ in. [12.5 mm]	**98.6**
⅜ in. [9.5 mm]	**98.6**
¼ in. [6.3 mm]	**98.6**
No. 4 [4.75 mm]	**98.6**
No. 10 [2.00 mm]	**98.6**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**98.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**96.6**
[0.0229 mm]	**94.2**
[0.0156 mm]	**91.5**
[0.0092 mm]	**88.9**
[0.0067 mm]	**86.3**
[0.0047 mm]	**78.5**
[0.0025 mm]	**62.8**
[0.0012 mm]	**31.4**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**26**
Plastic Limit (T 90), %	**24**
Plasticity Index (T 90), %	**2**
Specific Gravity, Corrected to 20°C (T 100)	**2.70**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**32.9**

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		

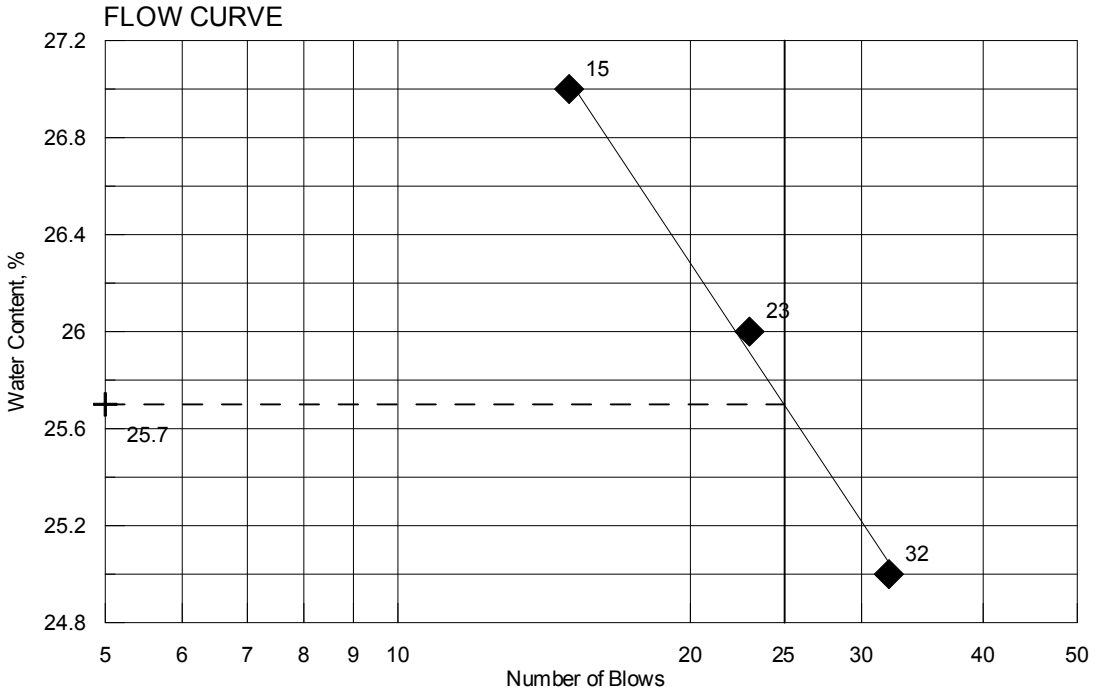
Comments:

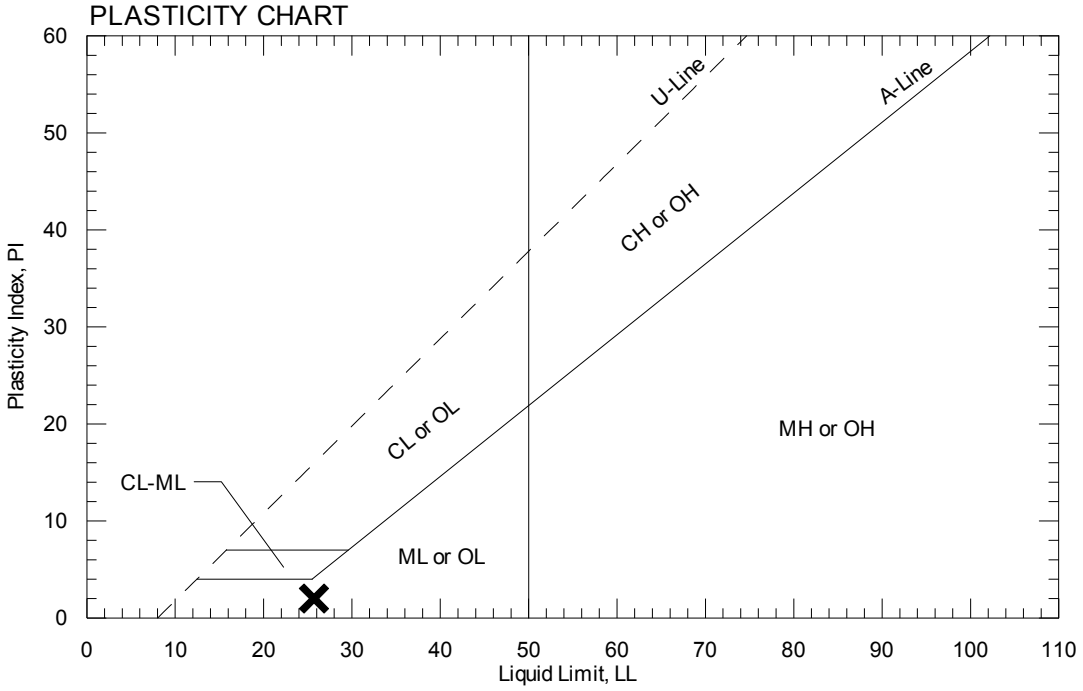
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/27/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Lovell	Reference No.	296582
WIN	022248.00	Water Content, %	32.9
Sampled	7/31/2018	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-LCB-101/8DA	Plastic Limit (T 90), %	24
Station	106+45.5	Plasticity Index (T 90), %	2
Depth	35.0-35.7	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296588	**BB-LCB-102/6D**	**GEOTECHNICAL (DISTURBED)**	**8/1/2018**	**8/16/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **106+86.6** Offset, ft: **5.4** **LT** Dbfg, ft: **25.0-27.0**

WIN/Town **022248.00 - LOVELL** 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]	**99.9**
No. 4 [4.75 mm]	**99.9**
No. 10 [2.00 mm]	**99.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**98.2**
[0.0229 mm]	**97.7**
[0.0147 mm]	**95.1**
[0.0088 mm]	**90.0**
[0.0069 mm]	**77.1**
[0.0054 mm]	**59.1**
[0.0028 mm]	**48.8**
[0.0013 mm]	**20.6**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**27**
Plastic Limit (T 90), %	**20**
Plasticity Index (T 90), %	**7**
Specific Gravity, Corrected to 20°C (T 100)	**2.62**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**35.5**

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		

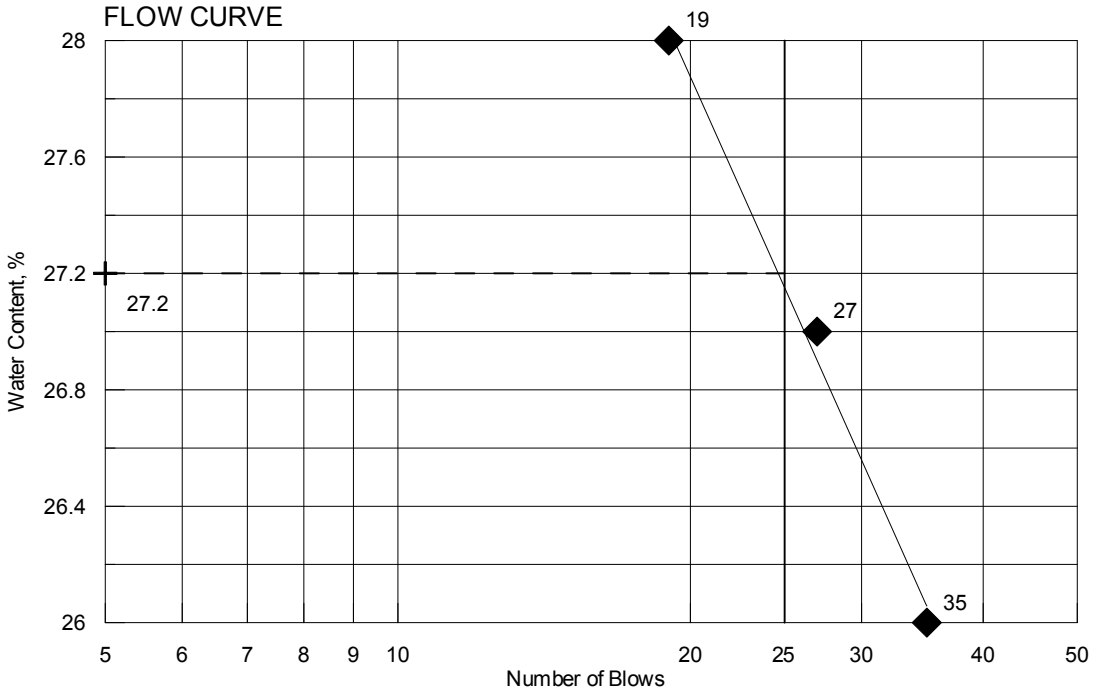
Comments:

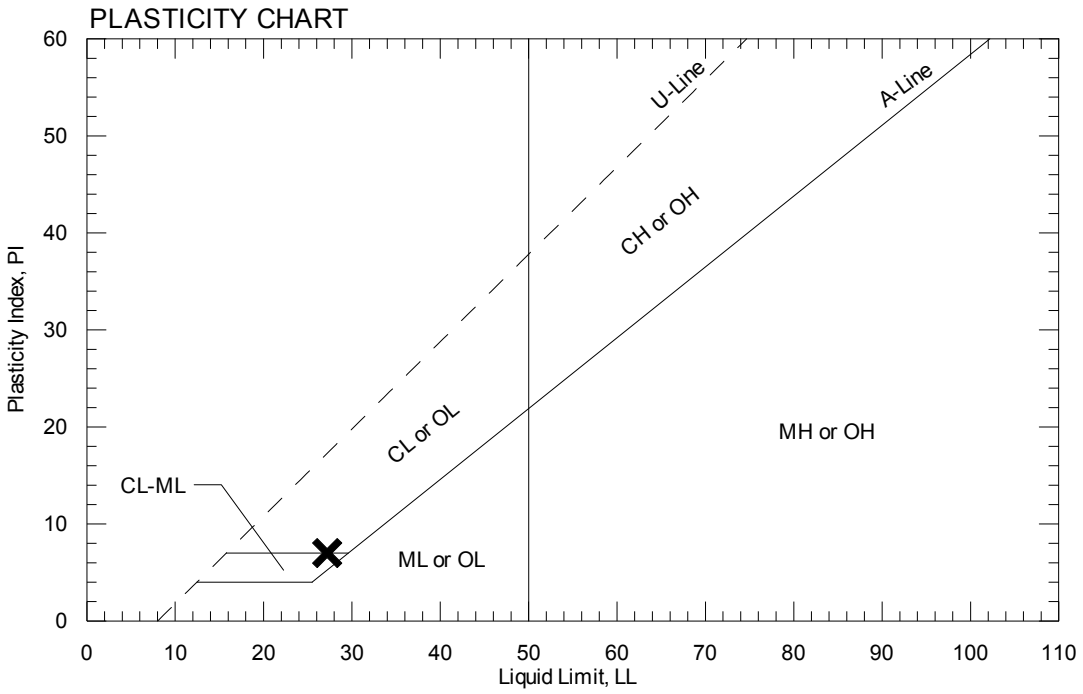
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/27/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Lovell	Reference No.	296588
WIN	022248.00	Water Content, %	35.5
Sampled	8/1/2018	Liquid Limit @ 25 blows (T 89), %	27
Boring No./Sample No.	BB-LCB-102/6D	Plastic Limit (T 90), %	20
Station	106+86.6	Plasticity Index (T 90), %	7
Depth	25.0-27.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296589	**BB-LCB-102/7DA**	**GEOTECHNICAL (DISTURBED)**	**8/1/2018**	**8/16/2018**

Sample Type: **GEOTECHNICAL** Location:

Station: **106+86.6** Offset, ft: **5.4** **LT** Dbfg, ft: **30.0-30.5**

WIN/Town **022248.00 - LOVELL**

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]	**99.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**92.1**
[0.0246 mm]	**88.9**
[0.0165 mm]	**81.2**
[0.0105 mm]	**68.6**
[0.0079 mm]	**55.9**
[0.0058 mm]	**40.6**
[0.0030 mm]	**27.9**
[0.0013 mm]	**10.2**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**28**
Plastic Limit (T 90), %	**24**
Plasticity Index (T 90), %	**4**
Specific Gravity, Corrected to 20°C (T 100)	**2.72**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**37.7**

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

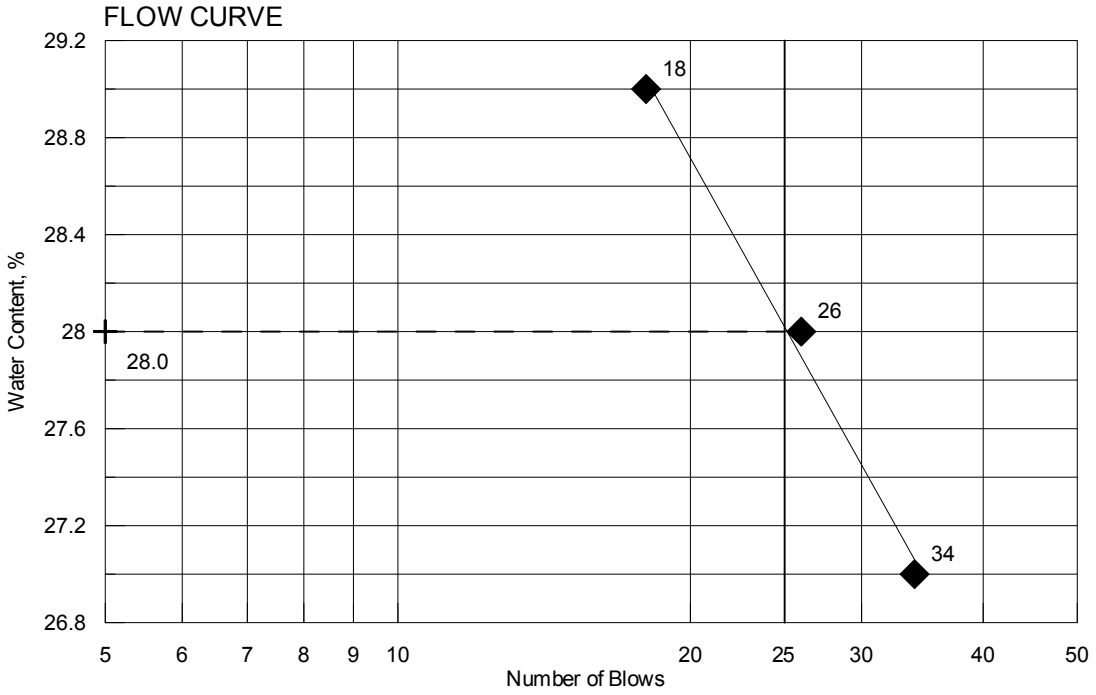
Comments:

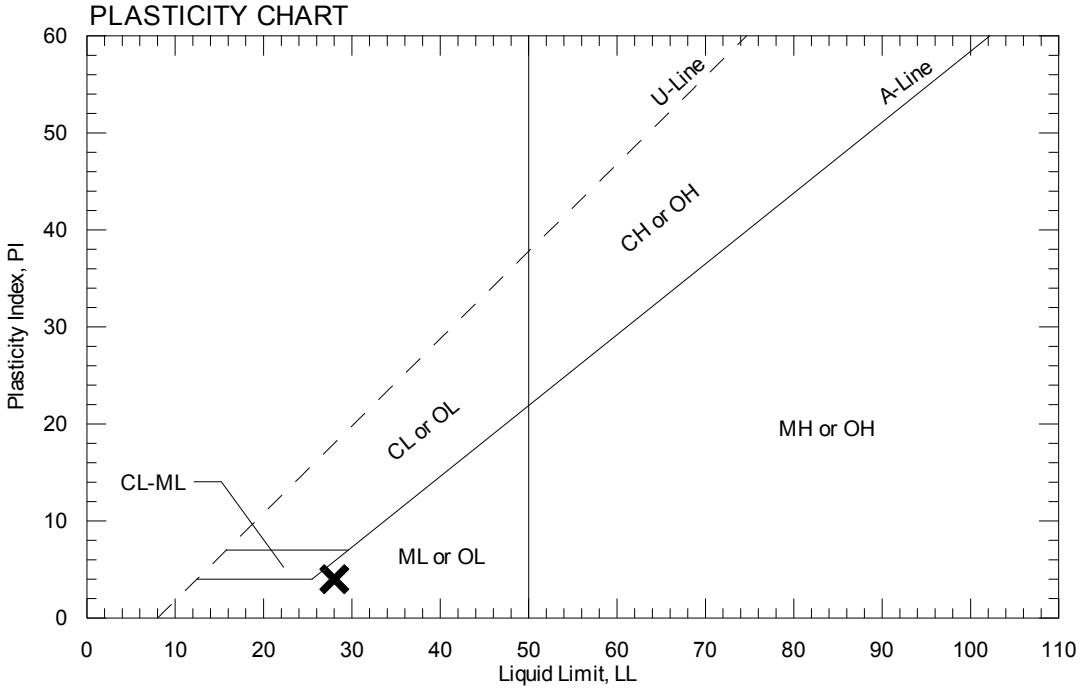
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **8/27/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Lovell	Reference No.	296589
WIN	022248.00	Water Content, %	37.7
Sampled	8/1/2018	Liquid Limit @ 25 blows (T 89), %	28
Boring No./Sample No.	BB-LCB-102/7DA	Plastic Limit (T 90), %	24
Station	106+86.6	Plasticity Index (T 90), %	4
Depth	30.0-30.5	Tested By	BBURR





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	$\gamma := 125 \cdot pcf$
Internal friction angle	$\phi := 32 \cdot deg$
Cohesion	$c := 0 \cdot psf$

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

Reference: Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2
Formula for normally consolidated soils.

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

$$K_o = 0.47$$

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

Outlet walls free to rotate - Active Earth Pressure - Rankine Theory

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

For cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

3.4 Construction Loads

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

3.5 Railroad Loads

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

3.6 Earth Loads

3.6.1 General

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

Table 3-3 Material Classification

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

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

Fang, Foundation Engineering Handbook 2nd ed. p. 224

6.1 AT-REST LATERAL PRESSURES

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

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

$$\sigma'_x = K_0(\sigma'_z) \tag{6.1}$$

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

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

$$\boxed{K_0 = 1 - \sin\phi'} \tag{6.2}$$

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

[a] Unloading cycle.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

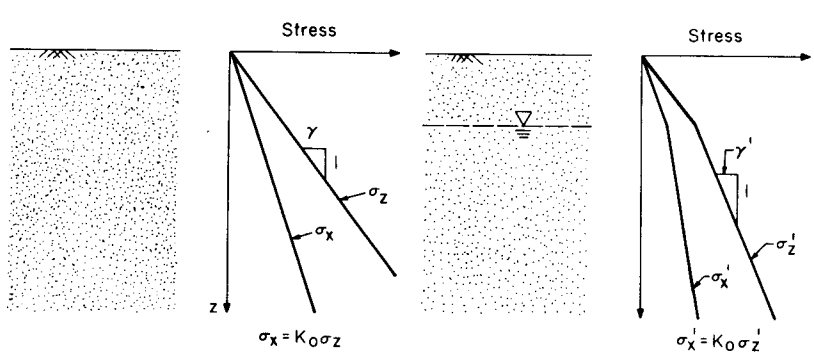


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

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 $\beta > 0°$, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

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

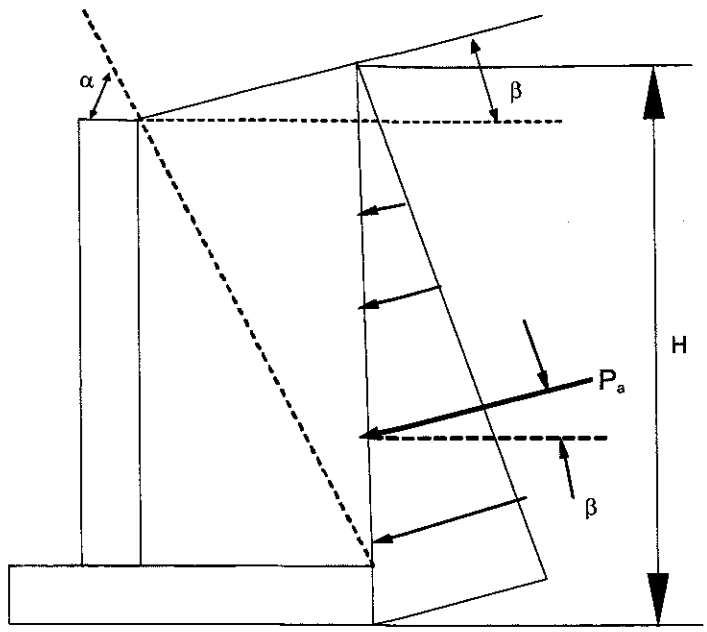


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

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. Lab test data
2. Soil strength properties based on correlations to SPT N-values

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 3 feet. Include 2 foot crushed stone mat as Foundation Footing, therefore Foundation Depth is 5.0 feet
2. The proposed bottom of slab bearing elevation slopes from El. 366.0 at inlet to 365.0 at outlet (BOF in Alluvium).
3. Bearing elevation of crushed stone mat is at approx. El. 364.0 at inlet and 363.0 at outlet (BOF in Alluvium).
4. Foundation Soils are Stream Alluvium: Soft Sandy Silt (N=3 bpf, CL) to Loose, fine Sand (N=5 bpf, SM).
5. Proposed precast concrete box has a span of 15 feet and a base width of 17 feet wide. Use dimensions of geosythetic wrapped crushed stone mat as the Foundation Width, 17 + 3 = 20 feet
6. 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
Fine sand, silty or clayey medium to fine sand (SM)	Loose	2-4	3

The Alluvial Stream deposit ranges from soft to loose in consistency. Recommend 2 ksf to limit settlement to 1.0 inch for Service Limit State Loads based on presumptive values - however based on settlement calculations, limit Service Limit State Load to 1.5 ksf.

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

Given:
1. Lab test data
2. Soil strength properties based on correlations to SPT N-values

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 3 feet. Include 2 foot crushed stone mat as Foundation Footing, therefore Foundation Depth is 5.0 feet
2. The proposed bottom of slab bearing elevation slopes from El. 366.0 at inlet to 365.0 at outlet (BOF in Alluvium).
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The Alluvial Stream deposit ranges from soft to loose in consistency. Recommend 2 ksf to limit settlement to 1.0 inch for Service Limit State Loads based on presumptive values - however based on settlement calculations, limit Service Limit State Load to 1.5 ksf.

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

Assumed Foundation Width, Depth, and Water Surface

$B := 20ft$

$D_f := 5.0 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

$\gamma_{backfill} := 125 \cdot pcf$

Foundation soils (Stream Alluvium ranges from a Sandy Silt to a Fine Sand:

$\gamma 1_d := 93 \cdot pcf$	Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 Soft Clay and Loose Sand
$w_{sat} := 38.1\%$	Highest moisture contents of saturated samples near bearing elevation: BB-LCB-101 5D.
$\gamma 1_{sat} := \gamma 1_d \cdot (1 + w_{sat})$	Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit weight relationships
$\gamma 1_{sat} = 128.4 \cdot pcf$	
$\phi := 0 \cdot deg$	Lambe and Whitman, Soil Mechanics, 1969 Figure 11.14 N vs. phi
$c_1 := 350 \cdot psf$	BB-LCB-101 5D

2a. Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - c soil.

Shape Factors for strip footing

$s_\gamma := 1.0$ $\quad s_c := 1.0$ $\quad$ Bowles 5th Ed., p. 220 Table 4-1

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) ϕ =0

$N_c := 5.14$ $\quad N_q := 1$ $\quad N_\gamma := 0$

Nominal Bearing Resistance per Terzaghi equation

$q := D_f \cdot (\gamma_{backfill} - \gamma_w)$ $\quad q = 0.313 \cdot ksf$ $\quad$ Das Principles of Foundation Engineering 7th Ed. p. 142: Eq. 3.16 Water table modification

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

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

$q_n = 2.1 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.50$

$q_r := q_n \cdot \varphi_b$

$q_r = 1.1 \cdot ksf$

2b. Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ soil.

$\phi := 27 \cdot deg$

BB-LCB-102 5D, N = 5 bpf

$c_1 := 0 \cdot psf$

Terzaghi and Vesic Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 24.04$ $N_q := 13.3$ $N_\gamma := 14.6$

Nominal Bearing Resistance per Terzaghi equation

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

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

$q_n = 13.8 \cdot ksf$

Factored Bearing Resistance for strength limit states

$\varphi_{b\_sand} := 0.45$

LRFD Table 10.5.5.2.2-1

$q_r := q_n \cdot \varphi_{b\_sand}$

$q_r = 6.2 \cdot ksf$

3. AASHTO LRFD - Estimate the nominal and factored bearing resistance at the Strength Limit State:

3a. Model: Cohesive Stream Alluvium subunits $\phi = 0$

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

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

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

Foundation Soil - ϕ=0

$\phi := 0 \cdot deg$

$c_1 := 350 \cdot psf$ BB-LCB-101 5D, N = 3 bpf

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

$L := 52ft$

$s_\gamma := 1 - 0.4 \cdot \left(\frac{B}{L}\right)$ $s_\gamma = 0.846$

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

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

$i_c := 1.0$ $\quad i_\gamma := 1.0$ $\quad i_q := 1.0$

Depth correction factors - only used when soils above the footing bearing elevation are as competent as the soils beneath the footing level. Otherwise 1.0

LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.25$$

Therefore :

$$d_q := 1.0$$

Terms

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

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

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

$N_{cm} = 5.14$ $\quad N_{\gamma m} = 0$ $\quad N_{qm} = 1$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c_1 \cdot N_{cm} + \gamma_{above} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$$q_n = 2.1 \cdot ksf$$

Factored Bearing Resistance for strength limit states

$\varphi_b = 0.5$ $\quad$ LRFD Table 10.5.5.2.2-1

$$q_r := q_n \cdot \varphi_b$$

$$\boxed{q_r = 1.1 \cdot ksf}$$

3b. Model: Fine Sandy Stream Alluvium subunits ϕ = 27 degrees

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

Foundation Soil

$\phi := 27 \cdot deg$ BB-LCB-102 5D, N = 5 bpf

$c_1 := 0 \cdot psf$

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

$N_c := 23.9$

$N_q := 13.2$

$N_\gamma := 14.5$

Terms

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

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

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

$N_{cm} = 23.9$ $N_{\gamma m} = 12.269$ $N_{qm} = 13.2$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c_1 \cdot N_{cm} + \gamma_{above} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma 1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$q_n = 12 \cdot ksf$

Factored Bearing Resistance - Model: Sand Stream Alluvium subunits

$\varphi_{b\_sand} := 0.45$ LRFD Table 10.5.5.2.2-1

$q_r := q_n \cdot \varphi_{b\_sand}$

$\boxed{q_r = 5.4 \cdot ksf}$

Recommend a factored bearing resistance of 3 ksf for precast box culvert constructed on variable Stream Alluvium (Soft, Sandy SILT and Loose, Fine SAND)

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.

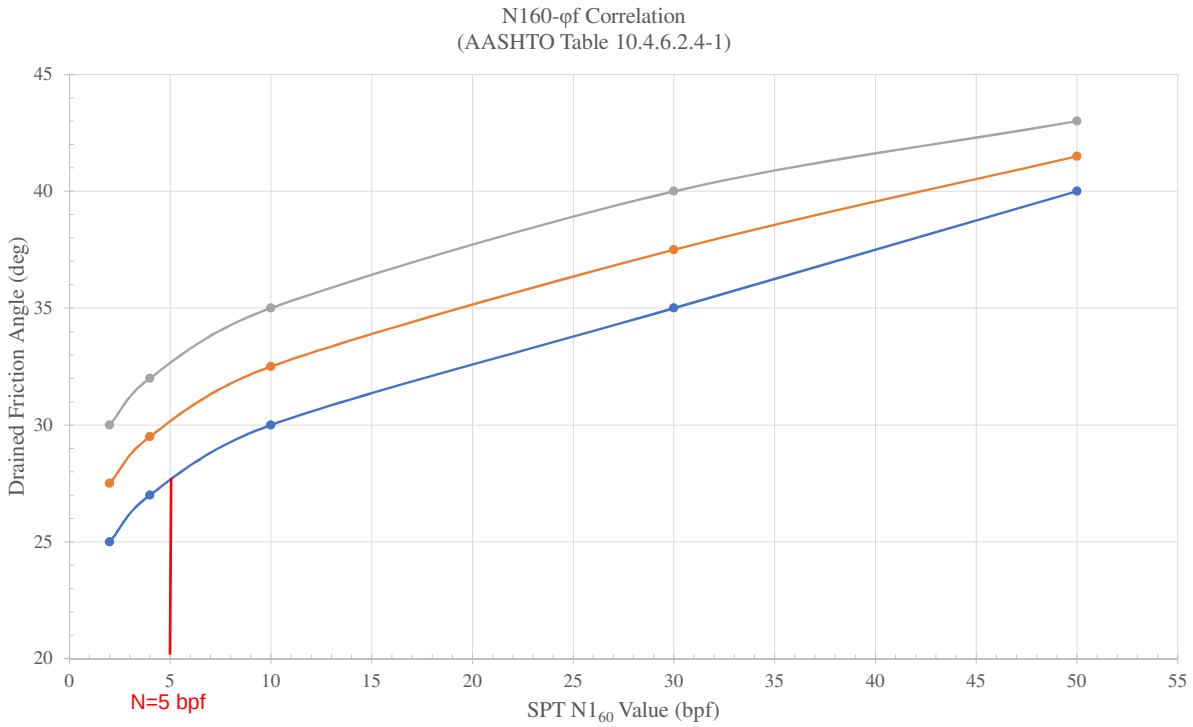
Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

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}	$\left(\frac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\frac{G_s\gamma_w}{1+\left(\frac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\frac{e}{w_{sat}}\right)\left(\frac{1+w_{sat}}{1+e}\right)\gamma_w$
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}	$n\left(\frac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\frac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\frac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\frac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\frac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

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

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

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



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3.4 Construction Loads

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

3.5 Railroad Loads

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

3.6 Earth Loads

3.6.1 General

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

Table 3-3 Material Classification

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

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

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:

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

in which:

$$N_{cm} = N_c s_c i_c \tag{10.6.3.1.2a-2}$$

$$N_{qm} = N_q s_q d_q i_q \tag{10.6.3.1.2a-3}$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \tag{10.6.3.1.2a-4}$$

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

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

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

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

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

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

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

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

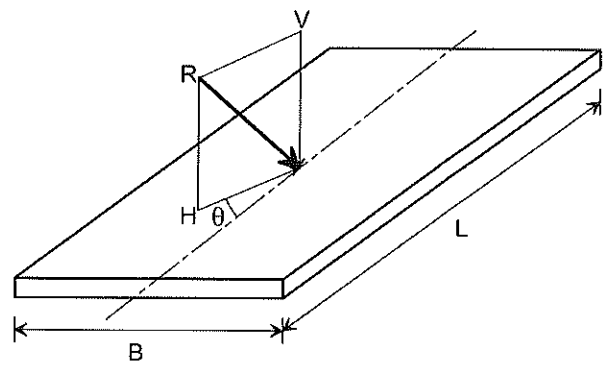


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

AASHTO LRFD Bridge Design Specification, 7th ed. 2014

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).

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

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.

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.

C10.5.5.2.2

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

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

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

Modulus of Subgrade Reaction

Objective:
Estimate the modulus of subgrade reaction for the box culvert base slab design.

Given:
Boring logs, SPT N-values, lab data

Assumptions:
1. The box culvert is embedded 5 feet into the streambed, from the bottom of the Crushed Stone to the top of the Special Fill.
2. The proposed bearing elevation of base slab slopes from Elev. 366.0 at inlet to 365.0 at outlet.
3. Proposed finished grade approximately matches existing.
4. Proposed precast concrete box is 17 feet wide and 52 feet long (excluding slab connecting slope tapered outlet and inlet sections).
5. Model subsurface conditions based on Stream Alluvium: Loose, fine SAND.
6. The bottom of the box culvert will be submerged for the structure's design life.
7. Sample BB-LCB-101, 5D is taken as the representative sample at the bearing elevation.

Published values of subgrade modulus

Published values of subgrade modulus for Loose Fine Sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction
Subgrade of Loose Fine Sand, Average: **k_s = 38 pci**

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Range of modulus of subgrade reaction
Loose Fine SAND, K_{v1}, 46 pci, per Note 2 reduce by 1/2. Kv1= 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
Loose, saturated Sands,: **$k_{0.3}$ (k_1) = 37** - 55 pci. Use average of 46

Adjust Published values for dimensions of base slab

Published range for loose, saturated sand subgrade is 17 - 46 pci.

Assign a subgrade modulus of 37 pci based on Bowles (38 pci) and lower bound of saturated, loose sand per Das (37 pci).

Value of k_{s1} is for a 1 ft x 1 ft plate. Adjust to the dimensions of the crushed stone bedding layer (Width B = 20, Length L = 90).

Square to rectangle base adjustment:

$k_{s1} := 37pci \qquad B := 20ft \qquad L := 90ft$

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

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

$k = 27.4 \cdot pci$

Recommend a subgrade modulus of 27 pci

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \tag{9-10}$$

r either horizontal or vertical members
for depth variation
terest below ground
give k_s the best fit (if load test or other data are available)

ation may be zero; at the ground surface A_s is zero for a lateral k_s
> 0. For footings and mats (plates in general), $A_s > 0$ and $B_s \cong 0$.
used with the proper interpretation of the bearing-capacity equa-
e d_i factors dropped) to give

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

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

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

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

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

depth of interest, say, the length of a pile
h of interest
timate of the exponent

to estimate a value of k_s to determine the correct order of magnitude
btained using one of the approximations given here. Obviously if a
three times larger than the table range indicates, the computations
possible gross error. Note, however, if you use a reduced value of
r 12 mm) instead of 0.0254 m you may well exceed the table range.
putational error (or a poor assumption) is found then use judgment
table values are intended as guides. The reader should not use, say,
ven as a "good" estimate.
d in Fig. 9-9c (and used in your diskette program FADBEMLP as
stimated at some small value of, say, 6 to 25 mm, or from inspection
if a load test was done. It might also be estimated from a triaxial
ltimate" or at the maximum pressure from the stress-strain plot.
$\epsilon_{\max}$ compute

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

TABLE 9-1
Range of modulus of subgrade reaction k_s

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

Use values as guide and for comparison when using approximate equations

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

Soil	k_s, kN/m³	ks, lb/in^3
Loose sand	4800–16 000	18 - 59
Medium dense sand	9600–80 000	35 - 295
Dense sand	64 000–128 000	236 - 472
Clayey medium dense sand	32 000–80 000	118 - 295
Silty medium dense sand	24 000–48 000	88 - 177
Clayey soil:		
$q_a \le 200$ kPa	12 000–24 000	44 - 88
$200 < q_a \le 800$ kPa	24 000–48 000	88 - 177
$q_a > 800$ kPa	> 48 000	> 177

Average
38.5 pci

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

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

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

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

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

For **center:**

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

From these we may write

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

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

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

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

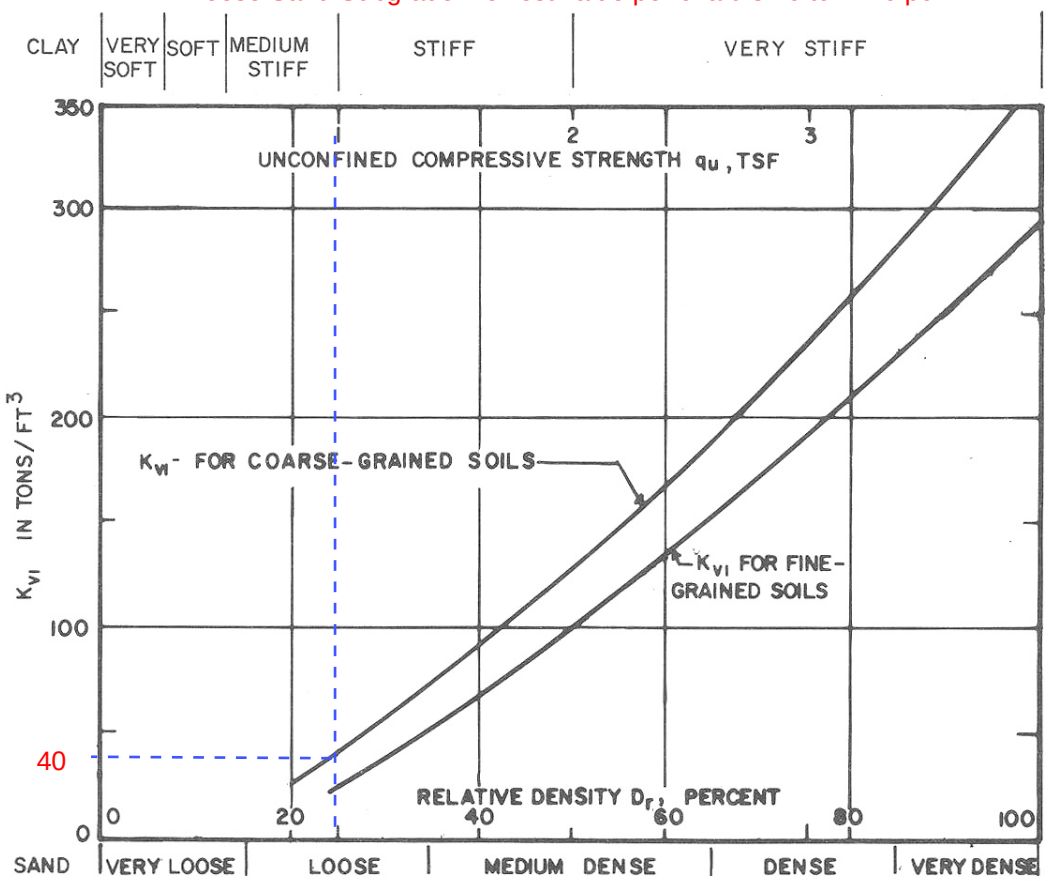
You should note that k_s does not depend on the contact pressure of the base q_o.

For **corner:**

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

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

Loose Sand Subgrade - lowest value per chart is 40 tcf = 46 pci



Loose, Alluvial Sand Subgrade

DEFINITIONS

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

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

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

93*1.15= 107 pci
38*1.15= 44 pci

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)
SHALLOW FOOTINGS D ≦ B
FOR B ≦ 20 FT:

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

FOR B ≧ 40 FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION D ≧ 5B
FOR B ≦ 20 FT:

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

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{vl} 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 $K_{vl}/2$ IN COMPUTING SETTLEMENT

Loose, Alluvial Sand Subgrade (saturated) k=46 pci/2 = 23 pci

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

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

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

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

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

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

or

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

where

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

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

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

Soil type	$k_{0.3}(k_1)$ MN/m³	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

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

Foundations on Sandy Soils

For foundations on sandy soils,

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

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

Foundations on Clays

For foundations on clays,

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

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

Method 1: For rectangular foundations having dimensions of $B \times L$ (for similar soil and q),

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

where

k = coefficient of subgrade modulus of the rectangular foundation $(L \times B)$

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

Settlement

Settle3D Analysis Information
Lovell Creek Bridge

Project Settings

Document Name	Lovell delta q=880 psf STRI-r1.s3z
Project Title	Lovell Creek Bridge
Analysis	1-year consolidation and secondary compression
Author	Krusinski
Company	MaineDOT
Date Created	10/19/2018, 1:21:00 PM

Comments

15' span - L=52 ft, B=17
Net Change in q due to STR I at z=17 ft is 880 psf
Soil profile model based on BB-LCB-101

Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	

Stage Settings

Stage #	Name	Time [years]
1	Immediate	0
2	Consolidation	1
3	Long-term	50

Results

Time taken to compute: 0.0648034 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.57741
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.57741
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.88
Effective Stress [ksf]	0	3.6172
Total Stress [ksf]	0	6.59962
Total Strain	0	0.0123866
Pore Water Pressure [ksf]	0	2.98242
Excess Pore Water Pressure [ksf]	0	0.88
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.009375	4.98033
Over-consolidation Ratio	1	2.5
Void Ratio	0	0.895485
Permeability [ft/y]	0	0.103338
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.829716

Stage: Consolidation = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.26634
Consolidation Settlement [in]	0	1.68893
Immediate Settlement [in]	0	1.57741
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.88
Effective Stress [ksf]	0	3.48679
Total Stress [ksf]	0	6.59962
Total Strain	0	0.0387139
Pore Water Pressure [ksf]	0	3.11283
Excess Pore Water Pressure [ksf]	0	0.336027
Degree of Consolidation [%]	0	66.0392
Pre-consolidation Stress [ksf]	0.009375	4.98033
Over-consolidation Ratio	1	1.8463
Void Ratio	0	0.894942
Permeability [ft/y]	0	0.103338
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	92.2319
Undrained Shear Strength	0	0.881501

Stage: Long-term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	5.4494
Consolidation Settlement [in]	0	2.55747
Immediate Settlement [in]	0	1.57741
Secondary Settlement [in]	0	1.31452
Loading Stress [ksf]	0	0.88
Effective Stress [ksf]	0	3.82282
Total Stress [ksf]	0	6.59962
Total Strain	0	0.0537521
Pore Water Pressure [ksf]	0	2.7768
Excess Pore Water Pressure [ksf]	0	6.15721e-014
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.009375	4.98033
Over-consolidation Ratio	1	1.8463
Void Ratio	0	0.868614
Permeability [ft/y]	0	0.103338
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.881501

Loads

1. Rectangular Load

Length	52 ft
Width	17 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	884 ft^2
Load	0.88 ksf
Depth	17 ft
Installation Stage	Immediate = 0 y

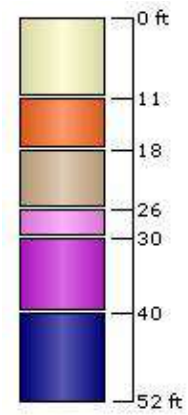
Coordinates

X [ft]	Y [ft]
-19.566	6.191
32.434	6.191
32.434	23.191
-19.566	23.191

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Fill	11	0	No
2	Loose Silty Sand (Alluvium)	7	11	No
3	Soft Sandy Silt & Loose Fine Sand (Alluvium)	8	18	No
4	Soft Clayey Silt	4	26	No
5	Medium Stiff Clayey Silt	10	30	No
6	Dense Till	12	40	No



Soil Properties

Property	Fill	Loose Silty Sand (Alluvium)	Soft Sandy Silt & Loose Fine Sand (Alluvium)	Soft Clayey Silt	Medium Stiff Clayey Silt	Dense Till
Color						
Unit Weight [kips/ft^3]	0.125	0.092	0.083	0.073	0.083	0.13
Saturated Unit Weight [kips/ft^3]	0.115	0.128	0.115	0.102	0.121	0.135
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled	Enabled
Es [ksf]	450	150	146	55.346	146.198	730
Esur [ksf]	1800	600	600	200	600	2000
Primary Consolidation	Disabled	Disabled	Disabled	Enabled	Enabled	Disabled
Material Type				Non-Linear	Non-Linear	
Cc				0.4	0.3	
Cr				0.04	0.03	
e0				0.9	0.9	
OCR	1	1	2.5	1	1	1
Cv [ft^2/y]				36.5	36.5	
B-bar				1	1	
Secondary Consolidation	Disabled	Disabled	Disabled	Mesri	Mesri	Disabled
Ca/Cc				0.04	0.04	
Undrained Su A [kips/ft2]	0	0	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

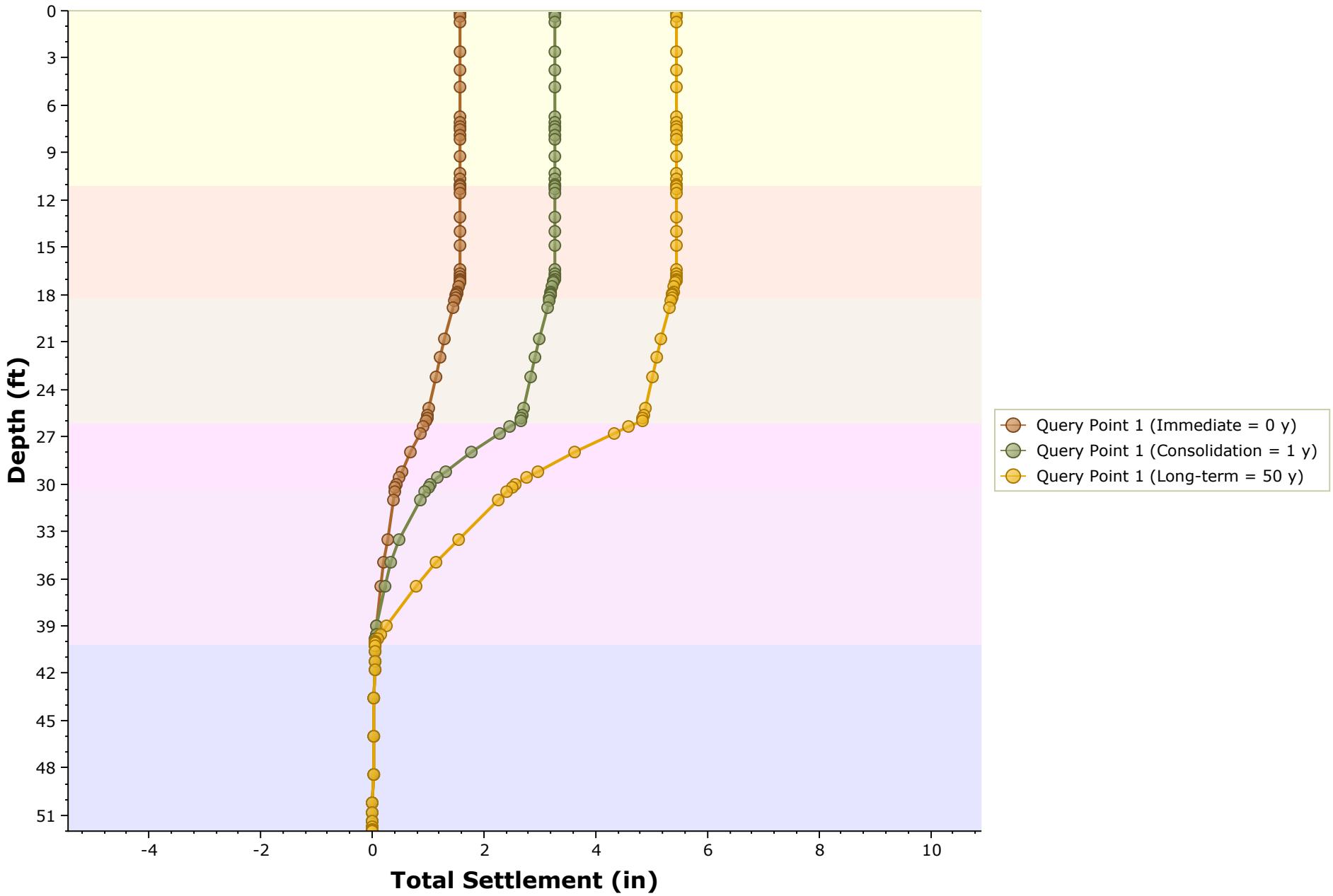
Piezometric Line Entities

ID	Depth (ft)
1	7.5 ft

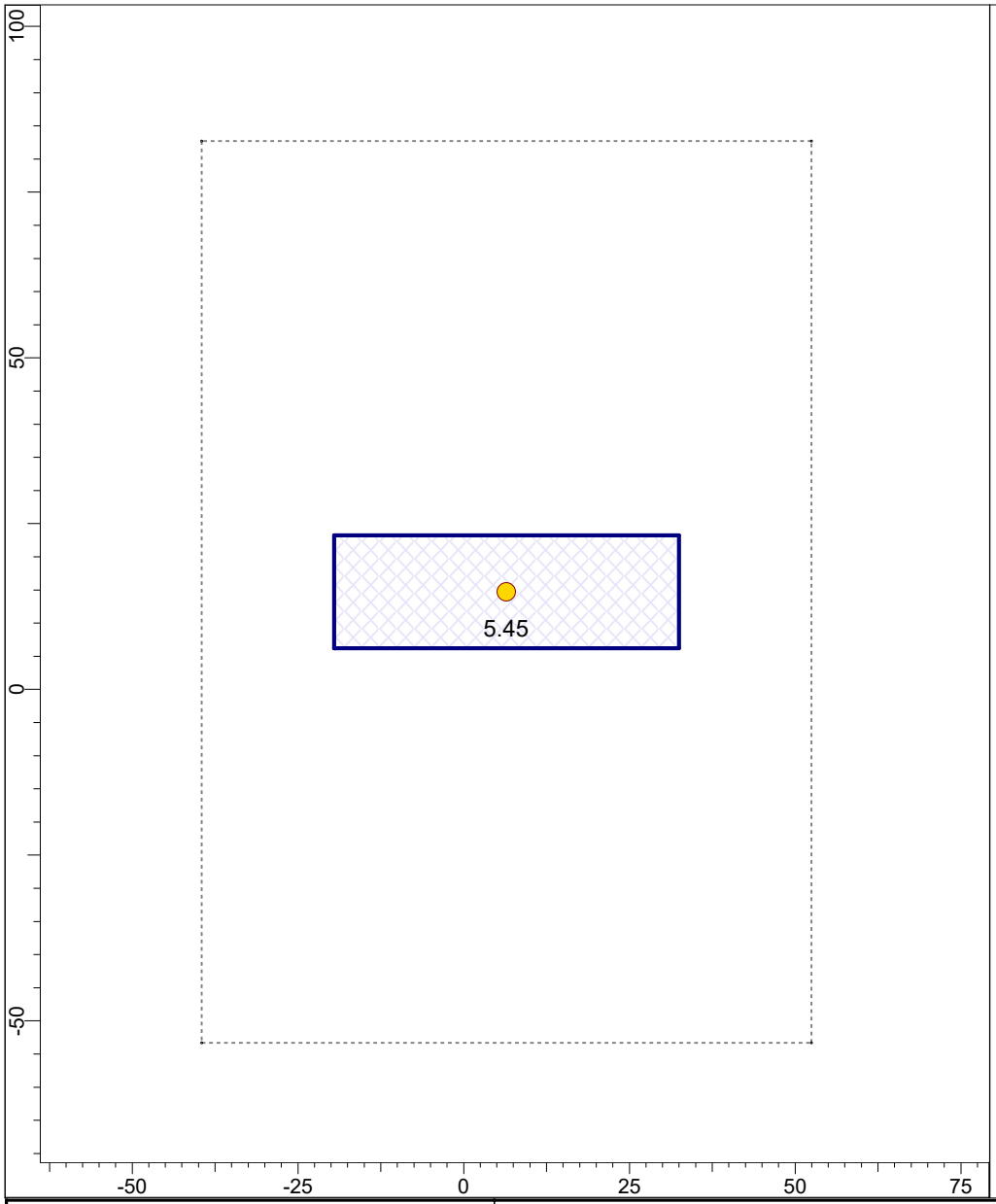
Query Points

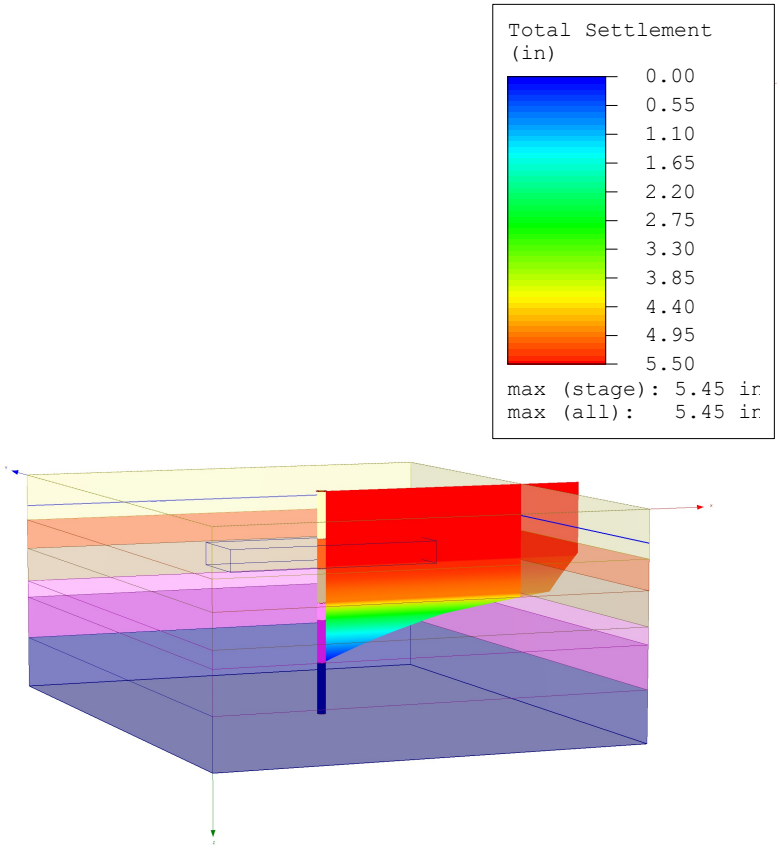
Point #	(X,Y) Location	Number of Divisions
1	6.434, 14.691	Auto: 77

Total Settlement vs. Depth



Reference Stage: None





rocscience SETTLE3D 3.015	*Project*	Lovell Creek Bridge	
	Analysis Description	1-year consolidation and secondary compression	
	Drawn By	Krusinski	*Company* MaineDOT
	Date	10/19/2018, 1:21:00 PM	*File Name* Lovell delta q=880 psf STRI-r1.s3z

Frost

MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: Lovell, **Maine**
Case 1 - saturated fine sands W=20%

DFI = 1400

Depth of Frost Penetration

$d_1 := 65.5\text{in}$ for w = 20%

$d_1 = 5.458\,\text{ft}$

Recommendation: 5.5 feet for design of foundations constructed on soil

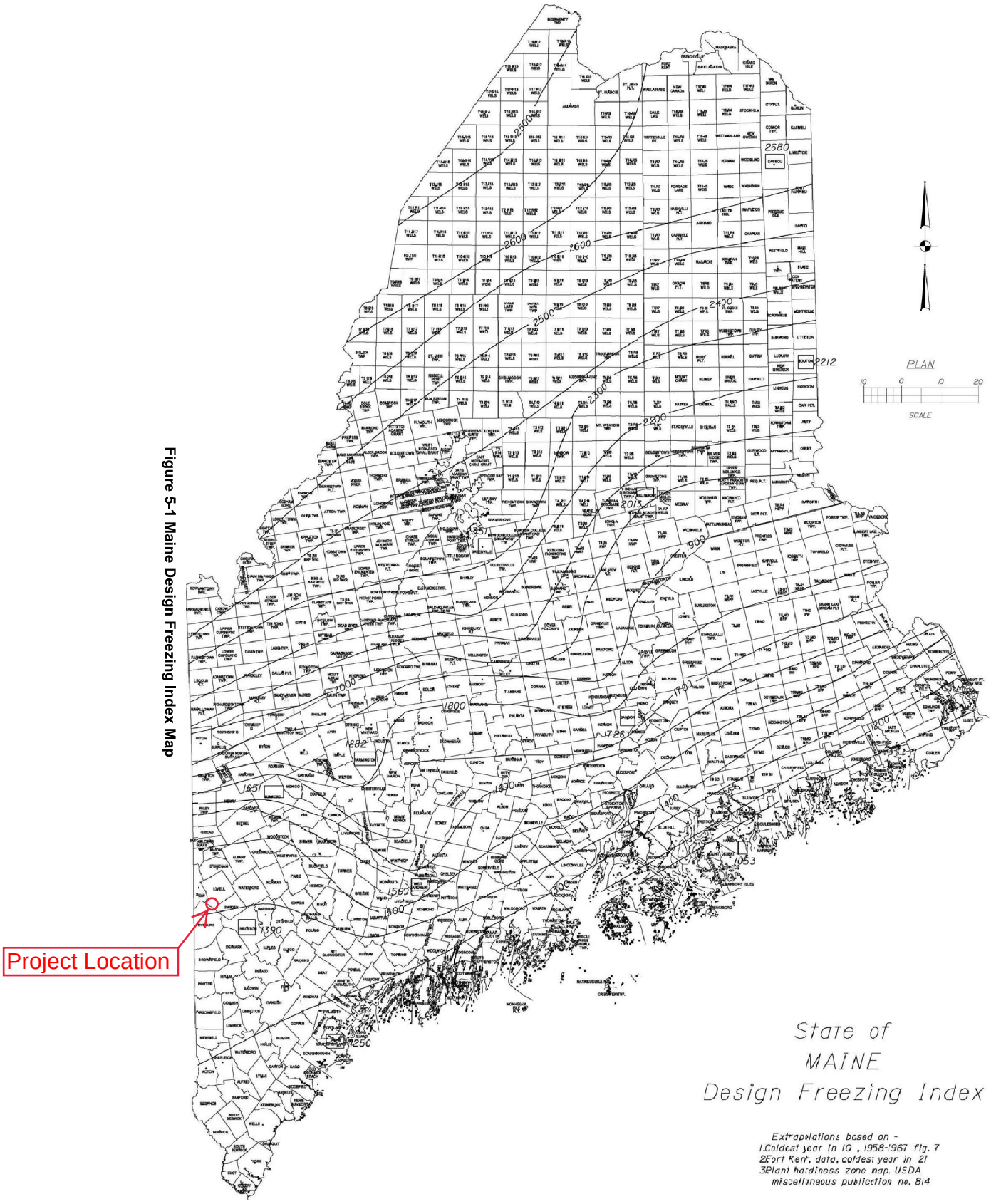


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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