MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

FRAZIER BRIDGE
STATE ROUTE 125 OVER DEARING BROOK
LISBON, MAINE

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Androscoggin County — Soils Report 2020-54
WIN 23118.00 — Bridge No. 3954

Federal Project No. 2311800
December 30, 2020

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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 Frazier Bridge which carries State Route 125 (Main Street) over Dearing Brook in Lisbon, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design parameters, and construction recommendations for the new box culvert.

The existing Frazier Bridge was constructed in 1952. The structure consists of a single steel pipe arch with a 14-foot span and bearing on a mat of timber grillage. According to the 2019 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the culvert is in poor condition and is rated a 4. The channel is rated a 6 with bank slump.

The proposed replacement structure is a 16-foot span and 6-foot rise, approximately 70-foot long, precast concrete box culvert. The box culvert shall have 1-foot tall precast headwalls and 2-foot deep toe walls at the inlet and outlet. The upstream and downstream ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) side slopes. The box culvert invert will be embedded 2 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. To provide a stable subgrade for the installation of the box culvert, a 2-foot-thick bed of crushed stone wrapped in geotextile and reinforced with geogrid is recommended.

The new box culvert will be located on nearly the same horizontal and vertical alignment as the existing bridge. The culvert skew will be increased to 25° to improve alignment with the stream and improve flow. Maintenance of traffic during construction will be accomplished by staged construction with alternating one-way traffic or a short-term bridge closure with a local detour.

2.0 GEOLOGIC SETTING

Frazier Bridge carries State Route 125 (Main Street) over Dearing Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Lisbon Falls North Quadrangle, Open-File No. 03-14 (2003), indicates the surficial soils in the vicinity of the bridge project consist of the Presumpscot Formation and Marine Nearshore Deposits. The Presumpscot Formation is typically a massive to laminated, silty clay. Marine Nearshore Deposits are gravel, sand, clay and silt deposited as a result of wave activity in shallow-marine environments.

The MGS Bedrock Geology Map of the Lewiston Quadrangle Open-File No. 83-4 (1983) maps the bedrock at the project site as Granofels of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

Three test borings explored subsurface conditions at the project location. Borings BB-LFB-101 and BB-LFB-103 were drilled behind the southeast corner of the existing pipe arch. Boring BB-LFB-102 was drilled behind the northwest corner of the pipe arch. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled in January and February 2020 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.

Borings were performed by using a combination of solid stem auger and cased wash boring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. 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" in June 2019. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.886 to the raw field N-values. This hammer efficiency factor (0.886) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

In-situ vane shear tests were conducted in soft soil deposits with a 55x110 mm Geonor vane to measure the shear strength of the soils. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, and logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program and surveyed.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of three standard grain size analyses with natural water content, eight grain size analyses with hydrometer and natural water content, seven Atterburg limit tests, and two consolidation 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 and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, Reworked Alluvium, Marine Nearshore Deposits, and Glaciomarine Clay. 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 fill layer was encountered in the test borings. The thickness of the fill unit encountered was approximately 5.5 to 8.0 feet. The unit generally consisted of brown, dry to moist SAND, little to some gravel, and trace to little silt.

Corrected SPT N-values in the fill layer ranged from 10 to 13 blows per foot (bpf), indicating the fill unit is loose to medium dense in consistency.

5.2 Stream Alluvium

Stream Alluvium was encountered beneath the fill soils in the borings. The thickness of the alluvial deposit encountered was approximately 4.0 to 4.5 feet. The alluvial deposit encountered generally consisted of:

- Grey to grey brown, damp to moist, SAND, some silt;
- Sandy SILT, trace gravel and organics, roots and wood.

A corrected SPT N-value of 16 bpf was recorded indicating the deposit is medium dense in consistency. One grain size analysis conducted on a sample from the alluvium indicated the soil is classified as A-4 under the AASHTO Soil Classification System and CL under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 31 percent.

5.3 Marine Nearshore Deposits

Marine Nearshore Deposits were encountered beneath the stream alluvium. The thickness of the deposit encountered was approximately 29 to 30 feet. The Marine Nearshore Deposits encountered generally consisted of layered:

- Grey brown, brown and grey, fine to medium SAND, some to little silt, trace clay;
- Grey, Silty, fine SAND, trace clay;
- Grey SILT, trace clay, trace fine sand.

Corrected SPT N-values of 7 to 15 bpf were recorded in the silt layers indicating the silt deposits are medium stiff to stiff in consistency. Corrected SPT N-values of 7 to 30 bpf were recorded in the sand layers indicating the sand units are loose to medium dense in consistency. Five grain size analyses conducted on samples from the marine nearshore deposits indicated the material is classified as A-4 and A-2-4 under the AASHTO Soil Classification System and CL, SM, SC-SM and SP-SM under the USCS. The natural water contents of the samples tested ranged from approximately 22 to 29 percent.

Two Atterberg limit tests conducted on samples obtained from the silt layers indicated the silt subunits are non-plastic.

5.4 Glaciomarine Deposit

A Glaciomarine Deposit was encountered beneath the Marine Nearshore Deposits in borings BB-LFB-101 and BB-LFB-103. The thickness of the Glaciomarine Clay was approximately 16 and 35 feet when the borings were terminated. The Glaciomarine Clay encountered generally consisted of:

- Grey, wet, Clayey SILT, trace fine sand;
- Grey, wet, SILT, some to little clay, trace fine sand;
- Dark grey, wet, Silty CLAY, trace sand.

Corrected SPT N-values of 1 to 3 bpf were recorded and multiple SPT samples were retrieved with only the weight of the rods (WOR) indicating the Glaciomarine Deposits are very soft in consistency. Grain size analyses conducted on samples from the Glaciomarine Clay indicated the material is classified as A-4 and A-6 under the AASHTO Soil Classification System and CL and CL-ML under the USCS. The natural water contents of the samples tested ranged from approximately 27 to 36 percent.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the Glaciomarine deposits. A 55 x 110 mm vane was used. Six (6) successful vane shear tests conducted within the glaciomarine deposit showed measured undisturbed undrained shear strengths ranging from approximately 625 psf to 1027 psf, indicating that the deposit is medium stiff to stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 89 to 223 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the silt-clay deposit has a sensitivity ranging from 4.6 to 8.0 and is classified as sensitive to very sensitive.

Atterberg limits tests were conducted on five samples of the Glaciomarine Clay and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-LFB-101, 9D	SILT	26.6	Non-Plastic			
BB-LFB-101, 10D	Clayey SILT	36.0	32	23	9	1.4
BB-LFB-101, 12D	Silty CLAY	33.3	34	22	12	0.9
BB-LFB-103, 1U	SILT	31.2	Non-Plastic			
BB-LFB-103, 2U	SILT, some clay	34.7	26	21	5	2.2

The plasticity indices of the samples indicate that the soils have slight to medium plasticity (Burmister, 1949). The natural water contents of the tested samples ranged from approximately 27 to 36 percent and liquid limits ranged from 26 to 34. The liquidity indices range from 0.9 to 2.2. Interpretation of these results indicates that the soils with liquidity indices of 1 or less are preconsolidated, while those 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 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. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as "quick."

All three borings were terminated within the Marine or Glaciomarine Deposits at depths ranging from 42 to 72 feet below the roadway surface (brs).

5.5 Groundwater

Groundwater was measured at depths ranging from approximately 3 to 5 feet brs upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured level may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Given the current span width of 14 feet, it was determined that either a steel arch culvert (replace-in-kind alternative) or a precast box culvert would satisfy the purpose and need of this bridge replacement project. A precast concrete box culvert was chosen because of the structure's long-term service life with minimal maintenance and ease and speed of construction. Therefore, the existing Frazier Bridge will be replaced with a 70.5-foot long precast concrete box culvert with a 16-foot span and 6-foot rise. The box will be embedded approximately 2 feet into the streambed and 2 feet of special fill will be placed inside to create a natural streambed.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 70.5-foot-long 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 2-foot tall inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 2 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. 2-foot thick riprap aprons should be constructed at the inlet and outlet and should be embedded a minimum of 6 inches into the streambed. The riprap aprons will be covered with the engineered streambed material to provide continuity of the natural streambed.

Due to the loose Marine Sands and the soft Glaciomarine Clays, it is recommended that the box culvert be constructed on a 2-foot thick layer of crushed stone reinforced with geogrid and wrapped in stabilization/reinforcement geotextile. The stabilization/reinforcement geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor (method of compaction approximating 97 percent of AASHTO T-108 maximum dry density).

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

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed 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 Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2017 with 2018 interims (LRFD).

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased 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 calculate earth loads and earth pressures from the soil envelope. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

The box culvert invert will be embedded 2 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. A 2-foot-thick bed of crushed stone wrapped in geotextile and reinforced with geogrid should be specified to provide a stable subgrade for the installation of the box culvert and mitigate long term consolidation settlement. 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.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1-foot thick by 1-foot high 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 sideslopes. The left and right outlet walls will share the same precast base slab. The sloped inlet and outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The inlet and outlet walls shall be designed to resist lateral earth pressures, vehicular loads and forces resulting from creep, temperature and shrinkage deformations of the concrete box culvert. The inlet and outlet walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet per LRFD Article 3.11.6.4. 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. Wingwalls sections that are independent of the box culvert and free to rotate should be designed using the Rankine active earth pressure coefficient, K_a, of 0.46 assuming a 2H:1V backslope. The active earth pressure coefficient will change if the backslope conditions are different.

7.1.3 Precast Concrete Inlet and Outlet Toe Walls

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

7.1.4 Bearing Resistance

To provide a stable subgrade and mitigate consolidation settlement, it is recommended that the precast concrete box culvert be bedded on a 2-foot-thick layer of crushed stone that is reinforced with geogrid and wrapped in stabilization/reinforcement geotextile placed on the native soil subgrade. The bearing elevation of the crushed stone mat will be approximately Elev. 112.3 at the inlet and 111.7 at the outlet.

For a precast concrete box culvert with a base width of 18 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 6 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 4 ksf. The service limit state 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 $0.3f'c$.

7.1.5 Modulus of Subgrade Reaction

Large span precast box culverts can be viewed similarly to a mat foundation. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 54.5 pounds per cubic inch (pci).

7.2 Subgrade Preparation for Box Culvert

The soils encountered in borings at the box subgrade elevation generally consisted of loose to medium dense Stream Alluvium and loose to medium dense Marine Nearshore Sands. Any unsuitable soils (i.e. low strength silts and clays, loose sands, and organic material), and all timber grillage, that may be encountered at the subgrade elevation, should be excavated down to expose competent, firm material and replaced with compacted granular borrow.

The saturated, loose sands 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;
- The box culvert shall be installed on a 2-foot thick layer of crushed stone be wrapped in stabilization/reinforcement geotextile.
- Hand-deploy the geotextile on the prepared soil subgrade prior to installing the geogrid-reinforced stone 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.

7.3 Settlement

The proposed box culvert will bear on a deposit of Alluvium and loose to medium dense, Marine Sand. The Marine Sands are underlain by a deep deposit of very soft to medium stiff Glaciomarine Clay. The soil deposits will undergo immediate and consolidation settlement in response to a net increase of vertical overburden pressure. Based on an estimated service limit state pressure of 2,750 psf for an 18-foot wide precast concrete box, combined elastic and consolidation settlement on the order of 3-inches is estimated within the first year of construction. An additional ½-inch of long-term settlement (consolidation and secondary) is anticipated over the remaining 50-year service life.

7.4 Frost Protection

Foundations placed on the fill or native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Lisbon has a design freezing index (DFI) of approximately 1400 F-degree days. A water content of 25% was used for coarse-grained soils. These components correlate to a frost depth of 5.1 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 5.1 feet for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.5 Scour and Riprap

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

Where required, slopes shall be armored with a 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot 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 below the streambed elevation. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

7.6 Seismic Design Considerations

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

8.0 CONSTRUCTION CONSIDERATIONS

The soil envelope and backfill for the box culvert 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. To minimize future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

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

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

- Construction phase dewatering is recommended to limit disturbance and rutting of the loose sands (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.
- Hand-deploy the stabilization/reinforcement geotextile on the prepared soil subgrade prior to installing the crushed stone mat.

The crushed stone mat can be topped 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 Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. All wood, organics and timber grillage encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow – Material for Underwater Backfill.

Earthwork and excavations may result in the exposure of silt or other soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with compacted Granular Borrow – Material for Underwater Backfill.

Soils may become saturated and water seepage may be encountered during construction and in excavations. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater and surface water infiltration using temporary ditches, sump pumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

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.

9.0 CLOSURE

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

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

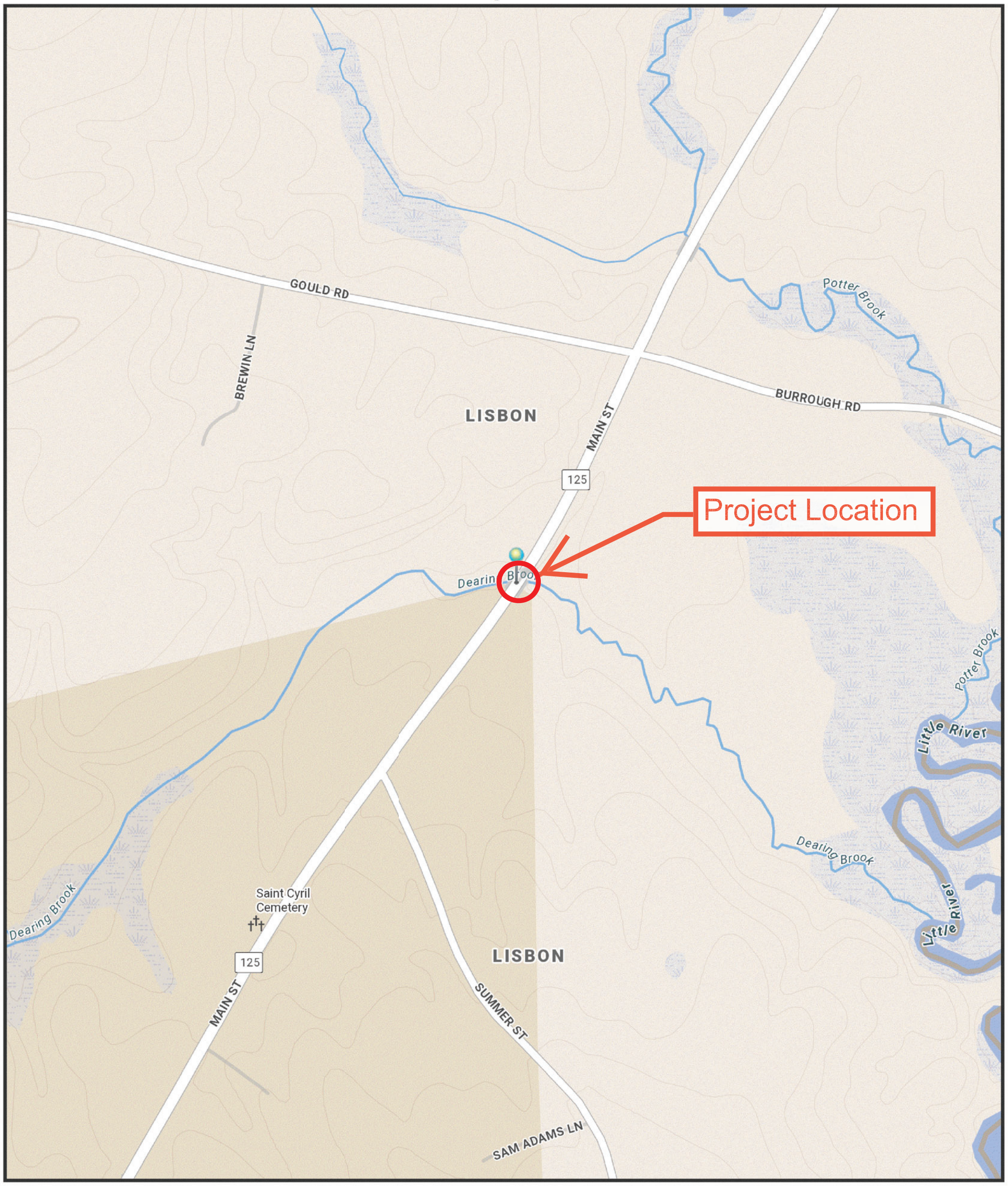
Sheets

Date:12/9/2020

Division: GEOTECH

Username: Terry.White

LISBON, MAINE



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0.1 Miles

1 inch = 0.11 miles

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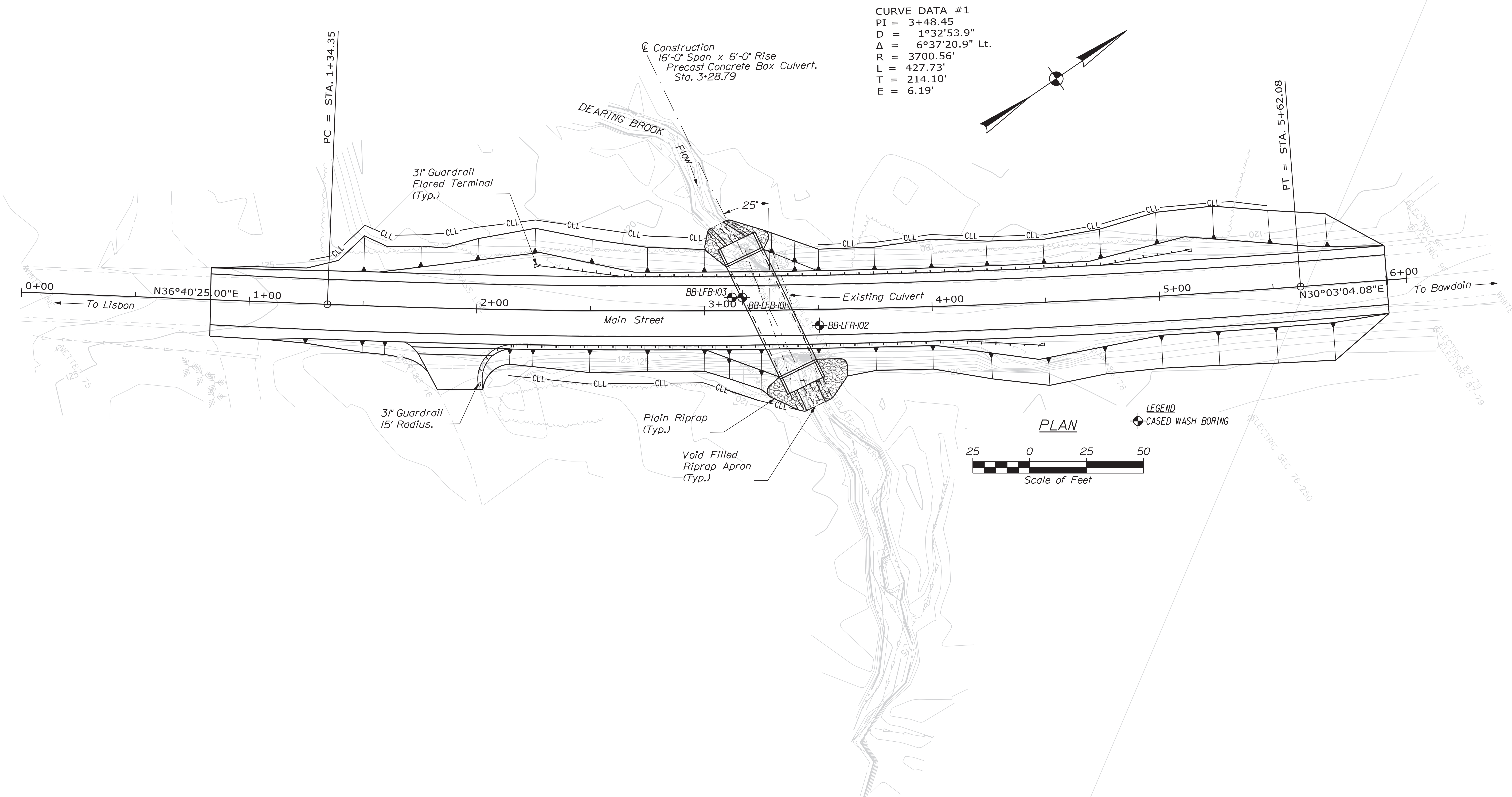
SHEET NUMBER 1 OF 4	FRAZIER BRIDGE DEERING BROOK LISBON ANDROSCOGGIN CTY.	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		2311800
		WIN
	LOCATION MAP	BRIDGE NO. 3954 23118.00 BRIDGE PLANS



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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2311800

BRIDGE NO. 3954 | WIN 23118.00 | BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	DEVAN EATON		
DESIGN-DETAILED	R. NADUS	M.CRITCHLOW	2/2020
CHECKED-REVIEWED	-----	-----	
DESIGN2-DETAILED2	J.MANAHAN	T.WHITE	FEB 2020
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRAZIER BRIDGE
DEERING BROOK
LISBON ANDROSCOGGIN COUNTY

BORING LOCATION PLAN

SHEET NUMBER

2

OF 4

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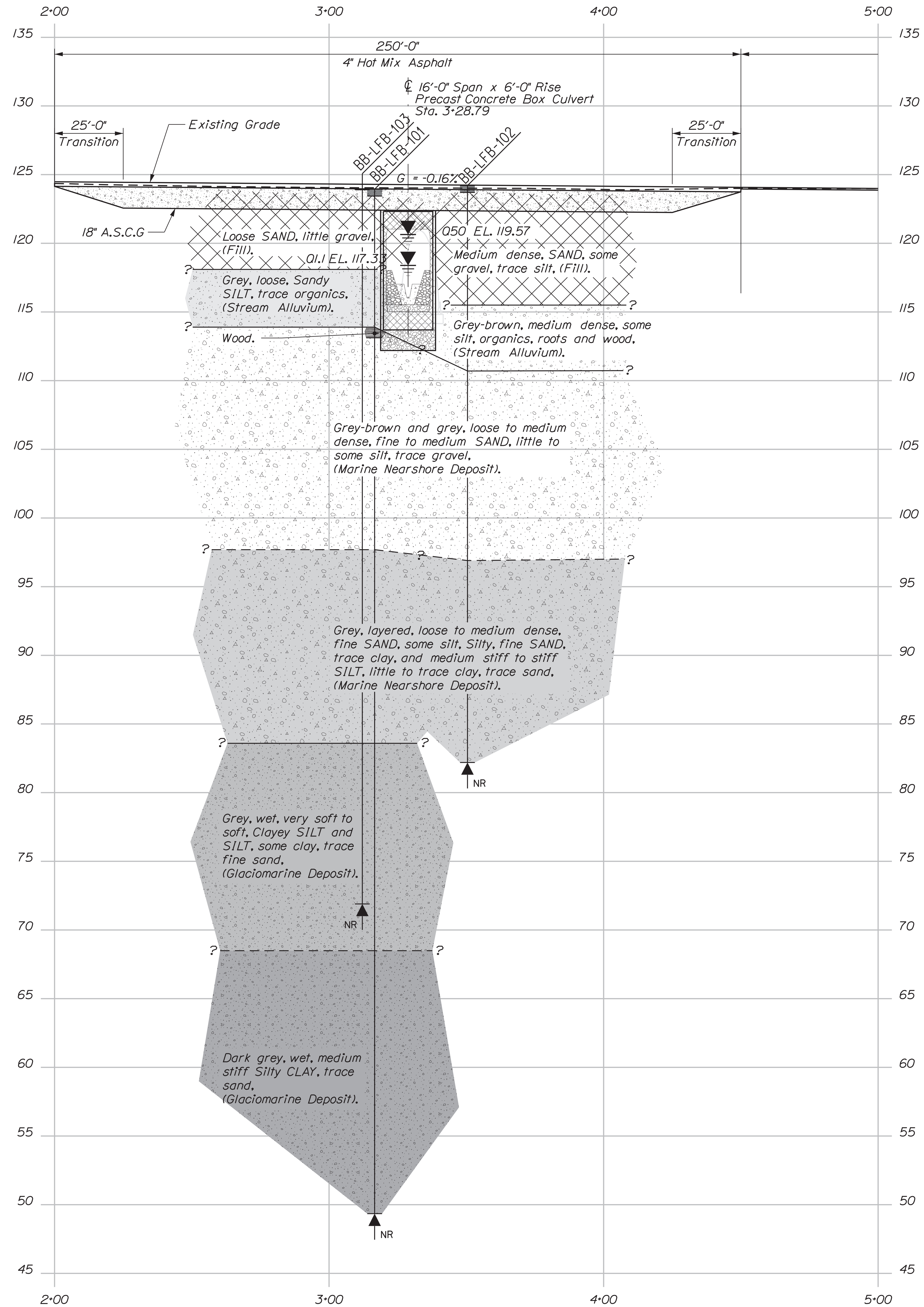

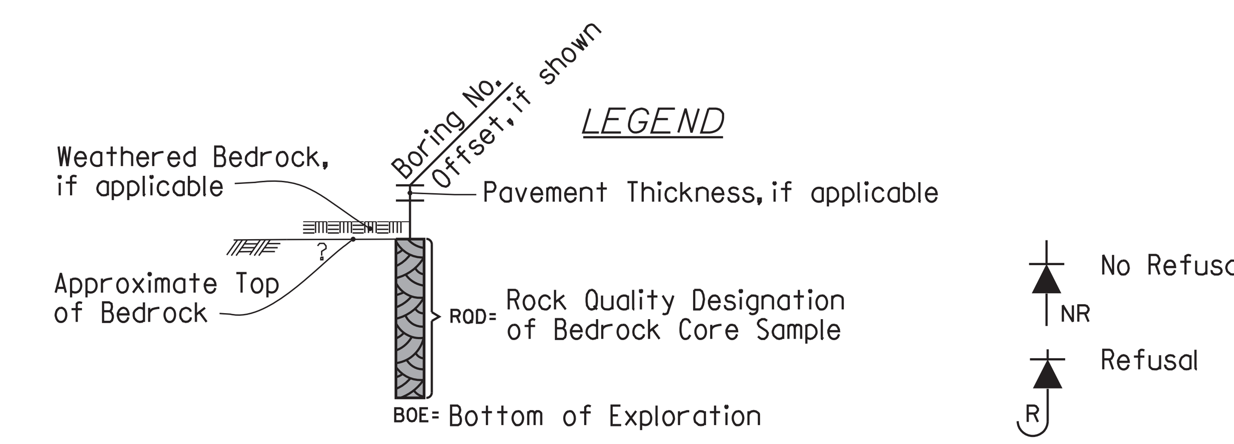

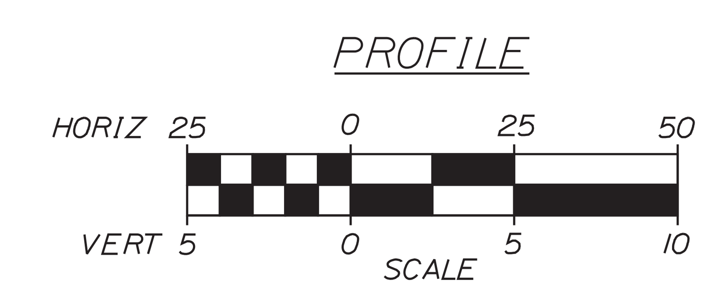

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and bedrock transitions may vary and are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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2311800

BRIDGE PLANS

WIN 23118.00

BRIDGE NO. 3954

	BY	DATE
PROJ. MANAGER	DEVAN EATON	
DESIGN-DETAILED	R. NAOUS	M.CRITCHLOW 2\2020
CHECKED-REVIEWED		
DESIGN2-DETAILED2	J.MANAHAN	T. WHITE JUN 2020
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

LISBON

FRAZIER BRIDGE
DEERING BROOK
ANDROSCOGGIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\MSTA\008_BORING LOGS1.dgn   Division: GEOTECH   Username: Terry.White   Date:12/31/2020

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-101
WIN: 23118.00

Driller:	MaineDOT	Elevation (ft.)	123.9	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Westtrack	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Manahan/Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/28/2020-1/29/2020	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	3+16.6, 5.8 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	4.5 ft bgs.
Hammer Efficiency Factor: 0.886		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	123.4	6" HMA. 0.5	
5	1D/A	24/18	5.00 - 7.00	4/4/3/3	7	10		117.9	1D (5.0-6.0 ft) Brown, moist, loose, fine to coarse SAND, little gravel, little silt, (Fill). 6.0 1D/A (6.0-7.0 ft) Grey, moist, loose, fine to coarse Sandy SILT, trace gravel, trace organics, (Stream Alluvium).	G#296596 A-4, CL WC=31.0%
10	2D	24/24	10.00 - 12.00	3/4/1/9	5	7	4	113.9	10.0 Brown grey, wet, loose, fine to medium SAND, some silt, (Marine Nearshore Deposit). Wood from 10.2-10.5 ft bgs.	
							22			
							24			
							33			
							37			
15	3D	24/12	15.00 - 17.00	4/4/4/7	8	12	5		Grey brown, wet, medium dense, fine to medium SAND, little silt, trace clay (Marine Nearshore Deposit).	G#296598 A-2-4, SC-SM WC=28.3%
							9			
							23			
							41			
							62			
20	4D	24/17	20.00 - 22.00	1/4/6/7	10	15	13		Grey brown, wet, medium dense, fine SAND, little silt (Marine Nearshore Deposit).	G#300265 A-2-4, SM WC=27.2%
							19			
							34			
							67			
							85			
25	5D	24/19	25.00 - 27.00	3/5/7/5	12	18	42	98.4	25.5 (5D) Grey, wet, medium dense, fine SAND, some silt, (Marine Nearshore Deposits).	
							50			
							73			
							61			
							71			
30	6D	24/14	30.00 - 32.00	3/4/3/3	7	10	57		Grey, wet, loose, Silty fine SAND, trace clay, (Marine Nearshore Deposits).	
							51			
							49			
							47			
							47			
35	7D	24/13	35.00 - 37.00	2/3/2/1	5	7	OPEN HOLE		Grey, wet, medium stiff, SILT, trace clay, trace fine sand, (Marine Nearshore Deposits). Hydraulic Pushed Casing from 35.0-50.0 ft bgs.	G#296599 A-4, CL WC=26.9% Non-Plastic
40	8D/A	24/24	40.00 - 42.00	WOH/WOH/2/WOH	2	3		83.9	40.0 8D (40.0-41.0 ft) Grey, wet, soft, Clayey SILT, trace fine sand. 8D/A (41.0-42.0 ft) Grey, wet, soft, SILT, some clay, trace fine sand, (Glaciomarine Clay).	
45	9D	24/22	45.00 - 47.00	2/1/WOH/WOH	1	1			Grey, wet, very soft, SILT, some clay, trace fine sand, (Glaciomarine Clay).	G#296600 A-4, CL WC=26.6% Non-Plastic
50	10D	24/24	50.00 - 52.00	WOR/WOR/WOR/WOR	---		HYD PUSH		Dark grey, wet, very soft, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	G#300261 A-4, CL WC=36.0% LL=32 PL=23 PI=9
55	11D V1	24/15	55.00 - 57.00 55.63 - 56.00	WOR/WOR/WOR/WOR Su=893/112 psf	---				Dark grey, wet, medium stiff, Silty CLAY, (Glaciomarine Deposit). 55x110 mm van raw torque readings: V1: 20.0/2.5 ft-lbs V2: 14.0/2.0 ft-lbs	
	V2		56.63 - 57.00	Su=625/89 psf						
60	12D V3	24/24	60.00 - 62.00 60.63 - 61.00	WOR/WOR/WOR/WOR Su=937/179 psf	---				Dark grey, wet, medium stiff, Silty CLAY, trace sand (Glaciomarine Deposit). 55x110 mm van raw torque readings: V3: 21.0/4.0 ft-lbs V4: 22.0/4.0 ft-lbs	G#300262 A-6, CL WC=33.3% LL=34 PL=22 PI=12
	V4		61.63 - 62.00	Su=982/179 psf						
65	V5		65.00 - 65.37	Su=915/179 psf					55x110 mm van raw torque readings: V5: 210.5/4.0 ft-lbs	
	V6		66.00 - 66.37	Su=1027/223 psf					V6: 23.0/5.0 ft-lbs	
70									Stiff Layer at 72.5 ft bgs.	
75								49.3	74.6 Bottom of Exploration at 74.6 feet below ground surface. Casing REFUSAL, bounced on hammer strike.	

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-LFB-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-102
WIN: 23118.00

Driller:	MaineDOT	Elevation (ft.)	124.2	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Westtrack	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Manahan/Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/29/2020; 09:00-?	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	3+50.4, 7.0 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	2.8 ft bgs.
Hammer Efficiency Factor: 0.886		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	123.7	6" HMA. 0.5	
5	1D	24/11	5.00 - 7.00	3/4/5/5	9	13			Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
								115.7	8.5	
10	2D	24/12	10.00 - 12.00	1/5/6/5	11	16	5		Grey brown, damp, medium dense, fine to medium SAND, some silt, organics and roots/wood, (Stream Alluvium).	
							13			
							30	111.2	13.0	
							62			
							61			
15	3D	24/14	15.00 - 17.00	9/12/8/7	20	30	24		Brown, wet, medium dense, fine to medium SAND, little silt, trace gravel (Nearshore Marine Deposit).	G#300263 A-2-4, SP-SM WC=24.3%
							23			
							30			
							51			
							55			
20	4D	24/13	20.00 - 22.00	4/5/8/9	13	19	26		Brown, wet, medium dense, fine SAND, trace silt, (Nearshore Marine Deposit).	
							28			
							44			
							56			
							64			
25	5D	24/17	25.00 - 27.00	4/5/7/9	12	18	41		Similar to above.	
							49	97.2	27.0	
							78			
							76			
							80			
30	6D	24/9	30.00 - 32.00	5/5/4/2	9	13	OPEN HOLE		Grey, wet, medium dense, Silty, fine SAND, (Nearshore Marine Deposits).	
35	7D	24/24	35.00 - 37.00	WOH/4/6/4	10	15			Grey, wet, stiff, SILT, trace fine sand, (Nearshore Marine Deposits).	
40	8D	24/18	40.00 - 42.00	3/4/1/WOH	5	7		82.2	Grey, wet, medium stiff, SILT, little clay, trace fine to coarse sand (Nearshore Marine Deposits). 42.0 Bottom of Exploration at 42.0 feet below ground surface. NO REFUSAL	G#300264 A-4, CL WC=22.2% Non-Plastic
45										
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.

Page 1 of 1

Boring No.: BB-LFB-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-103
WIN: 23118.00

Driller:	MaineDOT	Elevation (ft.)	123.9	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Westtrack	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	J. Manahan	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	2/11/2020-2/12/2020	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	3+12.1, 5.8 ft Lt.	Casing ID/OD:	HW-4"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.886		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			
10							SPUN		Pushed HW Casing to 10.0 ft bgs. Spun Casing ahead to 30.0 ft bgs.	
									Brown washwater to 28.5 ft bgs, grey washwater after 28.5 ft bgs.	
30							OPEN HOLE			
									Clay in washwater at 32.0 ft bgs.	
35	1D/A	24/19	35.00 - 37.00	2/2/WOR/WOR	2	3	HW	87.9	1D (35.0-36.0 ft bgs) Grey, wet, very soft, fine to medium SAND, some silt. Cased Boring from 35.0-45.0 ft bgs. 36.0 1D/A (36.0-37.0 ft bgs) Grey, wet, very soft, Clayey SILT, trace fine sand.	
45	1U	24/24	45.00 - 47.00	Piston Sampler					Grey SILT, little clay, trace fine sand.	G,C#300266 A-4, CL WC=31.2% Non-Plastic
50	2U	24/24	50.00 - 52.00	Piston Sampler				71.9	Grey SILT, some clay, trace fine sand. 52.0 Bottom of Exploration at 52.0 feet below ground surface. NO REFUSAL	G,C#300267 A-4, CL-ML WC=34.7% LL=26 PL=21 PI=5
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-LFB-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2311800

BRIDGE NO. 3954

WIN 23118.00

BRIDGE PLANS

PROJ. MANAGER	DEVAN EATON		
	BY	DATE	
DESIGN-DETAILED	R. NAOUS	M.CRITCHLOW	2\2020
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	J.MANAHAN	T.WHITE	DEC 2020
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

FRAZIER BRIDGE
DEERING BROOK
LISBON ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

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

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

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)

Rock Quality Based on RQD

Rock 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 quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/28/2020-1/29/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+16.6, 5.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	4.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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0							SSA	123.4		6" HMA. 0.5	
5	1D/A	24/18	5.00 - 7.00	4/4/3/3	7	10		117.9		1D (5.0-6.0 ft) Brown, moist, loose, fine to coarse SAND, little gravel, little silt, (Fill). 6.0 1D/A (6.0-7.0 ft) Grey, moist, loose, fine to coarse Sandy SILT, trace gravel, trace organics, (Stream Alluvium).	G#296596 A-4, CL WC=31.0%
10	2D	24/24	10.00 - 12.00	3/4/1/9	5	7	4	113.9		10.0 Brown grey, wet, loose, fine to medium SAND, some silt, (Marine Nearshore Deposit). Wood from 10.2-10.5 ft bgs.	
							22				
							24				
							33				
							37				
15	3D	24/12	15.00 - 17.00	4/4/4/7	8	12	5			Grey brown, wet, medium dense, fine to medium SAND, little silt, trace clay (Marine Nearshore Deposit).	G#296598 A-2-4, SC-SM WC=28.3%
							9				
							23				
							41				
							62				
20	4D	24/17	20.00 - 22.00	1/4/6/7	10	15	13			Grey brown, wet, medium dense, fine SAND, little silt (Marine Nearshore Deposit).	G#300265 A-2-4, SM WC=27.2%
							19				
							34				
							67				
25							85				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-101
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/28/2020-1/29/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+16.6, 5.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	4.5 ft bgs.
Hammer Efficiency Factor:	0.886	**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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/19	25.00 - 27.00	3/5/7/5	12	18	42	98.4		25.5 (5D) Grey, wet, medium dense, fine SAND, some silt, (Marine Nearshore Deposits).	
							50				
							73				
							61				
							71				
30	6D	24/14	30.00 - 32.00	3/4/3/3	7	10	57			Grey, wet, loose, Silty fine SAND, trace clay, (Marine Nearshore Deposits).	
							51				
							49				
							47				
							47				
35	7D	24/13	35.00 - 37.00	2/3/2/1	5	7	OPEN HOLE			Grey, wet, medium stiff, SILT, trace clay, trace fine sand, (Marine Nearshore Deposits). Hydraulic Pushed Casing from 35.0-50.0 ft bgs.	G#296599 A-4, CL WC=26.9% Non-Plastic
40	8D/A	24/24	40.00 - 42.00	WOH/WOH/2/WOH	2	3		83.9		40.0 8D (40.0-41.0 ft) Grey, wet, soft, Clayey SILT, trace fine sand. 8D/A (41.0-42.0 ft) Grey, wet, soft, SILT, some clay, trace fine sand, (Glaciomarine Clay).	
45	9D	24/22	45.00 - 47.00	2/1/WOH/WOH	1	1				Grey, wet, very soft, SILT, some clay, trace fine sand, (Glaciomarine Clay).	G#296600 A-4, CL WC=26.6% Non-Plastic
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-101
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/28/2020-1/29/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+16.6, 5.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	4.5 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	10D	24/24	50.00 - 52.00	WOR/WOR/WOR/ WOR	---		HYD PUSH			Dark grey, wet, very soft, Clayey SILT, trace fine sand, (Glaciomarine Deposit).	G#300261 A-4, CL WC=36.0% LL=32 PL=23 PI=9
55	11D	24/15	55.00 - 57.00	WOR/WOR/WOR/ WOR	---					Dark grey, wet, medium stiff, Silty CLAY, (Glaciomarine Deposit).	
	V1		55.63 - 56.00	Su=893/112 psf						55x110 mm van raw torque readings: V1: 20.0/2.5 ft-lbs	
	V2		56.63 - 57.00	Su=625/89 psf						V2: 14.0/2.0 ft-lbs	
60	12D	24/24	60.00 - 62.00	WOR/WOR/WOR/ WOR	---					Dark grey, wet, medium stiff, Silty CLAY, trace sand (Glaciomarine Deposit).	G#300262 A-6, CL WC=33.3% LL=34 PL=22 PI=12
	V3		60.63 - 61.00	Su=937/179 psf						55x110 mm van raw torque readings: V3: 21.0/4.0 ft-lbs	
	V4		61.63 - 62.00	Su=982/179 psf						V4: 22.0/4.0 ft-lbs	
65	V5		65.00 - 65.37	Su=915/179 psf						55x110 mm van raw torque readings: V5: 210.5/4.0 ft-lbs	
	V6		66.00 - 66.37	Su=1027/223 psf						V6: 23.0/5.0 ft-lbs	
70											
										Stiff Layer at 72.5 ft bgs.	
75								49.3		74.6	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-101
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/28/2020-1/29/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+16.6, 5.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	4.5 ft bgs.

Hammer Efficiency Factor: 0.886 **Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Definitions:
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MV = Unsuccessful Field Vane Shear Test Attempt
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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
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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.
75										**Bottom of Exploration at 74.6 feet below ground surface.** Casing REFUSAL, bounced on hammer strike.	
80											
85											
90											
95											
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-102

WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	124.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/29/2020; 09:00-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+50.4, 7.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	2.8 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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D = Split Spoon Sample
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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	123.7		6" HMA. 0.5	
5	1D	24/11	5.00 - 7.00	3/4/5/5	9	13				Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
								115.7		8.5	
10	2D	24/12	10.00 - 12.00	1/5/6/5	11	16	5			Grey brown, damp, medium dense, fine to medium SAND, some silt, organics and roots/wood, (Stream Alluvium).	
							13				
							30				
							62	111.2		13.0	
							61				
15	3D	24/14	15.00 - 17.00	9/12/8/7	20	30	24			Brown, wet, medium dense, fine to medium SAND, little silt, trace gravel (Nearshore Marine Deposit).	G#300263 A-2-4, SP-SM WC=24.3%
							23				
							30				
							51				
							55				
20	4D	24/13	20.00 - 22.00	4/5/8/9	13	19	26			Brown, wet, medium dense, fine SAND, trace silt, (Nearshore Marine Deposit).	
							28				
							44				
							56				
25							64				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-102
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	124.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Manahan/Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/29/2020; 09:00-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+50.4, 7.0 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	2.8 ft bgs.
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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)
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/17	25.00 - 27.00	4/5/7/9	12	18	41			Similar to above.	
							49				
							78	97.2		27.0	
							76				
							80				
30	6D	24/9	30.00 - 32.00	5/5/4/2	9	13	OPEN HOLE			Grey, wet, medium dense, Silty, fine SAND, (Nearshore Marine Deposits).	
35	7D	24/24	35.00 - 37.00	WOH/4/6/4	10	15				Grey, wet, stiff, SILT, trace fine sand, (Nearshore Marine Deposits).	
40	8D	24/18	40.00 - 42.00	3/4/1/WOH	5	7				Grey, wet, medium stiff, SILT, little clay, trace fine to coarse sand (Nearshore Marine Deposits).	G#300264 A-4, CL WC=22.2% Non-Plastic
								82.2		42.0 **Bottom of Exploration at 42.0 feet below ground surface.** NO REFUSAL	
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-103
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/11/2020-2/12/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+12.1, 5.8 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
LL = Liquid Limit
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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				
5											
10							SPUN			Pushed HW Casing to 10.0 ft bgs. Spun Casing ahead to 30.0 ft bgs.	
15											
20											
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-103
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/11/2020-2/12/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+12.1, 5.8 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
										Brown washwater to 28.5 ft bgs, grey washwater after 28.5 ft bgs.	
30							OPEN HOLE				
										Clay in washwater at 32.0 ft bgs.	
35	1D/A	24/19	35.00 - 37.00	2/2/WOR/WOR	2	3	HW	87.9		1D (35.0-36.0 ft bgs) Grey, wet, very soft, fine to medium SAND, some silt. Cased Boring from 35.0-45.0 ft bgs. 36.0 1D/A (36.0-37.0 ft bgs) Grey, wet, very soft, Clayey SILT, trace fine sand.	
40											
45	1U	24/24	45.00 - 47.00	Piston Sampler						Grey SILT, little clay, trace fine sand.	G,C#300266 A-4, CL WC=31.2% Non-Plastic
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Frazier Bridge #3954 carries Route 125 over Deering Brook
Location: Lisbon, Maine

Boring No.: BB-LFB-103
WIN: 23118.00

Driller:	MaineDOT	**Elevation (ft.)**	123.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Westtrack	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	J. Manahan	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/11/2020-2/12/2020	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+12.1, 5.8 ft Lt.	**Casing ID/OD:**	HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.886	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	2U	24/24	50.00 - 52.00	Piston Sampler						Grey SILT, some clay, trace fine sand.	G,C#300267 A-4, CL-ML WC=34.7% LL=26 PL=21 PI=5
								71.9		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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lisbon Work Number: 23118.00

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-LFB-101, 1D/A	3+16.6	5.8 Lt.	6.0-7.0	296596	1	31.0			CL	A-4	IV
BB-LFB-101, 3D	3+16.6	5.8 Lt.	15.0-17.0	296598	1	28.3			SC-SM	A-2-4	II
BB-LFB-101, 4D	3+16.6	5.8 Lt.	20.0-22.0	300265	1	27.2			SM	A-2-4	II
BB-LFB-101, 7D	3+16.6	5.8 Lt.	35.0-37.0	296599	1	26.9	-N	P-	CL	A-4	IV
BB-LFB-101, 9D	3+16.6	5.8 Lt.	45.0-47.0	296600	2	26.6	-N	P-	CL	A-4	IV
BB-LFB-101, 10D	3+16.6	5.8 Lt.	50.0-52.0	300261	2	36.0	32	9	CL	A-4	IV
BB-LFB-101, 12D	3+16.6	5.8 Lt.	60.0-62.0	300262	2	33.3	34	12	CL	A-6	III
BB-LFB-102, 3D	3+50.4	7.0 Rt.	15.0-17.0	300263	3	24.3			SP-SM	A-2-4	0
BB-LFB-102, 8D	3+50.4	7.0 Rt.	40.0-42.0	300264	3	22.2	-N	P-	CL	A-4	IV
BB-LFB-103, 1U	3+12.1	5.8 Lt.	45.0-47.0	300266	4	31.2	-N	P-	CL	A-4	IV
BB-LFB-103, 2U	3+12.1	5.8 Lt.	50.0-52.0	300267	4	34.7	26	5	CL-ML	A-4	IV

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.

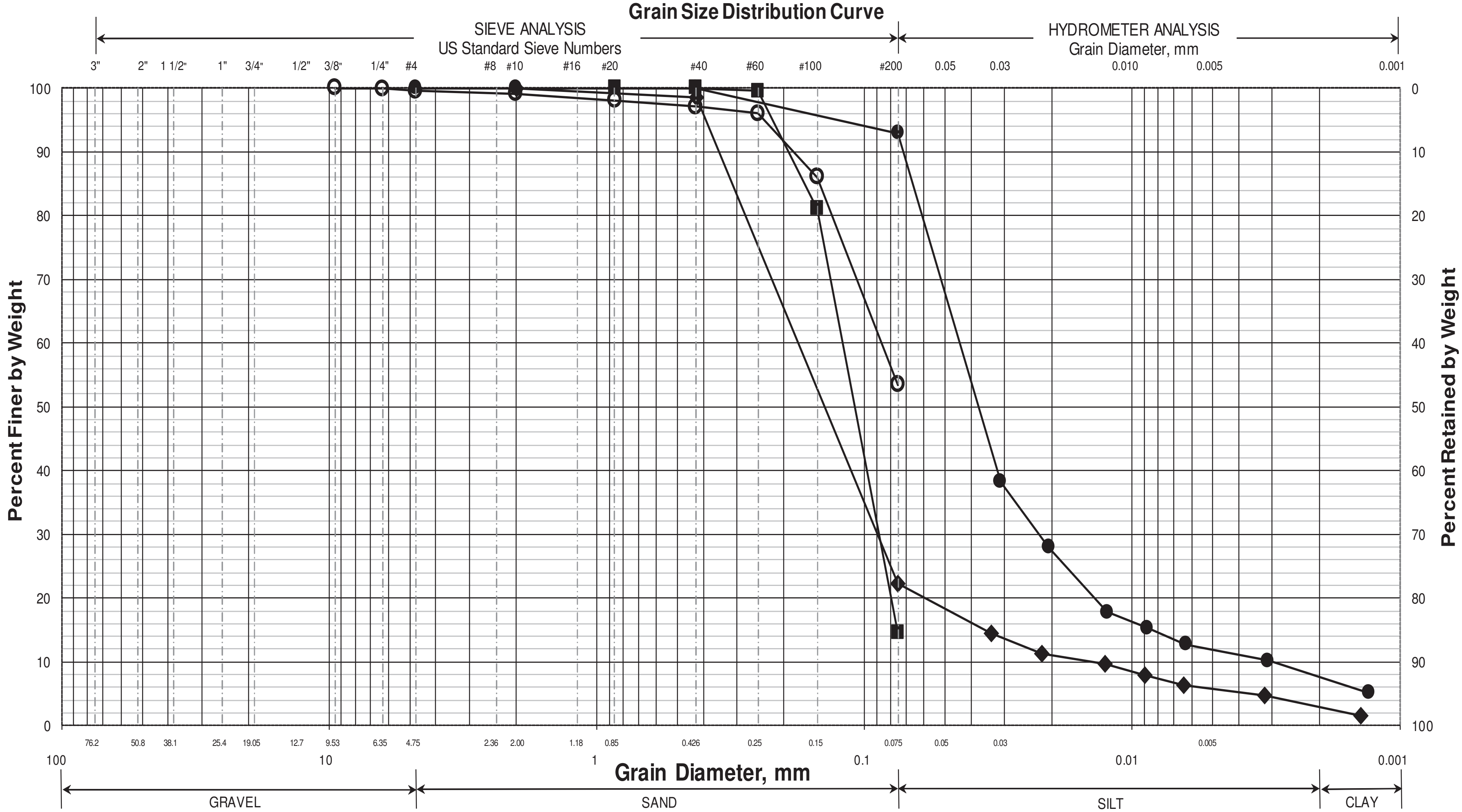
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



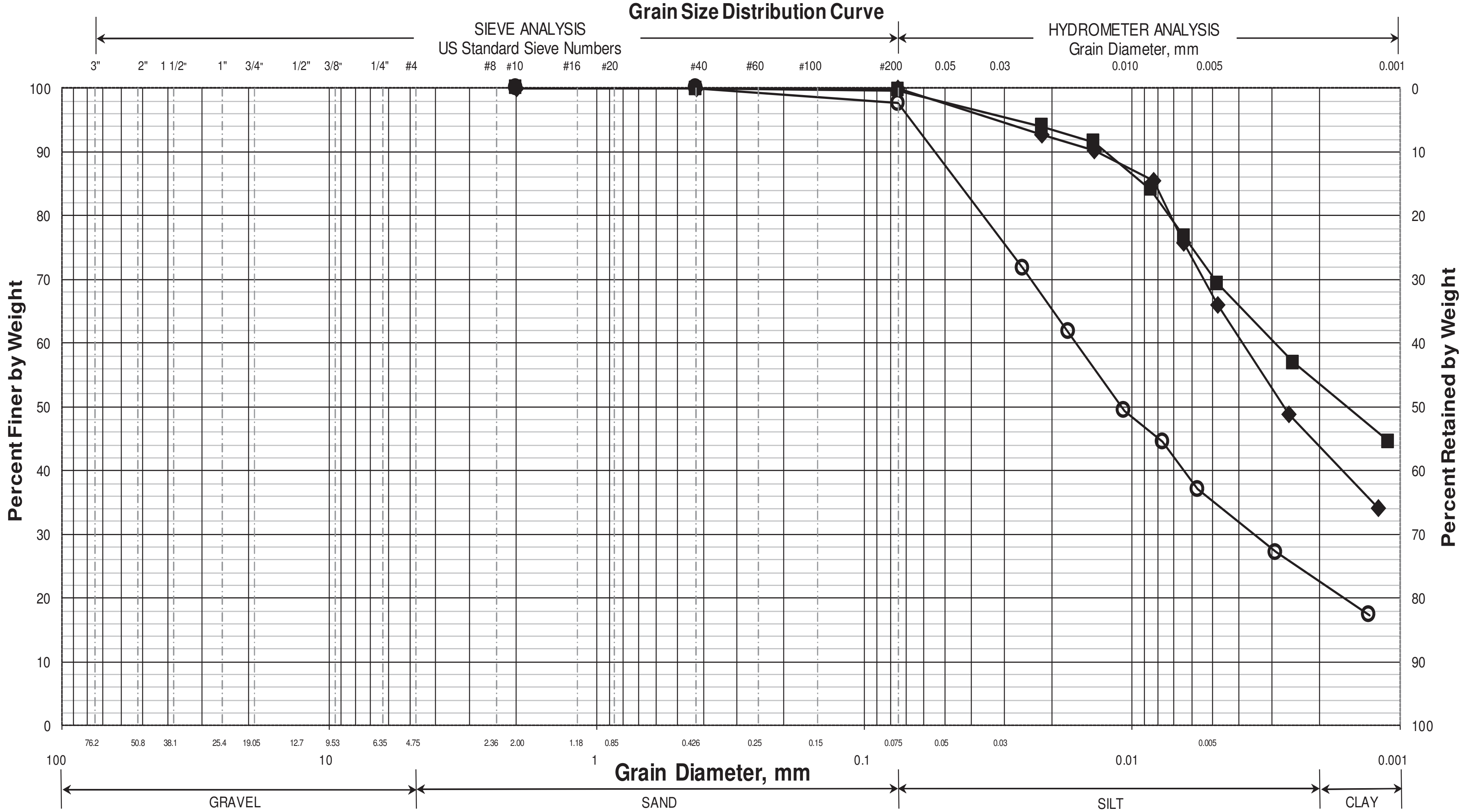
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LFB-101/1D-A	3+16.6	5.8 LT	6.0-7.0	Sandy SILT, trace gravel.	31			
◆	BB-LFB-101/3D	3+16.6	5.8 LT	15.0-17.0	SAND, little silt, trace clay.	28.3			
■	BB-LFB-101/4D	3+16.6	5.8 LT	20.0-22.0	SAND, little silt.	27.2			
●	BB-LFB-101/7D	3+16.6	5.8 LT	35.0-37.0	SILT, trace clay, trace sand.	26.9			NP
▲									
×									

WIN	
023118.00	
Town	
Lisbon	
Reported by/Date	
WHITE, TERRY A	2/20/2020

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



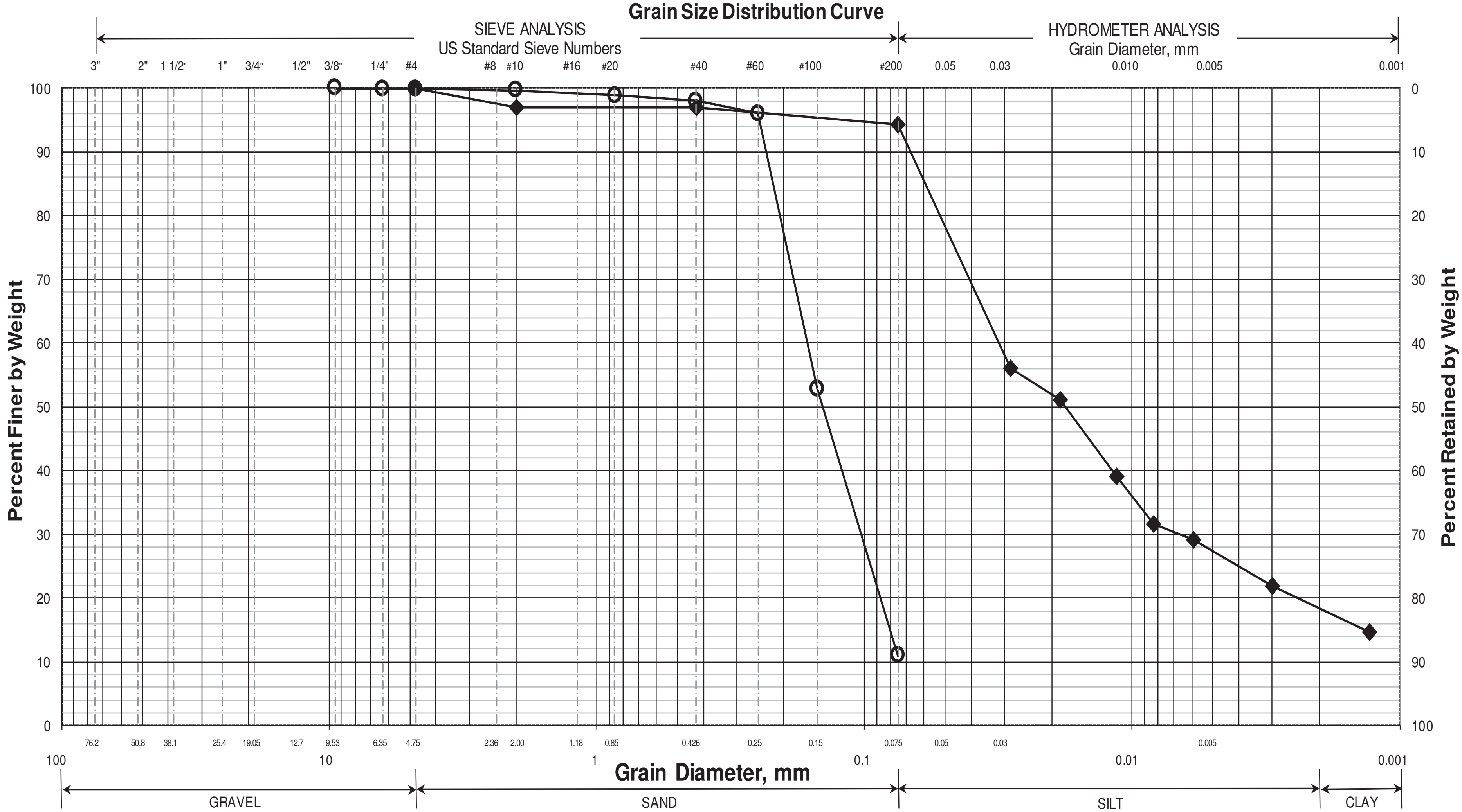
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LFB-101/9D	3+16.6	5.8 LT	45.0-47.0	SILT, some clay, trace sand.	26.6			NP
◆	BB-LFB-101/10D	3+16.6	5.8 LT	50.0-52.0	Clayay SILT, trace sand.	36	32	23	9
■	BB-LFB-101/12D	3+16.6	5.8 LT	60.0-62.0	Silty CLAY, trace sand.	33.3	34	22	12
●									
▲									
×									

WIN	
023118.00	
Town	
Lisbon	
Reported by/Date	
WHITE, TERRY A	2/20/2020

SHEET 2

Maine Department of Transportation
Grain Size Distribution Curve



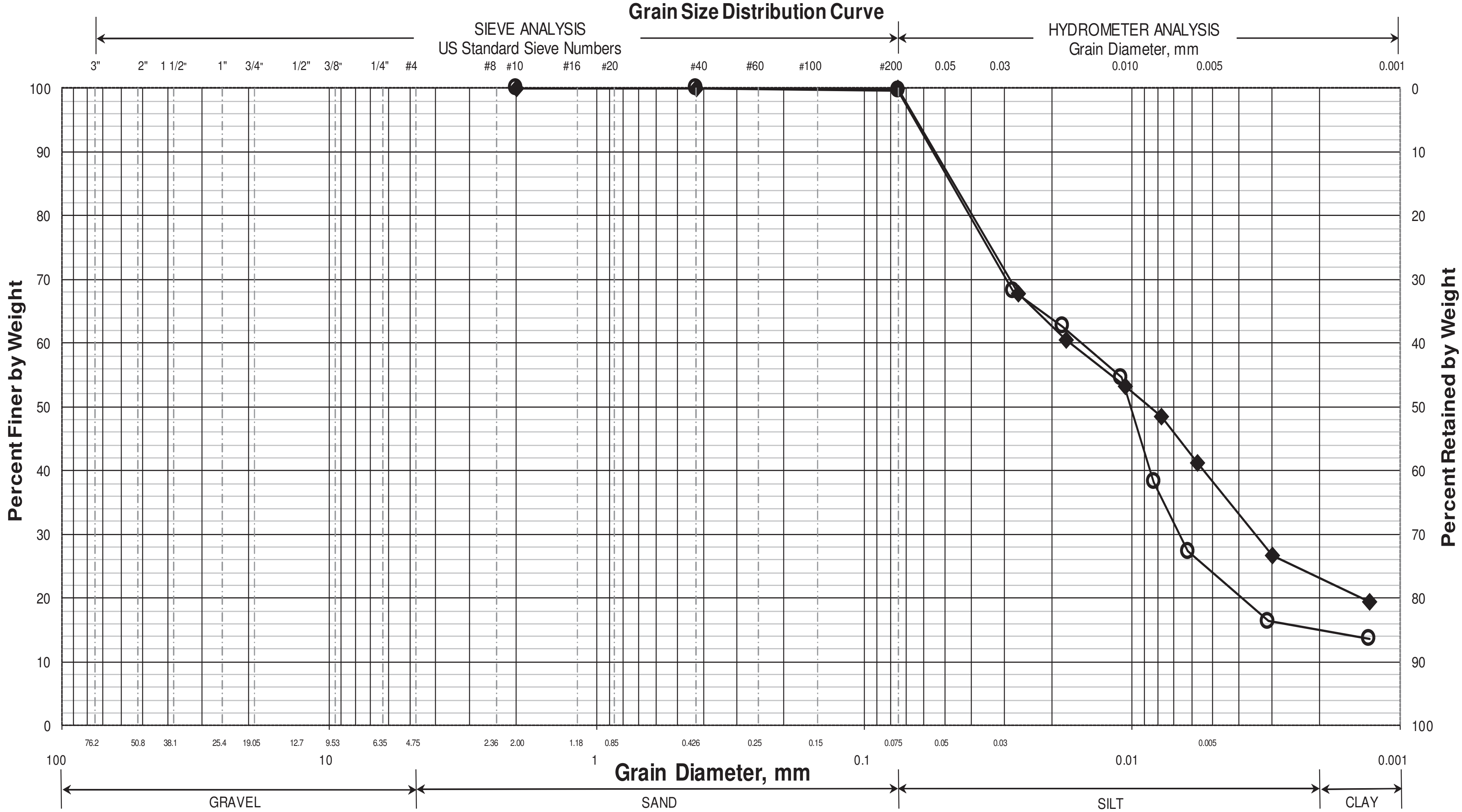
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-LFB-102/3D	3+50.4	7.0 RT	15.0-17.0	SAND, little silt, trace gravel.	24.3			
◆	BB-LFB-102/8D	3+50.4	7.0 RT	40.0-42.0	SILT, little clay, trace sand.	22.2			NP
■									
●									
▲									
×									

WIN
023118.00
Town
Lisbon
Reported by/Date
WHITE, TERRY A 2/20/2020

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-LFB-103/1U	3+12.1	5.8 LT	45.0-47.0	SILT, little clay, trace sand.	31.2			NP
◆	BB-LFB-103/2U	3+12.1	5.8 LT	50.0-52.0	SILT, some clay, trace sand.	34.7	26	21	5
■									
●									
▲									
×									

WIN	
023118.00	
Town	
Lisbon	
Reported by/Date	
WHITE, TERRY A	11/9/2020

SHEET 4

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296599	BB-LFB-101/7D	GEOTECHNICAL (DISTURBED)	1/28/2020	2/3/2020

Sample Type: **GEOTECHNICAL** Location: Station: **3+16.6** Offset, ft: **5.8 LT** Dbfg, ft: **35.0-37.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	92.9
[0.0310 mm]	38.2
[0.0205 mm]	28.0
[0.0124 mm]	17.8
[0.0088 mm]	15.3
[0.0063 mm]	12.7
[0.0031 mm]	10.2
[0.0013 mm]	5.1

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.73
Loss on Ignition, % (T 267)	
Water Content (T 265), %	26.9

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/13/2020**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296600	**BB-LFB-101/9D**	**GEOTECHNICAL (DISTURBED)**	**1/28/2020**	**2/3/2020**

Sample Type: **GEOTECHNICAL** Location: Station: **3+16.6** Offset, ft: **5.8 LT** Dbfg, ft: **45.0-47.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	**100.0**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**97.6**
[0.0256 mm]	**71.7**
[0.0173 mm]	**61.8**
[0.0107 mm]	**49.4**
[0.0077 mm]	**44.5**
[0.0057 mm]	**37.1**
[0.0029 mm]	**27.2**
[0.0013 mm]	**17.3**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	**NP**
Specific Gravity, Corrected to 20°C (T 100)	**2.73**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**26.6**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
300261	BB-LFB-101/10D	GEOTECHNICAL (DISTURBED)	1/28/2020	2/4/2020

Sample Type: **GEOTECHNICAL** Location: Station: **3+16.6** Offset, ft: **5.8** **LT** Dbfg, ft: **50.0-52.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.9
[0.0217 mm]	92.8
[0.0139 mm]	90.3
[0.0083 mm]	85.4
[0.0064 mm]	75.7
[0.0048 mm]	65.9
[0.0026 mm]	48.8
[0.0012 mm]	34.2

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	32
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	9
Specific Gravity, Corrected to 20°C (T 100)	2.75
Loss on Ignition, % (T 267)	
Water Content (T 265), %	36.0

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

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Reported by: **GREGORY LIDSTONE** Date Reported: **2/13/2020**

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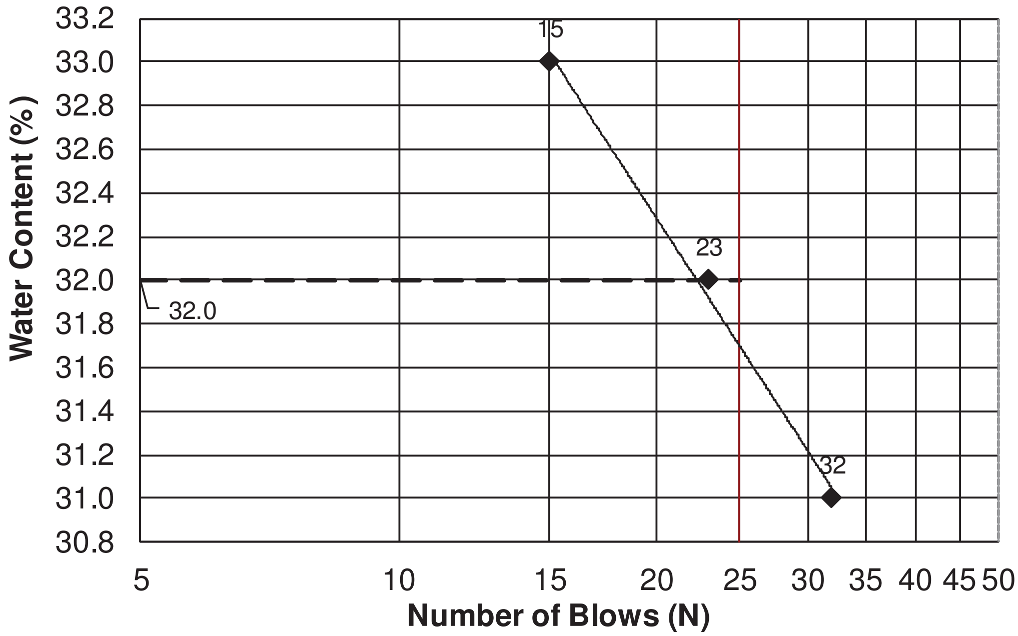
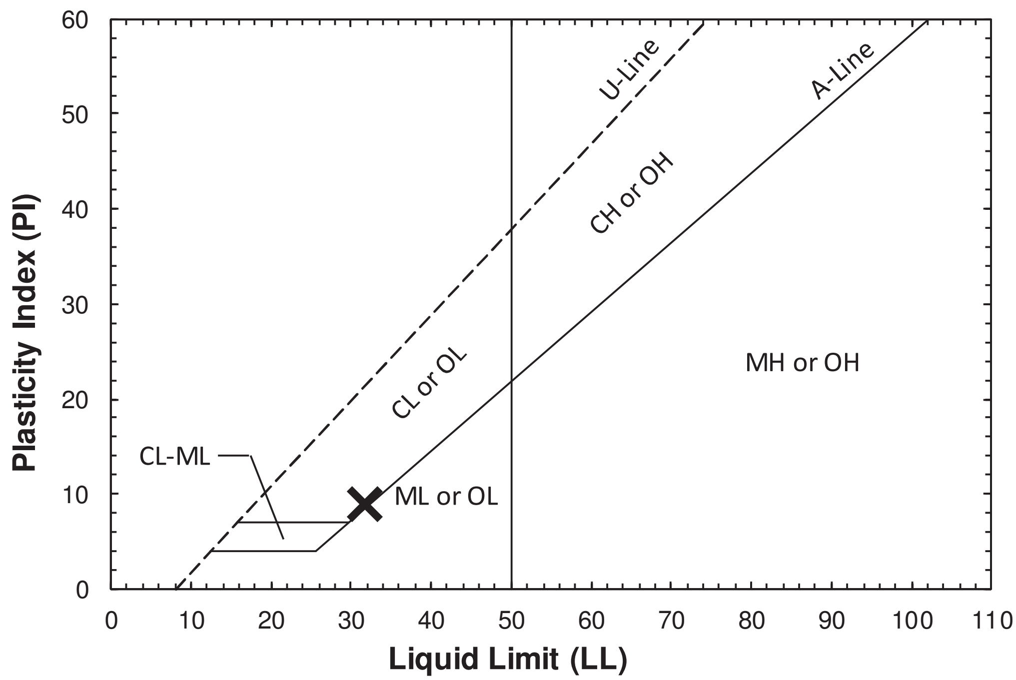
TOWN	Lisbon	Reference No.	300261
WIN	023118.00	Water Content, %	36
Sampled	1/28/2020	Liquid Limit @ 25 blows (T 89), %	32
Boring No./Sample No.	BB-LFB-101/10D	Plastic Limit (T 90), %	23
Station	3+16.6	Plasticity Index (T 90), %	9
Depth	50.0-52.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
300262	BB-LFB-101/12D	GEOTECHNICAL (DISTURBED)	1/28/2020	2/4/2020

Sample Type: **GEOTECHNICAL** Location: Station: **3+16.6** Offset, ft: **5.8 LT** Dbfg, ft: **60.0-62.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0217 mm]	94.0
[0.0139 mm]	91.5
[0.0085 mm]	84.1
[0.0064 mm]	76.7
[0.0048 mm]	69.3
[0.0025 mm]	56.9
[0.0011 mm]	44.5

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	34
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	12
Specific Gravity, Corrected to 20°C (T 100)	2.74
Loss on Ignition, % (T 267)	
Water Content (T 265), %	33.3

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/13/2020**

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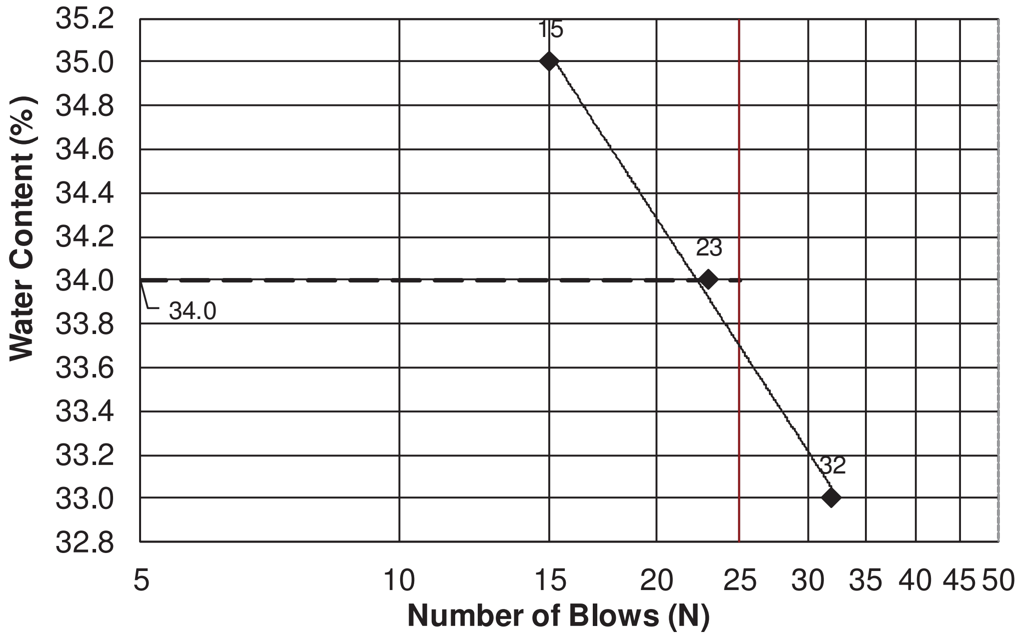
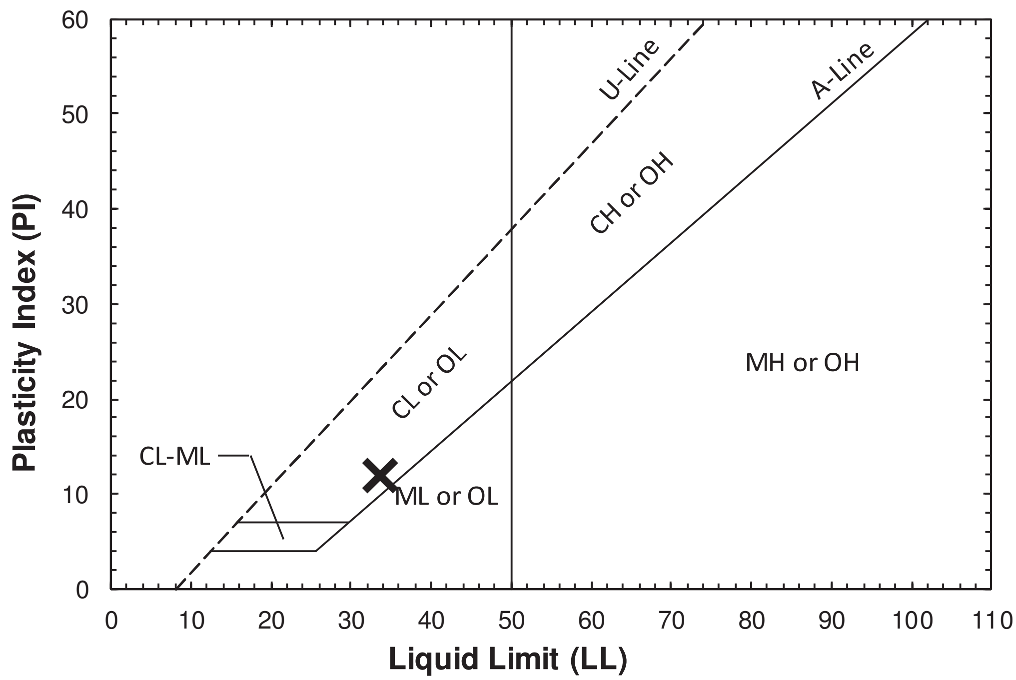
TOWN	Lisbon	Reference No.	300262
WIN	023118.00	Water Content, %	33.3
Sampled	1/28/2020	Liquid Limit @ 25 blows (T 89), %	34
Boring No./Sample No.	BB-LFB-101/12D	Plastic Limit (T 90), %	22
Station	3+16.6	Plasticity Index (T 90), %	12
Depth	60.0-62.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
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SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
300264	BB-LFB-102/8D	GEOTECHNICAL (DISTURBED)	1/29/2020	2/4/2020

Sample Type: **GEOTECHNICAL** Location: Station: **3+50.4** Offset, ft: **7.0 RT** Dbfg, ft: **40.0-42.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	96.9
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	96.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	94.3
[0.0284 mm]	56.0
[0.0186 mm]	51.1
[0.0114 mm]	39.0
[0.0083 mm]	31.7
[0.0059 mm]	29.2
[0.0030 mm]	21.9
[0.0013 mm]	14.6
[mm]	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.68
Loss on Ignition, % (T 267)	
Water Content (T 265), %	22.2

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **2/13/2020**

Paper Copy: Lab File; Project File; Geotech File

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MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
300266	BB-LFB-103/1U	GEOTECHNICAL (UNDISTURBED)	2/12/2020	2/13/2020

Sample Type: **GEOTECHNICAL** Location: Station: **3+12.1** Offset, ft: **5.8 LT** Dbfg, ft: **45.0-47.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0279 mm]	68.2
[0.0182 mm]	62.7
[0.0110 mm]	54.6
[0.0083 mm]	38.2
[0.0062 mm]	27.3
[0.0031 mm]	16.4
[0.0013 mm]	13.6

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	NP
Specific Gravity, Corrected to 20°C (T 100)	2.69
Loss on Ignition, % (T 267)	
Water Content (T 265), %	31.2

Consolidation (T 216)

Trimmings, Water Content, % **34.4**

	Initial	Final		Void Ratio	% Strain
Water Content, %	40.87	30.4	Pmin		
Dry Density, lbs/ft³	77.059	92.379	Pp		
Void Ratio	1.18	0.818	Pmax		
Saturation, %	93.22	100	Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-0.5	0.07	0	0.03	0	32.4	Alternating layers of light to dark grey clay.
0.63-1.0	0.21	0	0.19	0	30.7	Alternating layers of light to dark grey clay, silt line 11.5" to 12".
1.0-1.5	0.19	0	0.16	0	25.1	Alternating layers of light to dark grey clay, silt line at 15.25".
1.5-1.92	0.11	0	0.24	0	21.5	Alternating layers of light to dark grey clay, silt line at 20" and 20.5", fine sand line at 21.5", fine sand 22" to 23".

Comments:

.

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **5/15/2020**

Paper Copy: Lab File; Project File; Geotech File

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
300267	**BB-LFB-103/2U**	**GEOTECHNICAL (UNDISTURBED)**	**2/12/2020**	**2/13/2020**

Sample Type: **GEOTECHNICAL** Location: Station: **3+12.1** Offset, ft: **5.8** **LT** Dbfg, ft: **50.0-52.0**

WIN/Town **023118.00 - LISBON** Sampler: **BRUCE WILDER**

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]	**100.0**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**99.9**
[0.0265 mm]	**67.8**
[0.0176 mm]	**60.5**
[0.0106 mm]	**53.3**
[0.0078 mm]	**48.4**
[0.0057 mm]	**41.2**
[0.0030 mm]	**26.6**
[0.0013 mm]	**19.4**
[mm]	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**26**
Plastic Limit (T 90), %	**21**
Plasticity Index (T 90), %	**5**
Specific Gravity, Corrected to 20°C (T 100)	**2.70**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**34.7**

Consolidation (T 216)

Trimmings, Water Content, % **28.8**

	Initial	Final		Void Ratio	% Strain
Water Content, %	**34.99**	**23.09**	Pmin		
Dry Density, lbs/ft³	**88.56**	**103.83**	Pp		
Void Ratio	**0.903**	**0.623**	Pmax		
Saturation, %	**104.57**	**100**	Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-0.5	0.11	0	0.13	0	36.0	Alternating layers of light to dark grey clay, silt line at 4.25".
0.63-1.0	0.11	0	0.1	0	33.9	Alternating layers of light to dark grey clay, silt line at 8"
1.0-1.5	0.15	0	0.24	0	32.9	Alternating layers of light to dark grey clay, silt line at 12.5", black lines at 13.75" and 14".
1.5-1.75	0.06	0			31.6	Alternating layers of light to dark grey clay, black lines at 18.75" and 19.25". Vane stopped on wax at 21".

Comments:

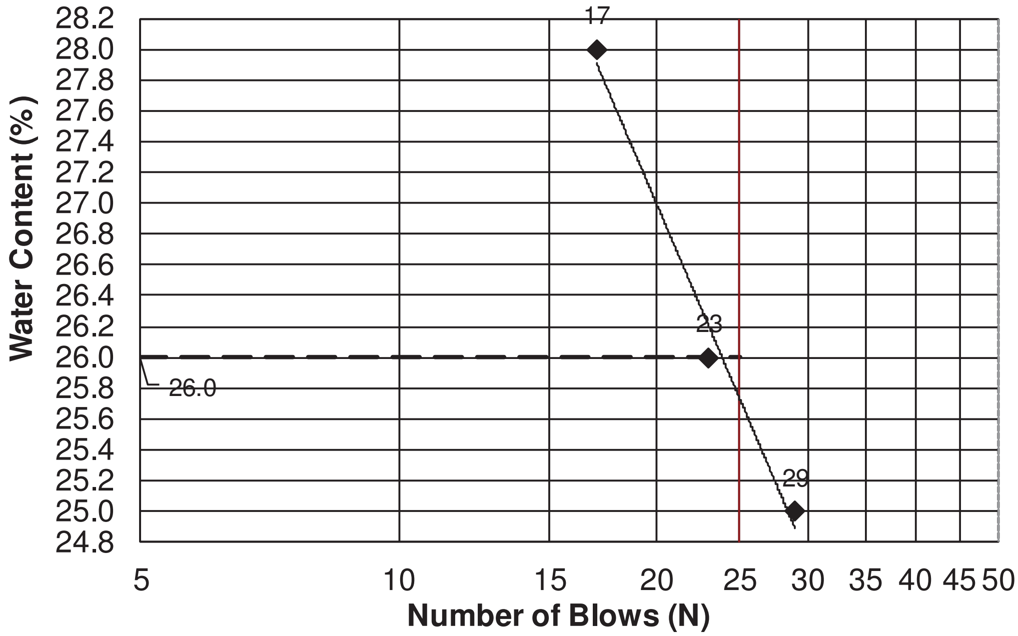
Maine Sensitive Loading Sequence.

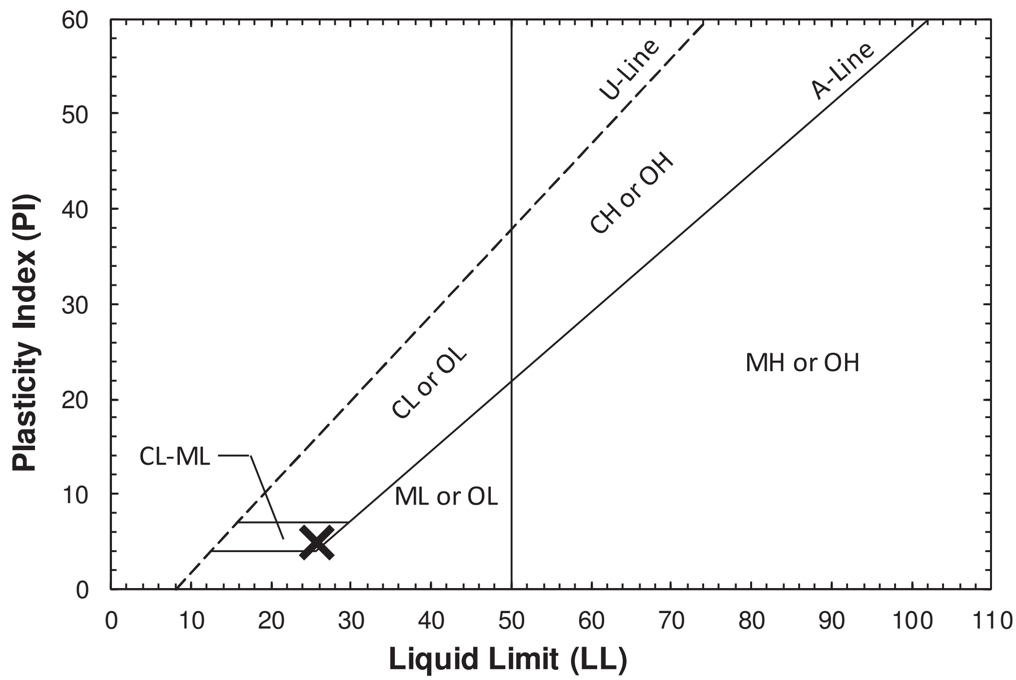
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **5/15/2020**

Paper Copy: Lab File; Project File; Geotech File

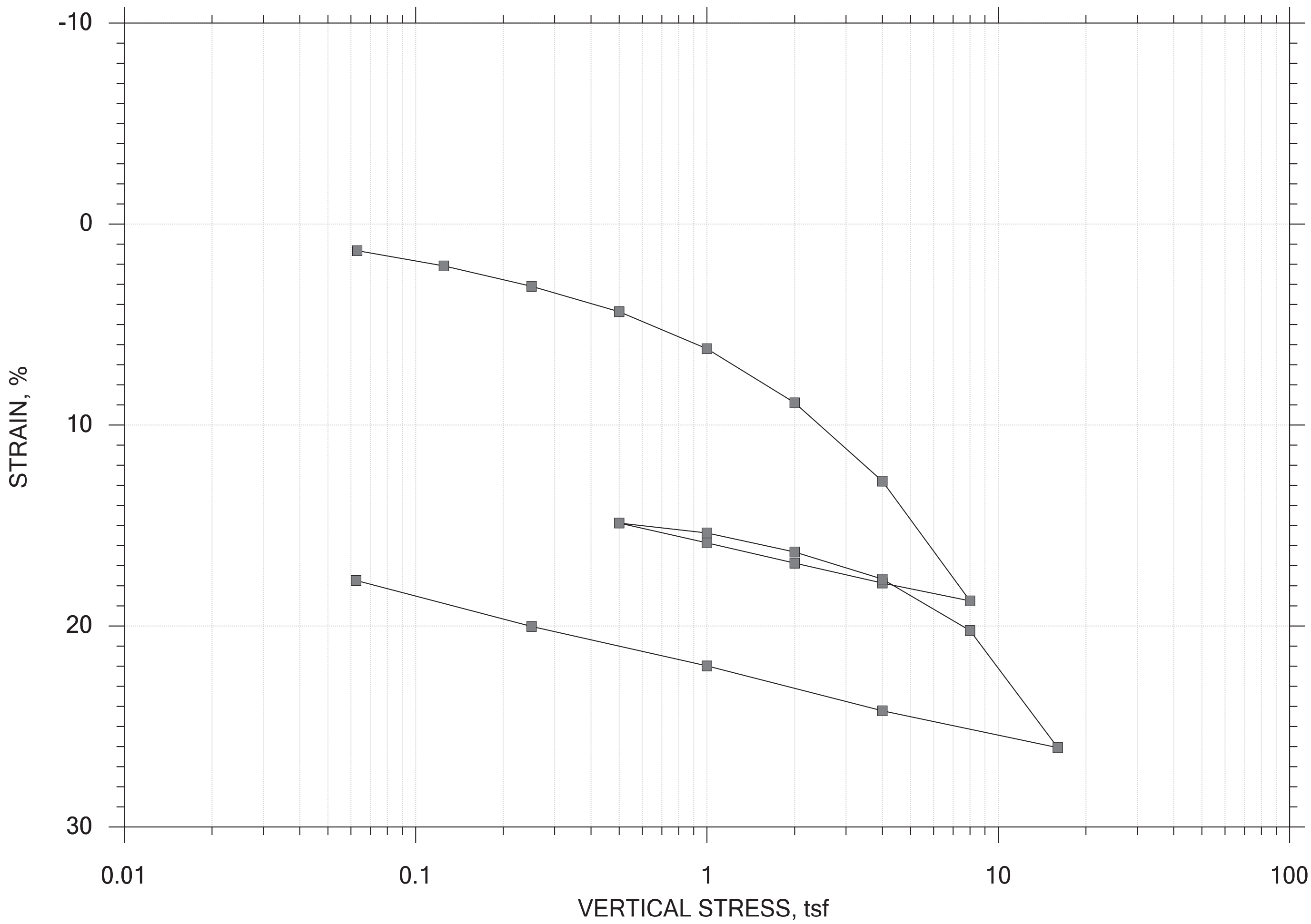
TOWN	Lisbon	Reference No.	300267
WIN	023118.00	Water Content, %	34.7
Sampled	2/12/2020	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-LFB-103/2U	Plastic Limit (T 90), %	21
Station	3+12.1	Plasticity Index (T 90), %	5
Depth	50.0-52.0	Tested By	BBURR

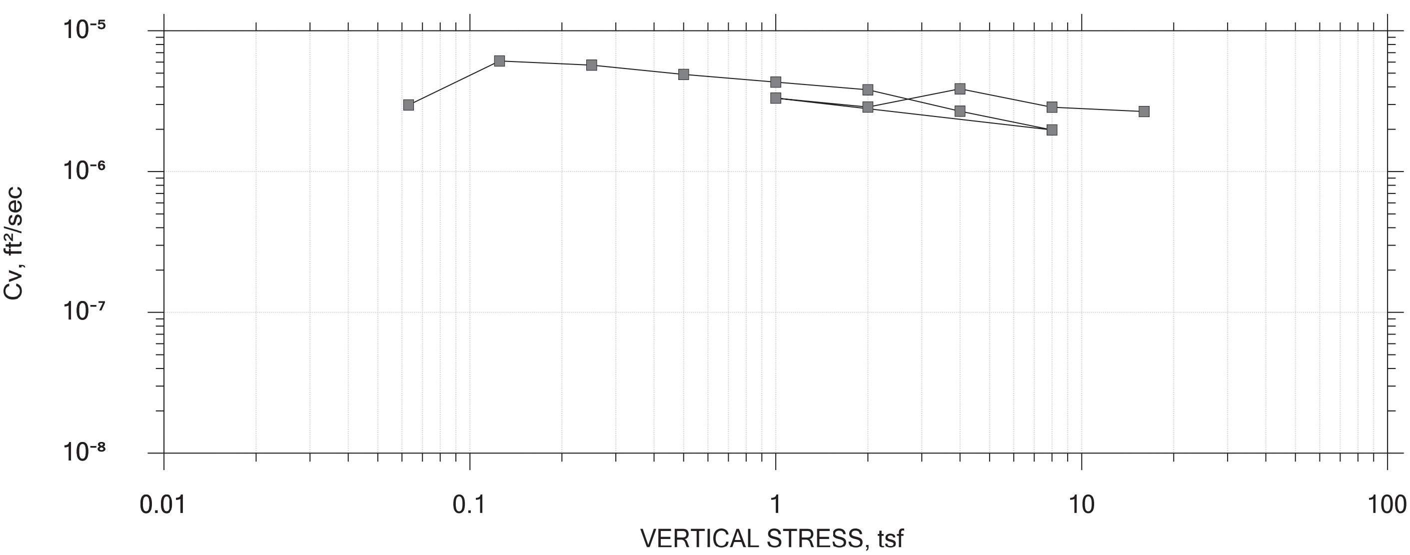




One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

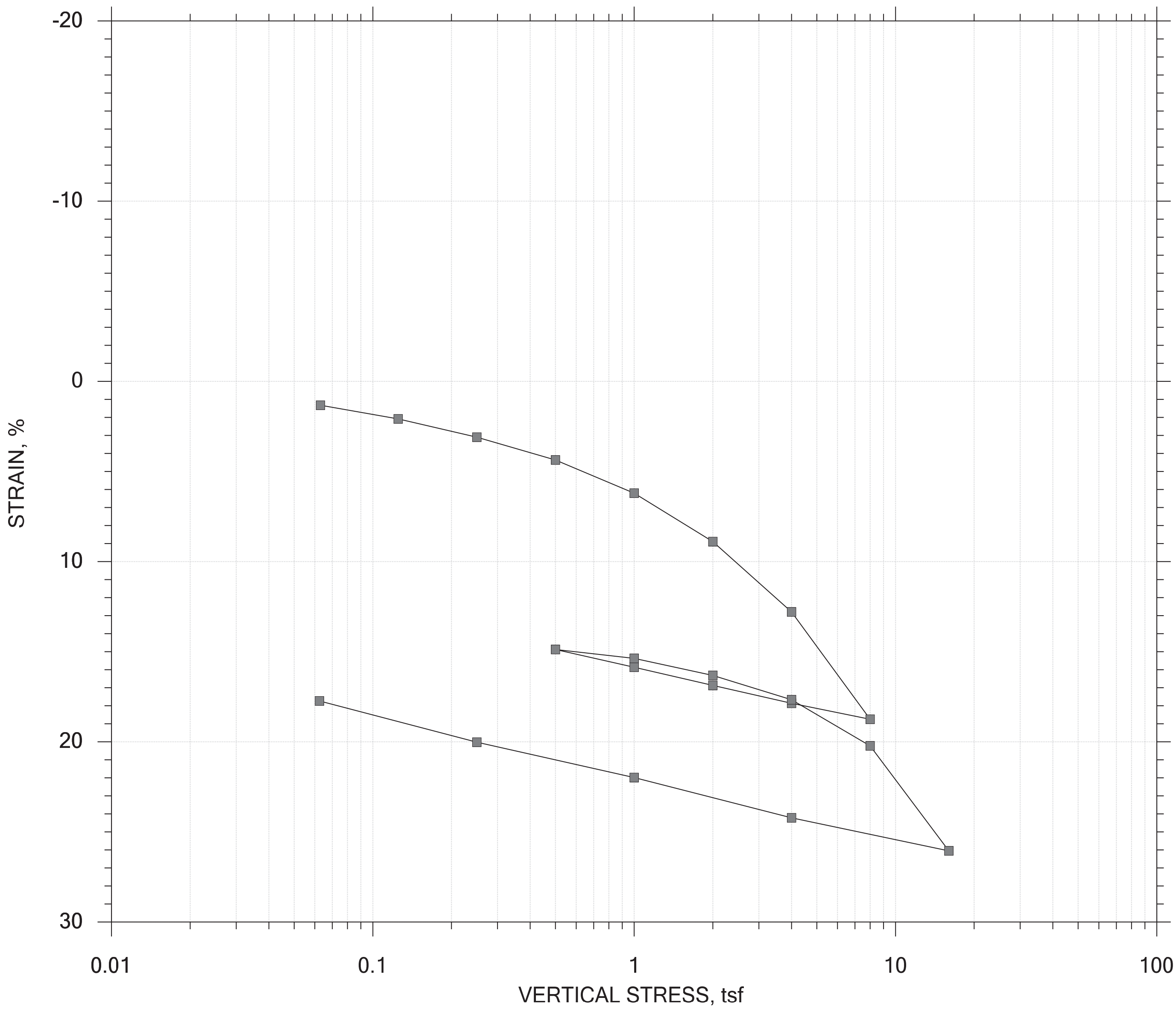




	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 3/3/2020	Test No.: 300266
	Depth: 45.0-47.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: --		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	40.87	30.40
Preconsolidation Stress: ---				Dry Unit Weight, pcf	77.059	92.379
Compression Ratio: ---				Saturation, %	93.22	100.00
Diameter: 2.495 in		Height: 0.9995 in		Void Ratio	1.18	0.82
LL: NP	PL: NP	PI: NP	GS: 2.69			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 3/3/2020	Test No.: 300266
	Depth: 45.0-47.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: --		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon
Location: --
Project No.: 023118.00
Boring No.: BB-LFB-103
Tested By: GSL
Checked By: --
Sample No.: 1U
Test Date: 3/3/2020
Depth: 45.0-47.0 FT
Test No.: 300266
Sample Type: UNDISTURBED
Elevation: --

Soil Description: --
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Measured Specific Gravity: 2.69
Initial Void Ratio: 1.18
Final Void Ratio: 0.818
Liquid Limit: NP
Plastic Limit: NP
Plasticity Index: NP
Specimen Diameter: 2.50 in
Initial Height: 1.00 in
Final Height: 0.83 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	41	RING	RING+BASE	150
Wt. Container + Wet Soil, gm	80.320	401.29	390.95	200.40
Wt. Container + Dry Soil, gm	75.990	360.90	360.90	170.12
Wt. Container, gm	63.410	262.05	262.05	70.530
Wt. Dry Soil, gm	12.580	98.846	98.846	99.590
Water Content, %	34.42	40.87	30.40	30.40
Void Ratio	---	1.18	0.818	---
Degree of Saturation, %	---	93.22	100.00	---
Dry Unit Weight, pcf	---	77.059	92.379	---

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon
Boring No.: BB-LFB-103
Sample No.: 1U
Test No.: 300266

Location: --
Tested By: GSL
Test Date: 3/3/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 45.0-47.0 FT
Elevation: --

Soil Description: --
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

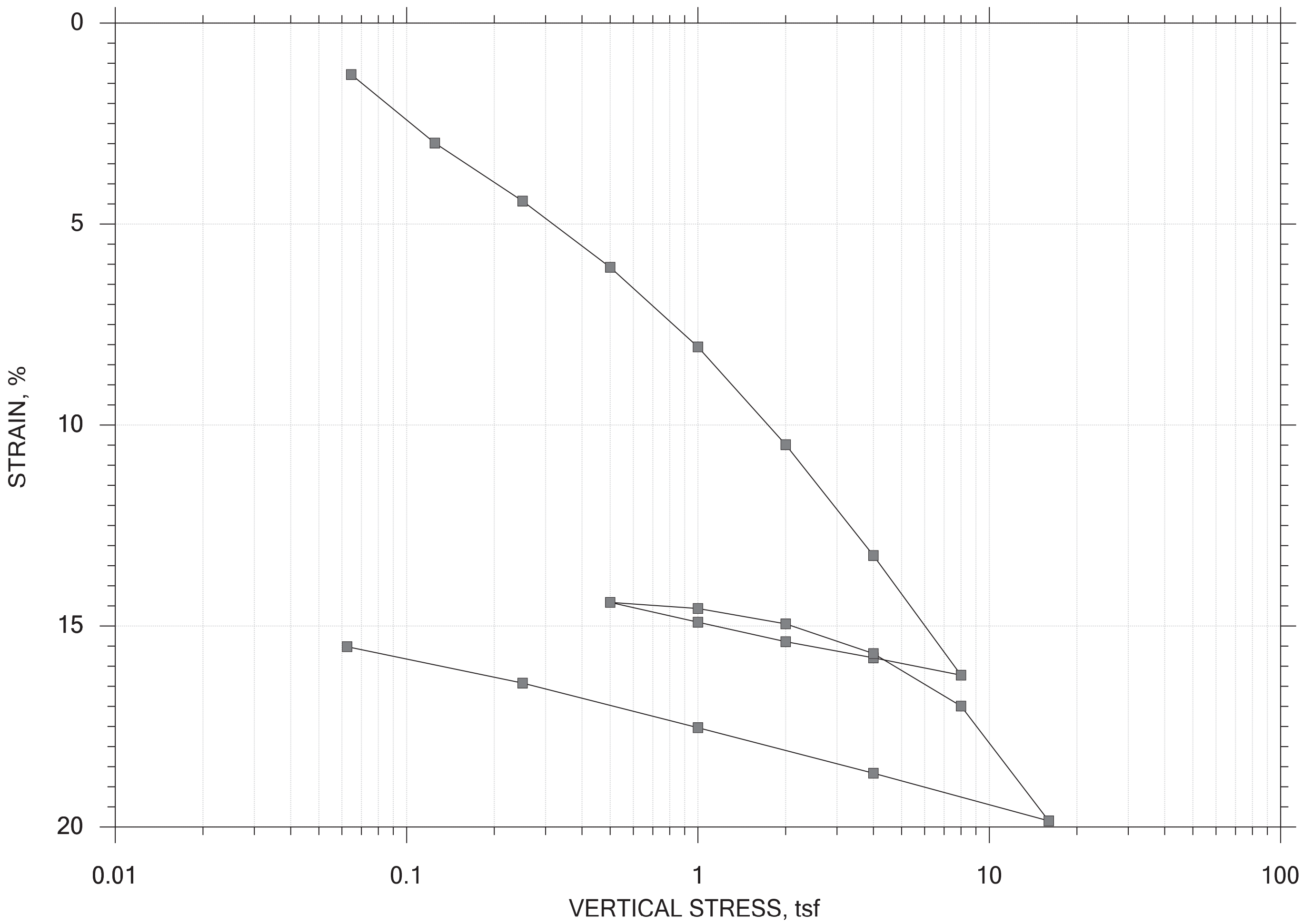
Displacement at End of Increment

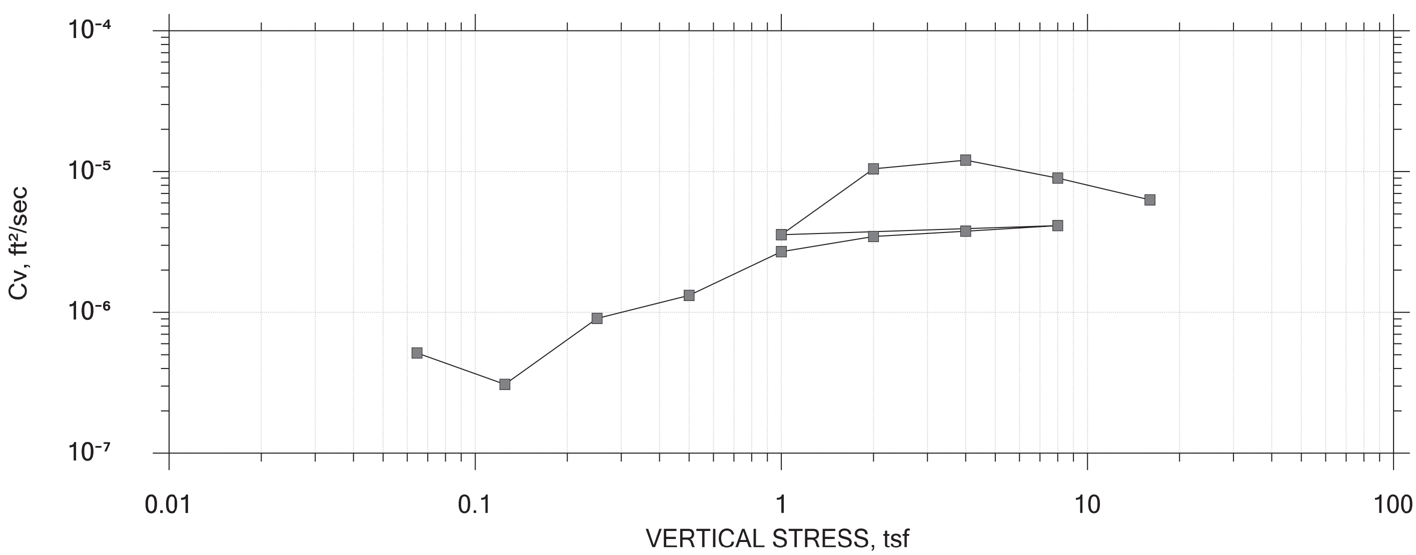
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0630	0.01324	1.15	1.33	8.125	2.98e-006	2.10e-001	1.69e-003
2	0.125	0.02078	1.13	2.08	3.876	6.11e-006	1.22e-001	2.01e-003
3	0.250	0.03090	1.11	3.09	4.077	5.71e-006	8.10e-002	1.25e-003
4	0.500	0.04356	1.08	4.36	4.635	4.90e-006	5.06e-002	6.69e-004
5	1.00	0.06197	1.04	6.20	5.087	4.32e-006	3.68e-002	4.30e-004
6	2.00	0.08889	0.985	8.89	5.501	3.81e-006	2.69e-002	2.77e-004
7	4.00	0.1279	0.900	12.8	7.259	2.68e-006	1.95e-002	1.41e-004
8	8.00	0.1874	0.771	18.7	8.798	1.98e-006	1.49e-002	7.93e-005
9	4.00	0.1785	0.790	17.9	1.851	8.84e-006	2.22e-003	5.30e-005
10	2.00	0.1686	0.812	16.9	4.489	3.73e-006	4.93e-003	4.96e-005
11	1.00	0.1585	0.834	15.9	8.600	1.99e-006	1.01e-002	5.45e-005
12	0.500	0.1487	0.855	14.9	19.656	8.93e-007	1.96e-002	4.72e-005
13	1.00	0.1536	0.844	15.4	5.309	3.33e-006	9.80e-003	8.79e-005
14	2.00	0.1630	0.824	16.3	6.039	2.87e-006	9.43e-003	7.31e-005
15	4.00	0.1765	0.794	17.7	4.358	3.88e-006	6.76e-003	7.07e-005
16	8.00	0.2021	0.739	20.2	5.612	2.87e-006	6.40e-003	4.95e-005
17	16.0	0.2604	0.612	26.1	5.419	2.67e-006	7.29e-003	5.25e-005
18	4.00	0.2421	0.651	24.2	6.065	2.27e-006	1.53e-003	9.32e-006
19	1.00	0.2197	0.700	22.0	10.102	1.43e-006	7.45e-003	2.88e-005
20	0.250	0.2001	0.743	20.0	38.962	3.93e-007	2.62e-002	2.78e-005
21	0.0625	0.1772	0.793	17.7	198.778	8.12e-008	1.22e-001	2.67e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0630	0.01324	1.15	1.33	1.661	3.38e-006	2.10e-001	1.92e-003	0.00e+000
2	0.125	0.02078	1.13	2.08	0.603	9.13e-006	1.22e-001	3.00e-003	0.00e+000
3	0.250	0.03090	1.11	3.09	0.534	1.01e-005	8.10e-002	2.21e-003	0.00e+000
4	0.500	0.04356	1.08	4.36	0.510	1.04e-005	5.06e-002	1.41e-003	0.00e+000
5	1.00	0.06197	1.04	6.20	0.684	7.47e-006	3.68e-002	7.43e-004	0.00e+000
6	2.00	0.08889	0.985	8.89	1.489	3.27e-006	2.69e-002	2.37e-004	0.00e+000
7	4.00	0.1279	0.900	12.8	1.069	4.23e-006	1.95e-002	2.23e-004	0.00e+000
8	8.00	0.1874	0.771	18.7	0.000	0.00e+000	1.49e-002	0.00e+000	0.00e+000
9	4.00	0.1785	0.790	17.9	0.434	8.76e-006	2.22e-003	5.25e-005	0.00e+000
10	2.00	0.1686	0.812	16.9	1.147	3.39e-006	4.93e-003	4.51e-005	0.00e+000
11	1.00	0.1585	0.834	15.9	2.307	1.73e-006	1.01e-002	4.72e-005	0.00e+000
12	0.500	0.1487	0.855	14.9	6.711	6.08e-007	1.96e-002	3.21e-005	0.00e+000
13	1.00	0.1536	0.844	15.4	1.718	2.39e-006	9.80e-003	6.32e-005	0.00e+000
14	2.00	0.1630	0.824	16.3	2.022	1.99e-006	9.43e-003	5.07e-005	0.00e+000
15	4.00	0.1765	0.794	17.7	1.113	3.52e-006	6.76e-003	6.43e-005	0.00e+000
16	8.00	0.2021	0.739	20.2	1.759	2.13e-006	6.40e-003	3.67e-005	0.00e+000
17	16.0	0.2604	0.612	26.1	1.912	1.76e-006	7.29e-003	3.46e-005	0.00e+000
18	4.00	0.2421	0.651	24.2	1.111	2.87e-006	1.53e-003	1.18e-005	0.00e+000
19	1.00	0.2197	0.700	22.0	3.853	8.74e-007	7.45e-003	1.76e-005	0.00e+000
20	0.250	0.2001	0.743	20.0	0.000	0.00e+000	2.62e-002	0.00e+000	0.00e+000
21	0.0625	0.1772	0.793	17.7	0.000	0.00e+000	1.22e-001	0.00e+000	0.00e+000

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

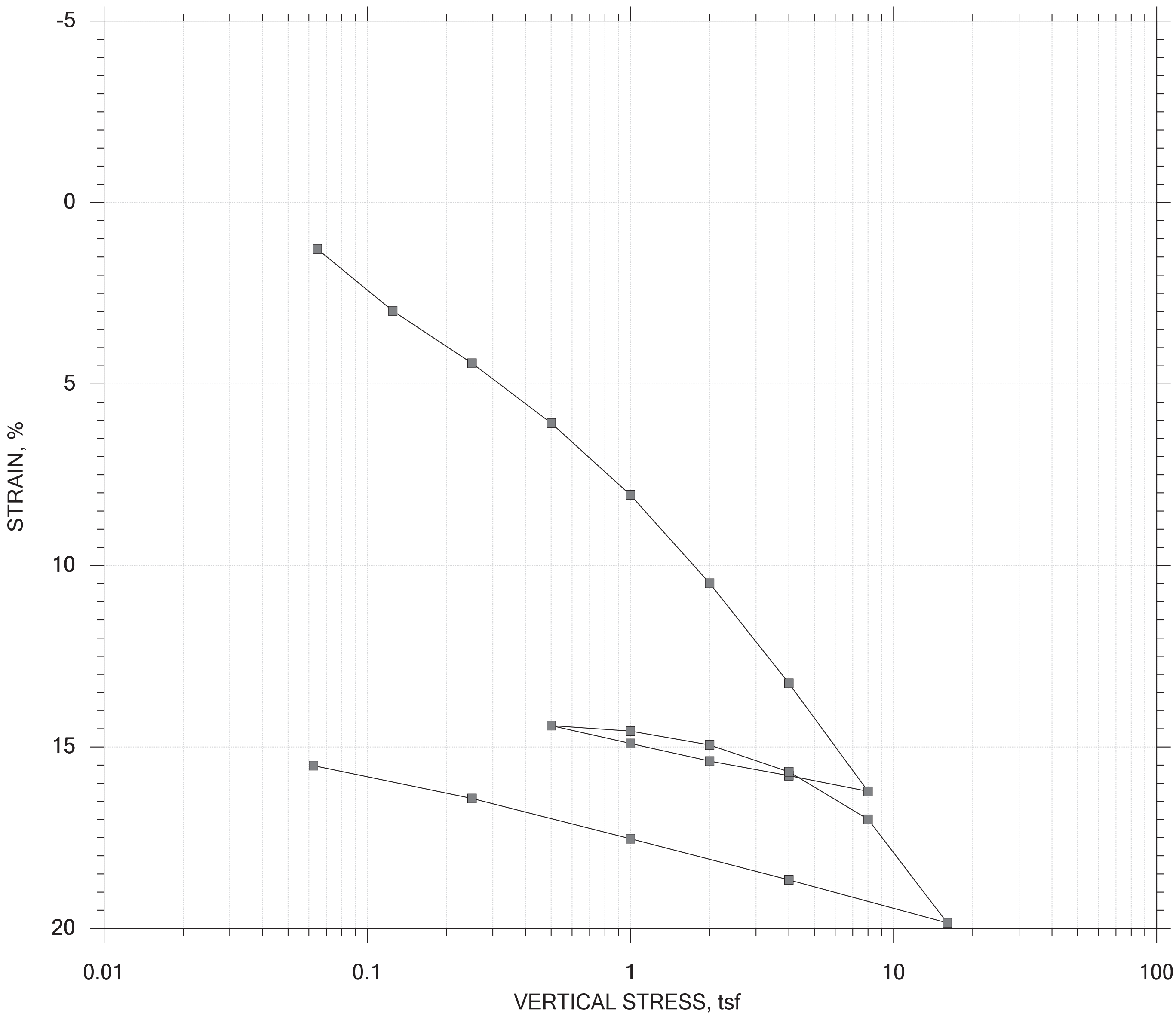




	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 2U	Test Date: 3/18/2020	Test No.: 300267
	Depth: 50.0-52.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: Grey SoftSensitive Clay		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	34.99	23.09
Preconsolidation Stress: ---				Dry Unit Weight, pcf	88.56	103.83
Compression Ratio: ---				Saturation, %	104.57	100.00
Diameter: 2.495 in		Height: 0.9946 in		Void Ratio	0.90	0.62
LL: 27	PL: 21	PI: 6	GS: 2.70			

Project: Lisbon	Location: --	Project No.: 023118.00
Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
Sample No.: 2U	Test Date: 3/18/2020	Test No.: 300267
Depth: 50.0-52.0 FT	Sample Type: UNDISTURBED	Elevation: --
Description: Grey SoftSensitive Clay		
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon
Boring No.: BB-LFB-103
Sample No.: 2U
Test No.: 300267

Location: --
Tested By: GSL
Test Date: 3/18/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 50.0-52.0 FT
Elevation: --

Soil Description: Grey SoftSensitive Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Estimated Specific Gravity: 2.70
Initial Void Ratio: 0.903
Final Void Ratio: 0.623

Liquid Limit: 27
Plastic Limit: 21
Plasticity Index: 6

Specimen Diameter: 2.50 in
Initial Height: 0.99 in
Final Height: 0.85 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	154	RING	RING+BASE	29
Wt. Container + Wet Soil, gm	115.74	414.63	401.18	202.13
Wt. Container + Dry Soil, gm	103.32	375.08	375.08	175.70
Wt. Container, gm	60.210	262.04	262.04	61.220
Wt. Dry Soil, gm	43.110	113.04	113.04	114.48
Water Content, %	28.81	34.99	23.09	23.09
Void Ratio	---	0.903	0.623	---
Degree of Saturation, %	---	104.57	100.00	---
Dry Unit Weight, pcf	---	88.560	103.83	---

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon
Boring No.: BB-LFB-103
Sample No.: 2U
Test No.: 300267

Location: --
Tested By: GSL
Test Date: 3/18/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 50.0-52.0 FT
Elevation: --

Soil Description: Grey SoftSensitive Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0646	0.01271	0.879	1.28	46.528	5.15e-007	1.98e-001	2.75e-004
2	0.125	0.02967	0.847	2.98	75.359	3.09e-007	2.82e-001	2.35e-004
3	0.250	0.04404	0.819	4.43	24.853	9.06e-007	1.16e-001	2.82e-004
4	0.500	0.06041	0.788	6.07	16.498	1.32e-006	6.58e-002	2.34e-004
5	1.00	0.08011	0.750	8.05	7.761	2.70e-006	3.96e-002	2.89e-004
6	2.00	0.1043	0.704	10.5	5.775	3.46e-006	2.43e-002	2.27e-004
7	4.00	0.1317	0.651	13.2	5.001	3.77e-006	1.38e-002	1.40e-004
8	8.00	0.1614	0.595	16.2	4.271	4.13e-006	7.44e-003	8.29e-005
9	4.00	0.1571	0.603	15.8	0.855	2.00e-005	1.08e-003	5.85e-005
10	2.00	0.1531	0.610	15.4	0.949	1.82e-005	2.00e-003	9.84e-005
11	1.00	0.1482	0.620	14.9	2.568	6.80e-006	4.87e-003	8.94e-005
12	0.500	0.1433	0.629	14.4	4.397	4.02e-006	9.85e-003	1.07e-004
13	1.00	0.1448	0.626	14.6	4.985	3.56e-006	3.03e-003	2.91e-005
14	2.00	0.1487	0.619	14.9	1.687	1.05e-005	3.84e-003	1.08e-004
15	4.00	0.1560	0.605	15.7	1.443	1.21e-005	3.68e-003	1.20e-004
16	8.00	0.1690	0.580	17.0	1.891	8.99e-006	3.28e-003	7.94e-005
17	16.0	0.1974	0.526	19.8	2.564	6.30e-006	3.57e-003	6.06e-005
18	4.00	0.1856	0.548	18.7	0.527	3.00e-005	9.88e-004	8.00e-005
19	1.00	0.1743	0.570	17.5	1.676	9.72e-006	3.77e-003	9.88e-005
20	0.250	0.1633	0.591	16.4	9.035	1.85e-006	1.48e-002	7.39e-005
21	0.0625	0.1543	0.608	15.5	25.056	6.84e-007	4.82e-002	8.90e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0646	0.01271	0.879	1.28	0.000	0.00e+000	1.98e-001	0.00e+000	0.00e+000
2	0.125	0.02967	0.847	2.98	0.000	0.00e+000	2.82e-001	0.00e+000	0.00e+000
3	0.250	0.04404	0.819	4.43	0.000	0.00e+000	1.16e-001	0.00e+000	0.00e+000
4	0.500	0.06041	0.788	6.07	4.556	1.11e-006	6.58e-002	1.97e-004	0.00e+000
5	1.00	0.08011	0.750	8.05	2.199	2.21e-006	3.96e-002	2.37e-004	0.00e+000
6	2.00	0.1043	0.704	10.5	1.915	2.42e-006	2.43e-002	1.59e-004	0.00e+000
7	4.00	0.1317	0.651	13.2	1.211	3.62e-006	1.38e-002	1.35e-004	0.00e+000
8	8.00	0.1614	0.595	16.2	1.149	3.57e-006	7.44e-003	7.16e-005	0.00e+000
9	4.00	0.1571	0.603	15.8	0.000	0.00e+000	1.08e-003	0.00e+000	0.00e+000
10	2.00	0.1531	0.610	15.4	0.000	0.00e+000	2.00e-003	0.00e+000	0.00e+000
11	1.00	0.1482	0.620	14.9	0.438	9.28e-006	4.87e-003	1.22e-004	0.00e+000
12	0.500	0.1433	0.629	14.4	1.012	4.06e-006	9.85e-003	1.08e-004	0.00e+000
13	1.00	0.1448	0.626	14.6	0.318	1.30e-005	3.03e-003	1.06e-004	0.00e+000
14	2.00	0.1487	0.619	14.9	0.000	0.00e+000	3.84e-003	0.00e+000	0.00e+000
15	4.00	0.1560	0.605	15.7	0.322	1.26e-005	3.68e-003	1.25e-004	0.00e+000
16	8.00	0.1690	0.580	17.0	0.441	8.95e-006	3.28e-003	7.91e-005	0.00e+000
17	16.0	0.1974	0.526	19.8	0.690	5.44e-006	3.57e-003	5.23e-005	0.00e+000
18	4.00	0.1856	0.548	18.7	0.000	0.00e+000	9.88e-004	0.00e+000	0.00e+000
19	1.00	0.1743	0.570	17.5	0.607	6.23e-006	3.77e-003	6.33e-005	0.00e+000
20	0.250	0.1633	0.591	16.4	2.219	1.75e-006	1.48e-002	6.99e-005	0.00e+000
21	0.0625	0.1543	0.608	15.5	6.497	6.13e-007	4.82e-002	7.97e-005	0.00e+000

Appendix C

Calculations

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

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

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

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

Outlet Walls Fixed to Box
At-Rest Earth Pressure - Rankine Theory

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

$K_o = 0.47$

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

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

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

For cantilver walls with horizontal backslope:

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

$K_{ar} = 0.31$

For a sloped 2H:1V backfill

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

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

$K_{ar\_slope} = 0.46$

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

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

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

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

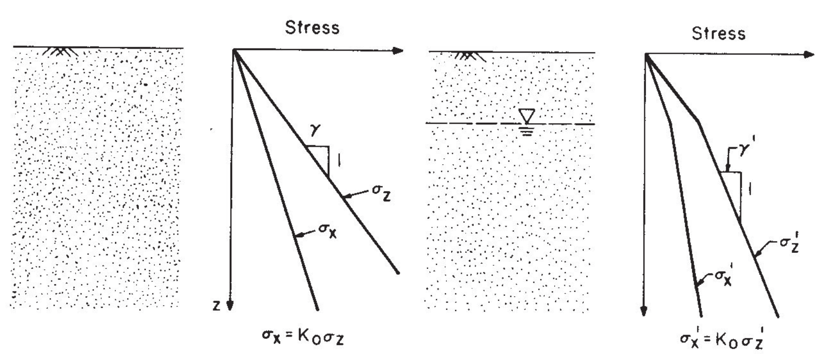


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

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:

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

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:

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

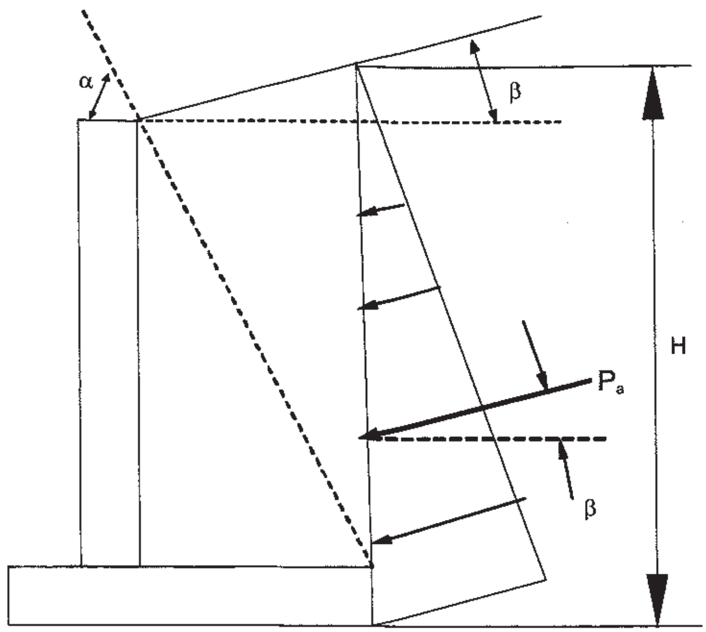


Figure 3-3 Rankine Theory

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

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 1 foot, which accounts for the possible scouring away of 1 foot of special fill.
2. Subgrade at the base of the 2-foot thick bedding mat is medium dense, fine sand (Marine Nearshore Deposits).
3. The proposed bearing elevation is approximately 113.7 feet.
4. Proposed finish roadway grade elevation is approximately 124.3 feet at the low point.
5. Proposed precast concrete box base is 18 feet wide.
6. The bottom of the box culvert will be submerged for the structure's design life.

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 8th Edition Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2*, *May 1983*, *Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Fine to medium sand, silty or clayey medium to fine coarse sand (SW, SM, SC)	Medium dense to dense	4-8	4

Recommend 4 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Foundation Width, Depth, and Water Surface

$$B := 18\text{ft}$$

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

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

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

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

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

$\gamma_{above\_sat} := 135 \cdot pcf$

Foundation soils:

Foundation soils based on BB-LFB-101 3D

$\gamma 1_d := 103.5 \cdot pcf$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2, Medium Dense Fine Sand, use av. of 92 and 115 pcf.

$w_{sat} := 28.5\%$

$\gamma 1_{sat} := \gamma 1_d \cdot (1 + w_{sat})$ Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit weight relationships

$\gamma 1_{sat} = 133 \cdot pcf$

$\phi := 30 \cdot deg$ AASHTO Table 10.4.6.2.4 Correlation of N60 Values to Drained Friction Angle of Granular Soils, 2017, based on N=12 bpf

$c := 0$

Nominal Bearing Resistance for Strength Limit States

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

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

$N_c := 30.1$

$N_q := 18.4$

$N_\gamma := 22.4$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$L := 53.5 \cdot ft$

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

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

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

$s_c = 1.2$ $\quad s_\gamma = 0.9$ $\quad s_q = 1.2$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

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

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

Load Inclination factors

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

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

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

LRFD Table 10.6.3.1.2a-4 $\quad \dfrac{D_f}{B} = 0.06$

Therefore : $\quad d_q := 1.0$

Terms

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

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

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

$N_{cm} = 36.3$ $\quad N_{\gamma m} = 19.4$ $\quad N_{qm} = 22$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c \cdot N_{cm} + \gamma_{above\_sat} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma1_{sat} \cdot \overrightarrow{\left(B \cdot N_{\gamma m}\right)} \cdot C_{w\gamma}\right]$$

$q_n = 13.1 \cdot ksf$

Factored Bearing Resistance

$$\phi_b := 0.45$$

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

$$\boxed{q_r = 5.9 \cdot ksf}$$

Recommend a factored bearing resistance of 6 ksf for box bottom slabs 18 ft or greater.

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.

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	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				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

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3 f'_c$.

AASHTO LRFD 8th edition, 2017

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.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3—Driven Piles

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

C10.5.5.2.3

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method (see Articles 10.7.3.8.2, 10.7.3.8.3, 10.7.3.8.4, or 10.7.3.8.5). The actual penetration depth where the pile is stopped using the driving criterion (e.g., a blow count measured during pile driving) will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the nominal pile bearing resistance is dependent on the reliability of the

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

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:

$$\boxed{q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \ B N_{\gamma m} C_{w\gamma}} \qquad (10.6.3.1.2a\text{-}1)$$

in which:

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

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

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

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)

C10.6.3.1.2a

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.

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)

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.

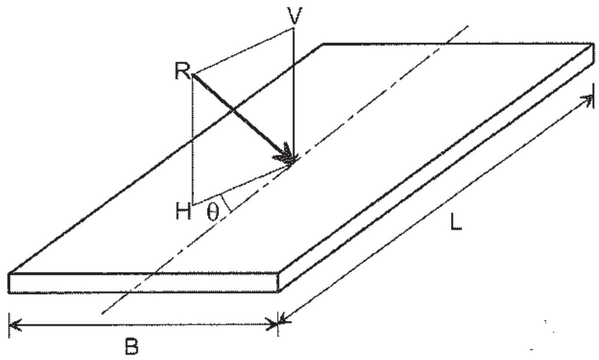


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, ϕ_f. Information beyond the range indicated is not available at this time.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2\text{b-1})$$

$$\phi^* = \tan^{-1}(0.67\tan\phi_f) \quad (10.6.3.1.2\text{b-2})$$

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)

ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

C10.6.3.1.2b

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

Modulus of Subgrade Reaction

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

Given:
1. Limited lab data and SPT N-values.

Assumptions:
1. The proposed bearing elevation of base slab is approximately 113.7 feet.
2. Proposed finished roadway grade is approximately 124.3.
3. Proposed precast concrete box is 18 feet wide and approximately 70.5 feet long.
4. The subsurface conditions present at the proposed bearing elevation is Marine Nearshore Deposit SAND, based on BB-LFB-101 3D N=12 bpf (medium dense).
5. The bottom of the box culvert will be submerged for the structure's design life.

Published values of subgrade modulus

Published values of subgrade modulus in medium sand:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction 35 to 295 pci
Subgrade of medium dense sand, average of upper and lower limit: k_s = 165 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Saturated fine grained medium dense sand K_{v1}, 145 pci / 2 = 72.5 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:
Typical subgrade reaction values for 0.3 m x 0.3 m plate
Saturated medium dense sand, 129 - 147 pci: $k_{0.3}$ (k_1) = 138 pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:
Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand
Submerged dense sand: k_{s1} = 93 pci

Adjust Published values for dimensions of base slab

Select a subgrade modulus of 145 pci for saturated dense sand based on GEC No. 6. and divided by 2 per Note 2 (see references attached).

Value of k_{s1} = 72.5 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B - 18 ft, Length L = 70.5 ft).

Square to rectangle base adjustment:

$$k_{s1} := 72.5\text{pci} \quad B := 18\text{ft} \qquad L := 70.5\text{ft}$$

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

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

$$k = 54.5 \cdot \text{pci}$$

Recommend a subgrade modulus of 54.5 pci for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \qquad (9\text{-}10)$$

r either horizontal or vertical members
for depth variation
erest 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

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

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

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:

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

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

lepth 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

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

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

TABLE 9-1
Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

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

Soil	k_s, kN/m³	ks, lb/in^3
Loose sand	4800–16 000	18 - 59
Medium dense sand	9600–80 000	35 - 295	165 pci
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

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

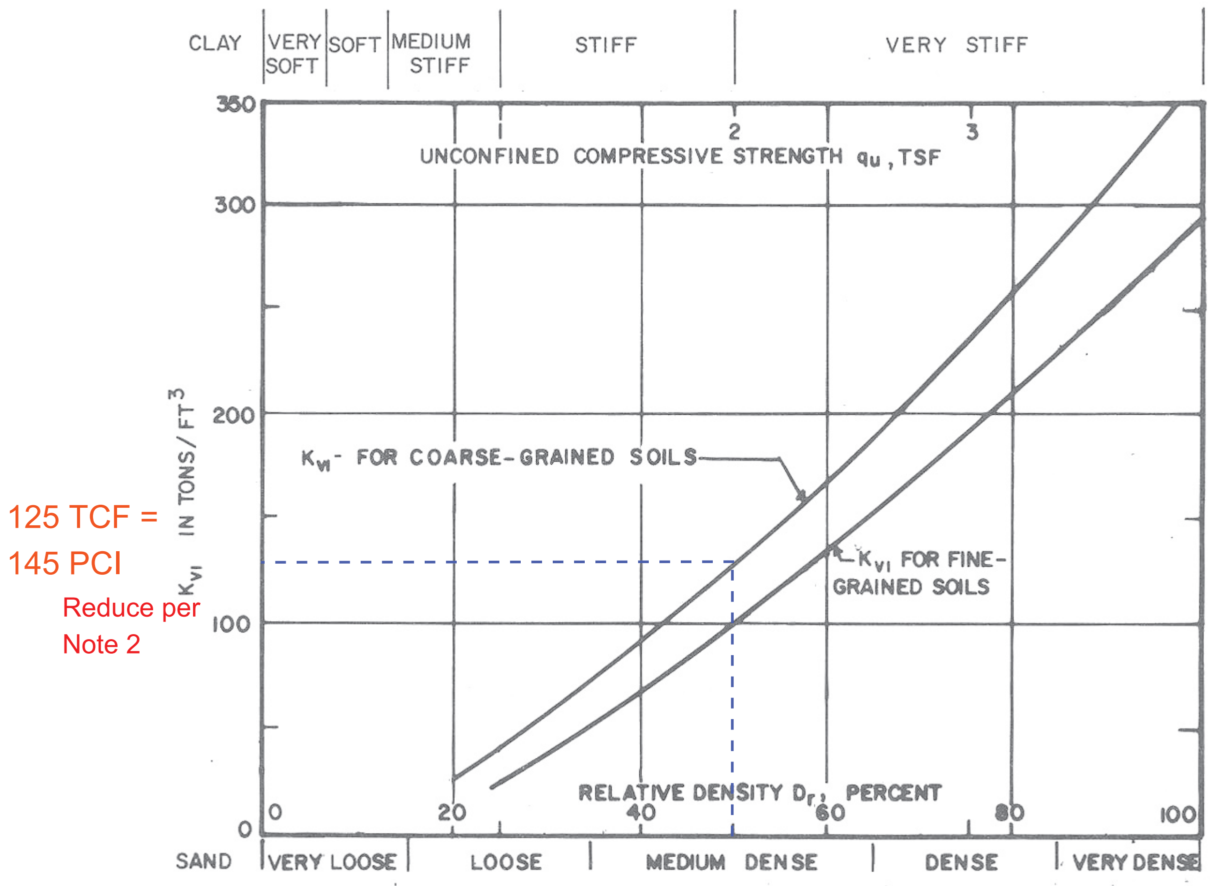
$$k_s = \frac{1}{0.61(0.077\,65)(4 \times 0.597)(0.8)} = \mathbf{11.05} \text{ MN/m}^3$$

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)]



DEFINITIONS

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

D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{v1} = MODULUS OF VERTICAL SUBGRADE REACTION

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

COARSE-GRAINED SOILS

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

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

FOR $B \geq 40$ FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$
FOR $B \leq 20$ FT:

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

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{v1} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{v1}/2$ IN COMPUTING SETTLEMENT

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

 Das, Priciples of Foundation Engineering 7th Ed.

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

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

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

Terzaghi, Geotechnique, Vol. 5 No. 4

the bending moments in piles which are acted upon by horizontal forces above the ground surface (Cummings, 1937) and of those in core-walls of earth- and rock-fill dams (Löfquist, 1951).

Attempts have also been made to apply the theories to the solution of bulkhead problems (Rifaat, 1935). Baumann (1935) used them for estimating the stresses in an anchored bulkhead which had failed. Quite recently Blum (1951) proposed a procedure for the design of anchored bulkheads by means of the theory of horizontal subgrade reaction. All these investigations and design procedures were based on the tacit assumption that K'_0 in equation (15) is identical with the coefficient of active earth pressure K_a. The error due to this assumption may be quite important.

EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION

General procedure

The numerical values of the coefficients of subgrade reaction k_s and k_h required for the solution of engineering problems can either be estimated on the basis of published observational data or else they can be derived from the results of field tests to be performed on the subgrade of the proposed structure. For practical purposes, rough estimates of these values fully serve their purpose.

Vertical subgrade reaction

As a basis for estimating the coefficient of subgrade reaction k_s for beams and slabs, the value $\bar{k}_{s1}$ for a square plate with a width of 1 ft has been selected, because this value can, if necessary, be determined by averaging the results of several loading tests in the field, at the site of the structure.

If the subgrade consists of cohesionless or slightly cohesive sand, k_s can be estimated on the basis of the empirical values of $\bar{k}_{s1}$ given in Table 1. The density-category of the sand can be ascertained by means of a standard penetration test or other convenient means. The greatest error on the unsafe side results from using the proposed 130 value in the case of medium sand if its real value is equal to the lower limiting value of 60 tons/cu. ft.

Table 1.

Values of $\bar{k}_{s1}$ in tons/cu. ft for square plates, 1 ft × 1 ft, or beams 1 ft wide, resting on sand

	Relative density of sand	Loose	Medium	Dense
	Dry or moist sand, limiting values for $\bar{k}_{s1}$	20–60	60–300	300–1,000
	Dry or moist sand, proposed values	40	130	500
Medium Dense sand	Submerged sand, proposed values	25	$\boxed{80}$	300

In order to investigate the influence of such an error on the results of the computation of the bending moments in a beam, the maximum bending moment M_{max} in the beam shown in Fig. 1 was computed on the basis of both the assumed and the real value of $\bar{k}_{s1}$ for the supporting sand. The value of M_{max} for this beam is determined by equation (4). It was found that the moment computed by means of the proposed value exceeds the actual bending moment by not more than about 5%.

Once the value $\bar{k}_{s1}$ has been selected, the value of k_s to be used in the solution of a given

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

80 ton/ft^3 x 1.157 = 93 pci

EVAL

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

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

Va fo

Values Range Propos

For rec

* Highe

If the subgrade ately in simple pro basis of our presen numerical values o pressures which ar The latter is indep The proposed v medium sand, Tab of the loaded area normally consolida beams and rafts sh perfectly rigid. The $\bar{k}_{s1}$ values f of the tests can be of such tests is too the test results can of the block shoul If the contact the value :

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

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 $\times$ 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 $\times$ 0.3 m and $B\,(\mathrm{m}) \times B\,(\mathrm{m})$, respectively (unit is kN/m^3).

Foundations on Clays

For foundations on clays,

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

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

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

Method 1:

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

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 $B \times B$

Frost

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

From Design Freezing Index Map: Lisbon**, Maine**
DFI = 1400 degree-days.
Case 1 - coarse grained granular fill soils W=25% (assumed).

For DFI = 1400

at w=20% $d_1 := 65.5in$

at w=30% $d_2 := 56.4in$

Depth of Frost Penetration

$$d := \frac{d_2 + d_1}{2} \qquad d = 60.95 \cdot in \qquad d = 5.1 \cdot ft$$

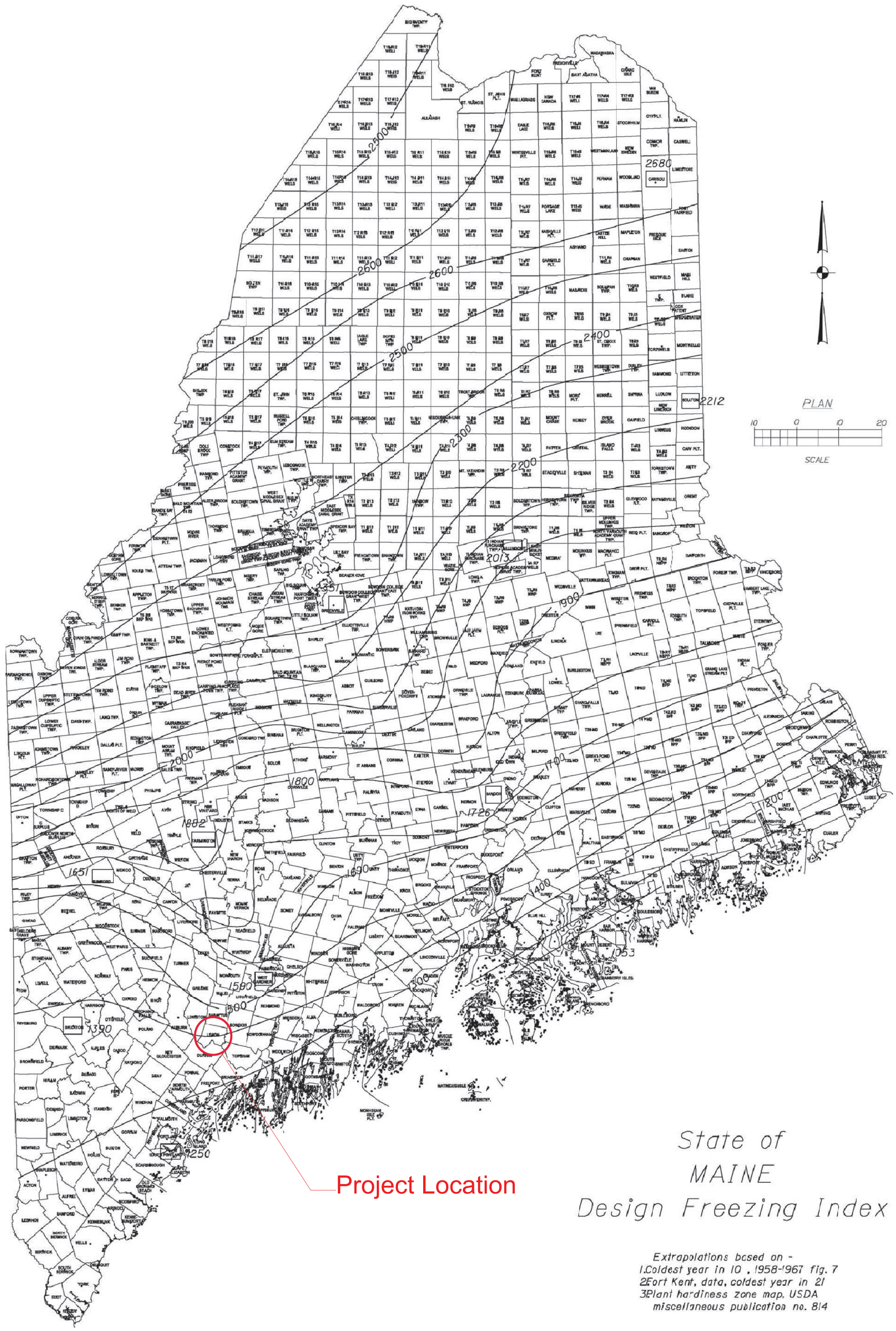


Figure 5-1 Maine Design Freezing Index Map

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

Settlement

Settle3D Analysis Information
Lisbon Frazier Bridge

Project Settings

Document Name	23118LisbonFrazierSettleR7.s3z
Project Title	Lisbon Frazier Bridge
Analysis	Immediate, 1-year consolidation, and secondary consolidation
Author	Manahan/Check L Krusinski
Company	MaineDOT
Date Created	8/13/2020 rev7 12/9/2020
Comments	
SVI load = 2750 psf (RN)	
Delta q = 1781 psf at bottom slab ele. outside horizontal	
limits of existing pipe arch	
Model 2-foot crushed stone mat with geogrid	
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Calculate settlement with mean stress	
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: 1.21365 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.73208
Consolidation Settlement [in]	-0.00113087	0.395981
Immediate Settlement [in]	0	2.3361
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.781
Effective Stress [ksf]	0	3.69041
Mean Stress [ksf]	-6.30698e-011	1.33577
Total Stress [ksf]	0	7.1052
Total Strain	-0.000200545	0.0092997
Pore Water Pressure [ksf]	0	3.4148
Excess Pore Water Pressure [ksf]	-8.4093e-011	1.78102
Degree of Consolidation [%]	0	64.159
Pre-consolidation Stress [ksf]	0.025875	5.44038
Over-consolidation Ratio	1	1.80619
Void Ratio	0	1.17985
Permeability [ft/y]	0	0.0291212
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.983282

Stage: Consolidation = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.80048
Consolidation Settlement [in]	0	0.464386
Immediate Settlement [in]	0	2.3361
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.781
Effective Stress [ksf]	0	3.67187
Mean Stress [ksf]	-6.30698e-011	1.33577
Total Stress [ksf]	0	7.1052
Total Strain	-0.000200545	0.0102706
Pore Water Pressure [ksf]	0	3.43333
Excess Pore Water Pressure [ksf]	0	0.126134
Degree of Consolidation [%]	0	74.3074
Pre-consolidation Stress [ksf]	0.025875	5.44038
Over-consolidation Ratio	1	1.79816
Void Ratio	0	1.17956
Permeability [ft/y]	0	0.0291212
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	62.1317
Undrained Shear Strength	0	0.986178

Stage: Long-term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.20755
Consolidation Settlement [in]	0	0.624902
Immediate Settlement [in]	0	2.3361
Secondary Settlement [in]	0	0.24655
Loading Stress [ksf]	0	1.781
Effective Stress [ksf]	0	3.79796
Mean Stress [ksf]	-6.30698e-011	1.33577
Total Stress [ksf]	0	7.1052
Total Strain	-0.000200545	0.0123946
Pore Water Pressure [ksf]	0	3.30724
Excess Pore Water Pressure [ksf]	0	4.11419e-005
Degree of Consolidation [%]	0	99.9917
Pre-consolidation Stress [ksf]	0.025875	5.44038
Over-consolidation Ratio	1	1.78847
Void Ratio	0	1.17956
Permeability [ft/y]	0	0.0291212
Coefficient of Consolidation [ft^2/y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	99.9883
Undrained Shear Strength	0	0.992429

Loads

1. Rectangular Load

Length	53.5 ft
Width	18 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	963 ft^2
Load	1.781 ksf
Depth	11 ft
Installation Stage	Immediate = 0 y

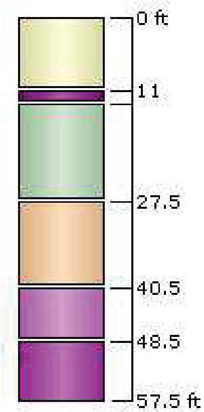
Coordinates

X [ft]	Y [ft]
-26.75	-9
26.75	-9
26.75	9
-26.75	9

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Fill	11	0	No
2	2-foot Crushed Stone Mat with Geogrid	2	11	No
3	Med Dense Marine Sands	14.5	13	No
4	Med Dense Marine Sands Med Stiff Silt	13	27.5	No
5	Soft Upper Silt-Clay	8	40.5	No
6	V. Soft Silt-Clay	9	48.5	No



Soil Properties

Property	Loose Fill	Med Dense Marine Sands	Med Dense Marine Sands Med Stiff Silt	Soft Upper Silt-Clay	2-foot Crushed Stone Mat with Geogrid	V. Soft Silt-Clay
Color						
Unit Weight [kips/ft^3]	0.115	0.115	0.115	0.11	0.125	0.11
Saturated Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.115	0.13	0.115
Poisson's Ratio	0.25	0.25	0.25	0.2	0.2	0.2
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled	Enabled
E [ksf]	250	319	261	73	2000	73
Eur [ksf]	1000	1276	1044	292	8000	292
Primary Consolidation	Disabled	Disabled	Disabled	Enabled	Disabled	Enabled
Material Type				Non-Linear		Non-Linear
Cc				0.427		0.17
Cr				0.068		0.035
e0				1.18		0.9
OCR	1	1	1	1.8	1	1
Cv [ft^2/y]				36.5		36.5
B-bar				1		1
Secondary Consolidation	Disabled	Disabled	Disabled	Mesri	Disabled	Mesri
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	4.5 ft

Query Points

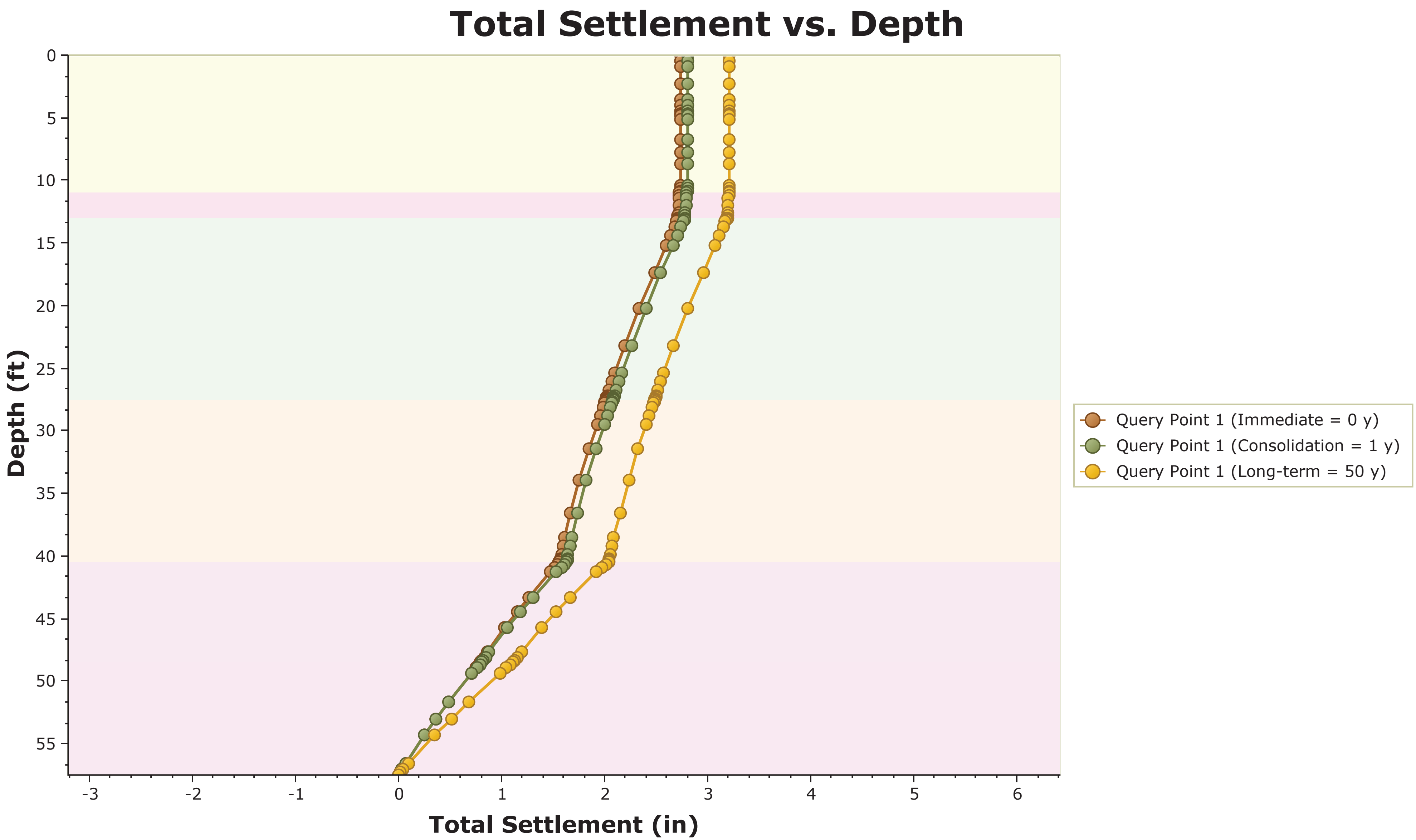
Point #	(X,Y) Location	Number of Divisions
1	-2.13163e-014, 7.10543e-015	Auto: 79

Field Point Grid

Number of points	294
Expansion Factor	2

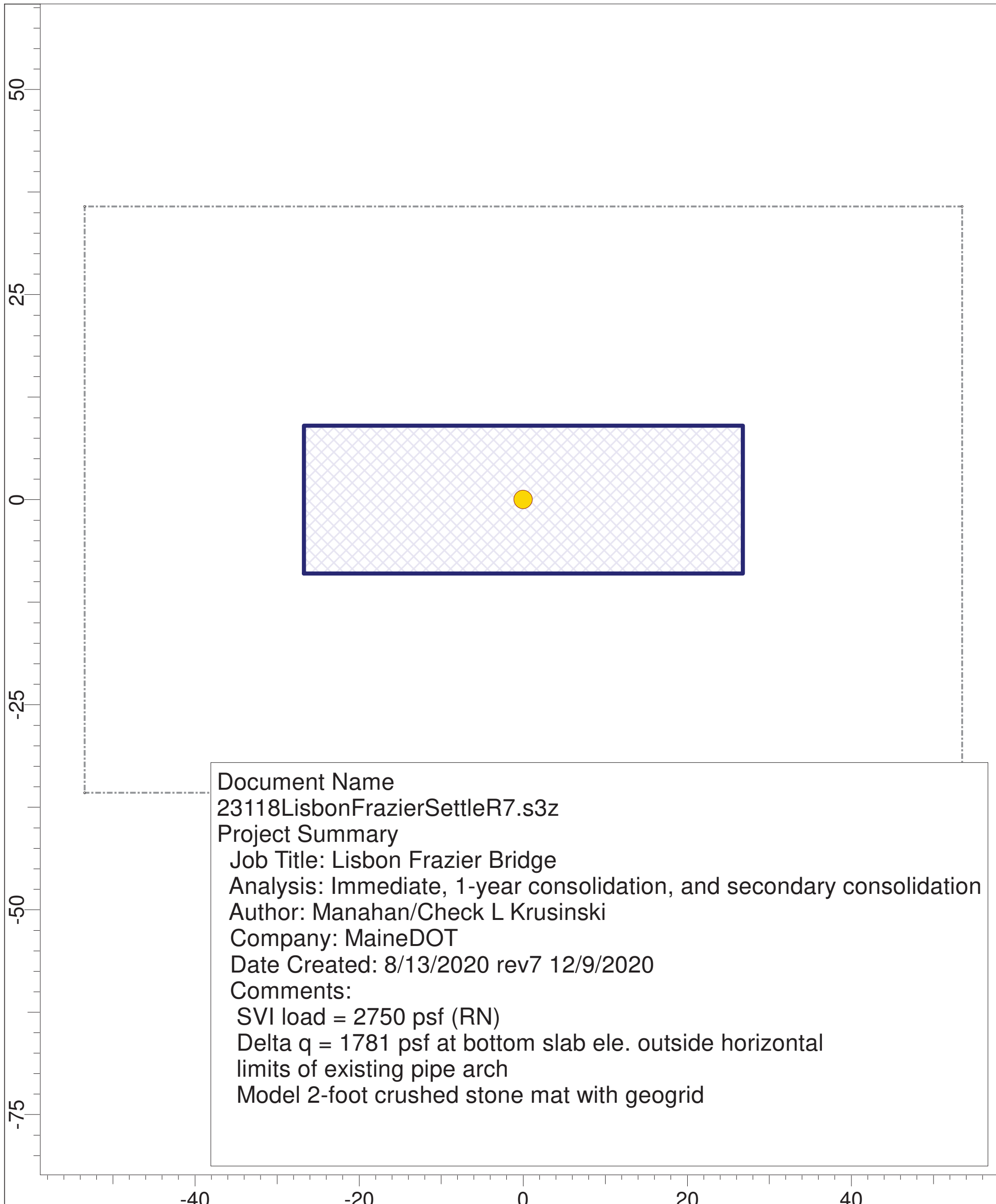
Grid Coordinates

X [ft]	Y [ft]
53.5	35.75
53.5	-35.75
-53.5	-35.75
-53.5	35.75

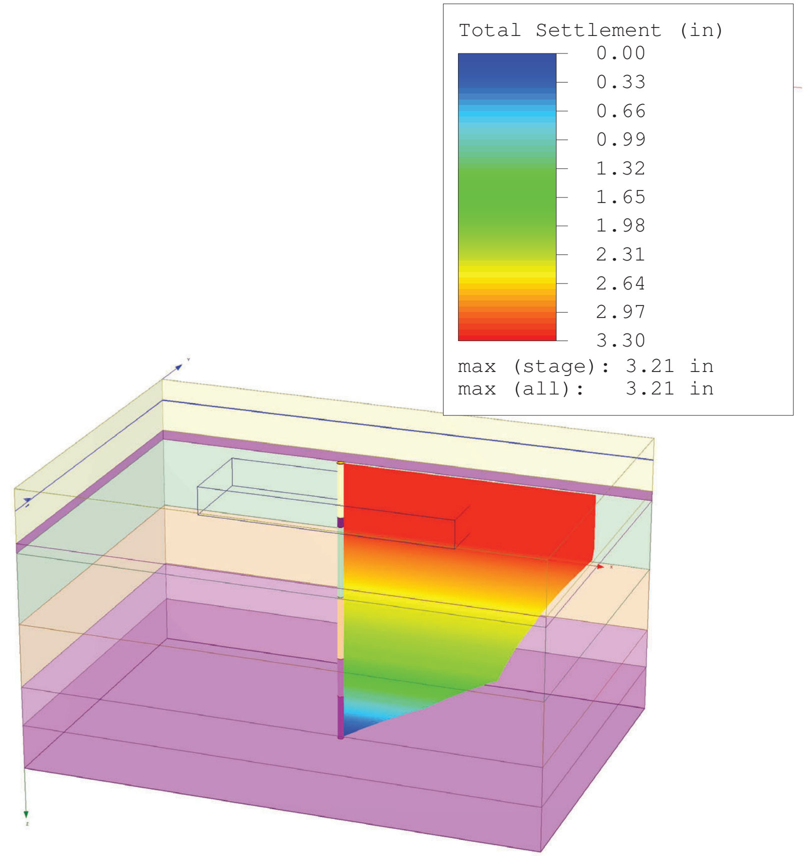


Reference Stage: None

SETTLE3D 3.015	Project	Lisbon Frazier Bridge	
	Analysis Description	Immediate, 1-year consolidation, and secondary consolidation	
	Drawn By	Manahan/Check L Krusinski	Company: MaineDOT
	Date	8/13/2020 rev7 12/9/2020	File Name: 23118LisbonFrazierSettleR7.s3z



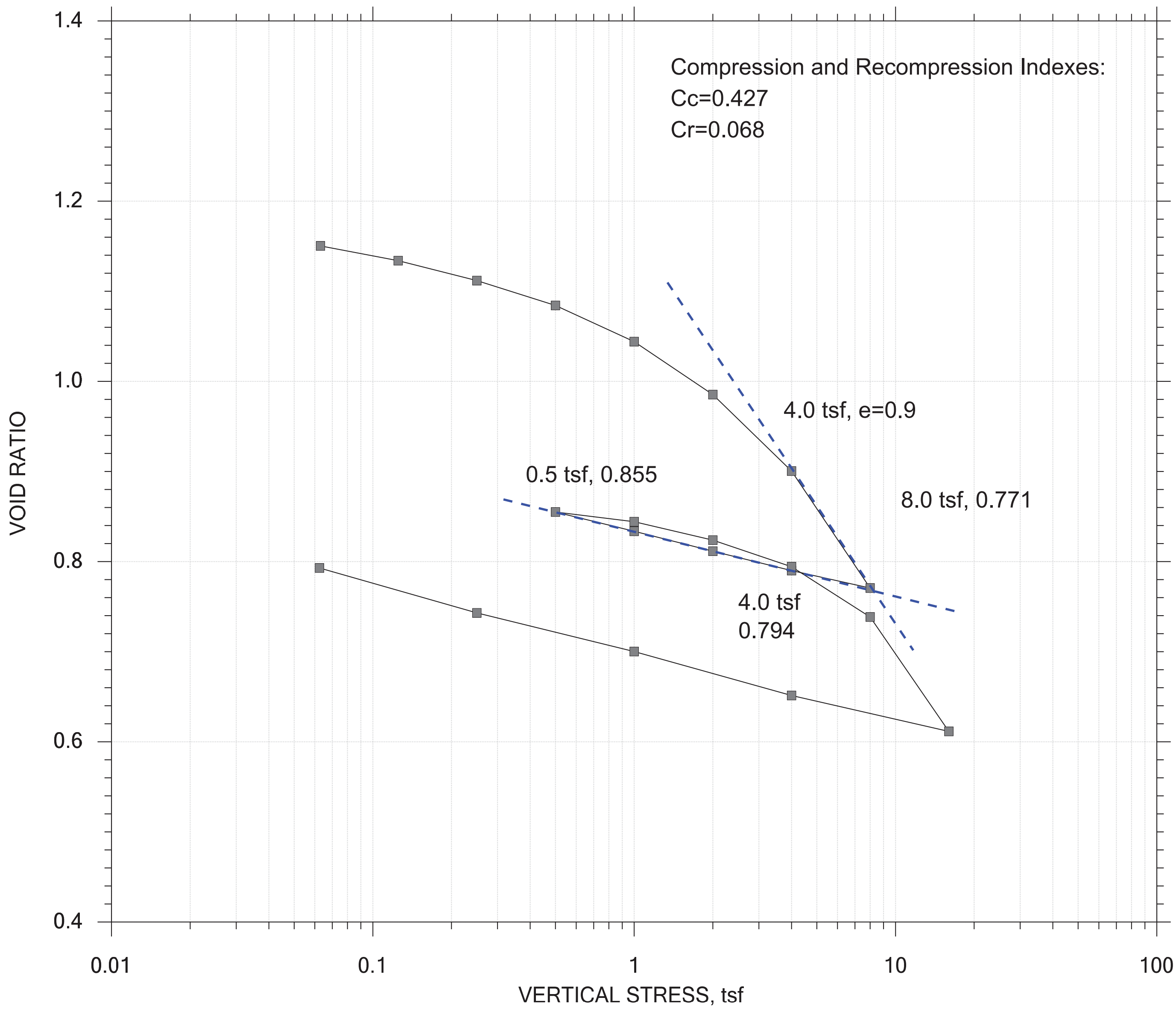
Document Name
23118LisbonFrazierSettleR7.s3z
Project Summary
Job Title: Lisbon Frazier Bridge
Analysis: Immediate, 1-year consolidation, and secondary consolidation
Author: Manahan/Check L Krusinski
Company: MaineDOT
Date Created: 8/13/2020 rev7 12/9/2020
Comments:
SVI load = 2750 psf (RN)
Delta q = 1781 psf at bottom slab ele. outside horizontal
limits of existing pipe arch
Model 2-foot crushed stone mat with geogrid



rocscience SETTLE3D 3.015	*Project* Lisbon Frazier Bridge		
	Analysis Description Immediate, 1-year consolidation, and secondary consolidation		
	Drawn By Manahan/Check L Krusinski	*Company* MaineDOT	
	Date 8/13/2020 rev7 12/9/2020	*File Name* 23118LisbonFrazierSettleR7.s3z	

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

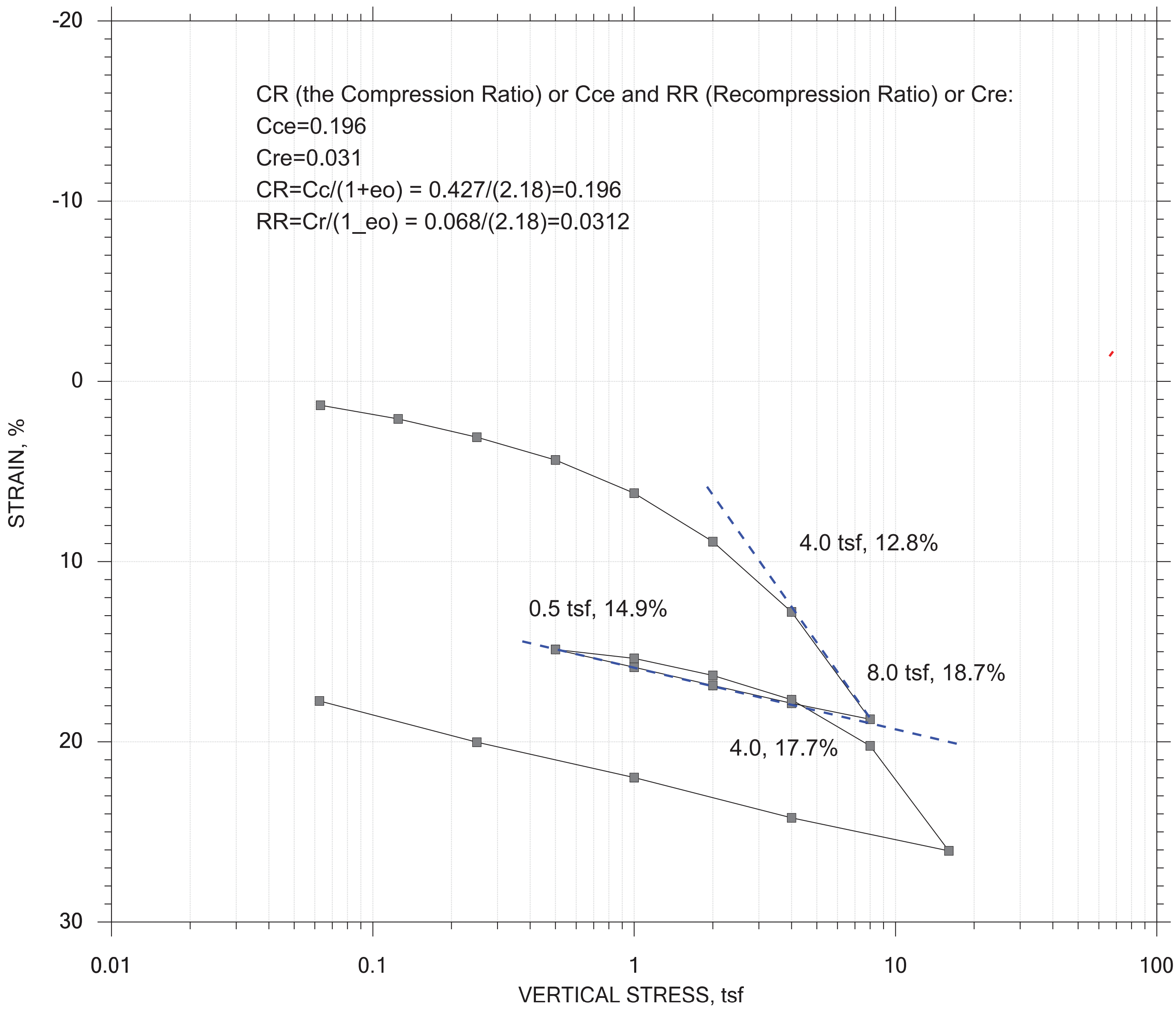


					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	40.87	30.40
Preconsolidation Stress: ---				Dry Unit Weight, pcf	77.059	92.379
Compression Ratio: ---				Saturation, %	93.22	100.00
Diameter: 2.495 in		Height: 0.9995 in		Void Ratio	1.18	0.82
LL: NP	PL: NP	PI: NP	GS: 2.69			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 3/3/2020	Test No.: 300266
	Depth: 45.0-47.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: --		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

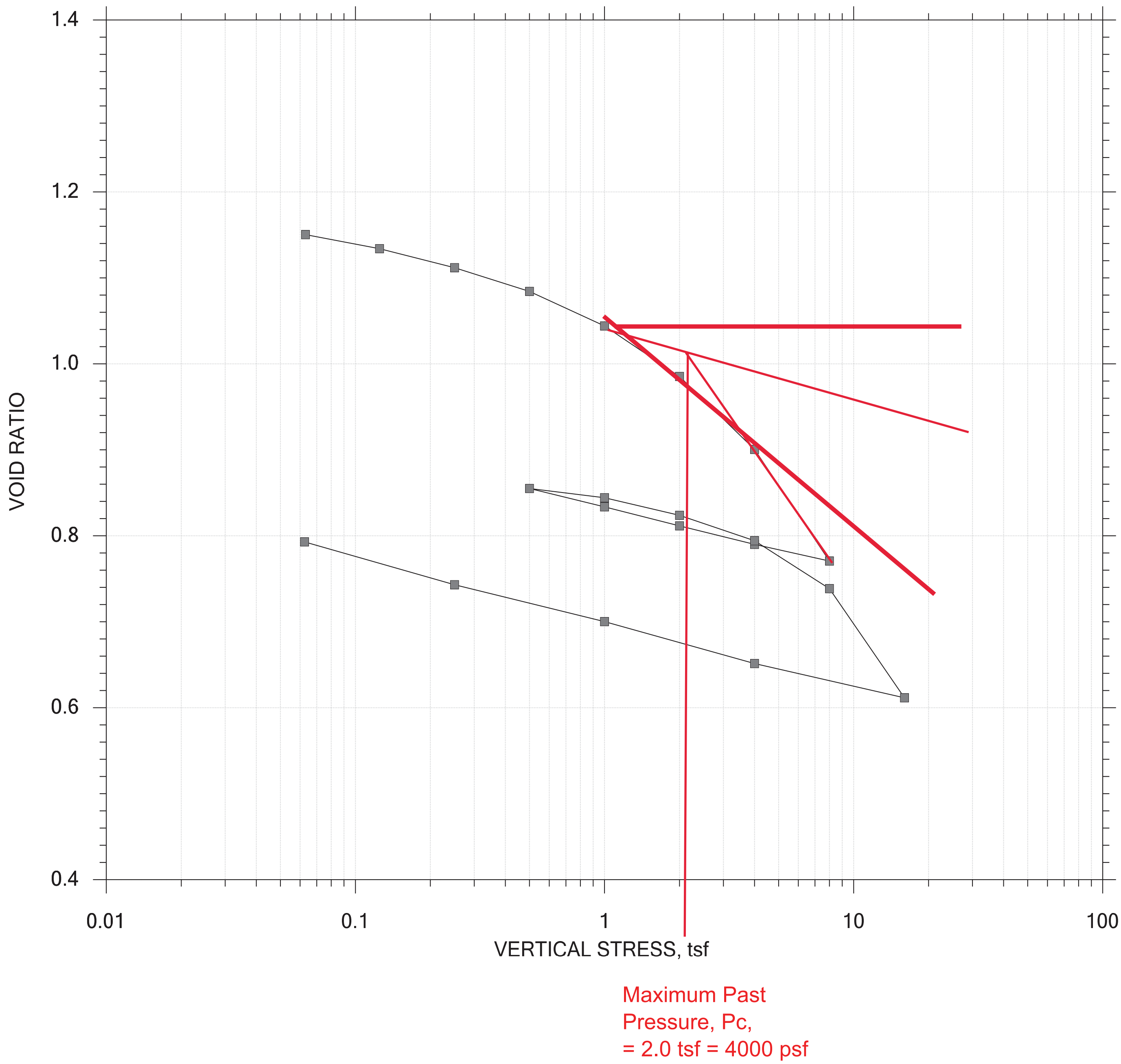


					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	40.87	30.40
Preconsolidation Stress: ---				Dry Unit Weight, pcf	77.059	92.379
Compression Ratio: ---				Saturation, %	93.22	100.00
Diameter: 2.495 in		Height: 0.9995 in		Void Ratio	1.18	0.82
LL: NP	PL: NP	PI: NP	GS: 2.69			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 3/3/2020	Test No.: 300266
	Depth: 45.0-47.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: --		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	40.87	30.40
Preconsolidation Stress: ---				Dry Unit Weight, pcf	77.059	92.379
Compression Ratio: ---				Saturation, %	93.22	100.00
Diameter: 2.495 in		Height: 0.9995 in		Void Ratio	1.18	0.82
LL: NP	PL: NP	PI: NP	GS: 2.69			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 1U	Test Date: 3/3/2020	Test No.: 300266
	Depth: 45.0-47.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: --		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon	Location: --	Project No.: 023118.00
Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
Sample No.: 1U	Test Date: 3/3/2020	Depth: 45.0-47.0 FT
Test No.: 300266	Sample Type: UNDISTURBED	Elevation: --

Soil Description: --
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Measured Specific Gravity: 2.69	Liquid Limit: NP	Specimen Diameter: 2.50 in
Initial Void Ratio: 1.18	Plastic Limit: NP	Initial Height: 1.00 in
Final Void Ratio: 0.818	Plasticity Index: NP	Final Height: 0.83 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	41	RING	RING+BASE	150
Wt. Container + Wet Soil, gm	80.320	401.29	390.95	200.40
Wt. Container + Dry Soil, gm	75.990	360.90	360.90	170.12
Wt. Container, gm	63.410	262.05	262.05	70.530
Wt. Dry Soil, gm	12.580	98.846	98.846	99.590
Water Content, %	34.42	40.87	30.40	30.40
Void Ratio	---	1.18	0.818	---
Degree of Saturation, %	---	93.22	100.00	---
Dry Unit Weight, pcf	---	77.059	92.379	---

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Lisbon
Boring No.: BB-LFB-103
Sample No.: 1U
Test No.: 300266

Location: --
Tested By: GSL
Test Date: 3/3/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 45.0-47.0 FT
Elevation: --

Soil Description: --
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

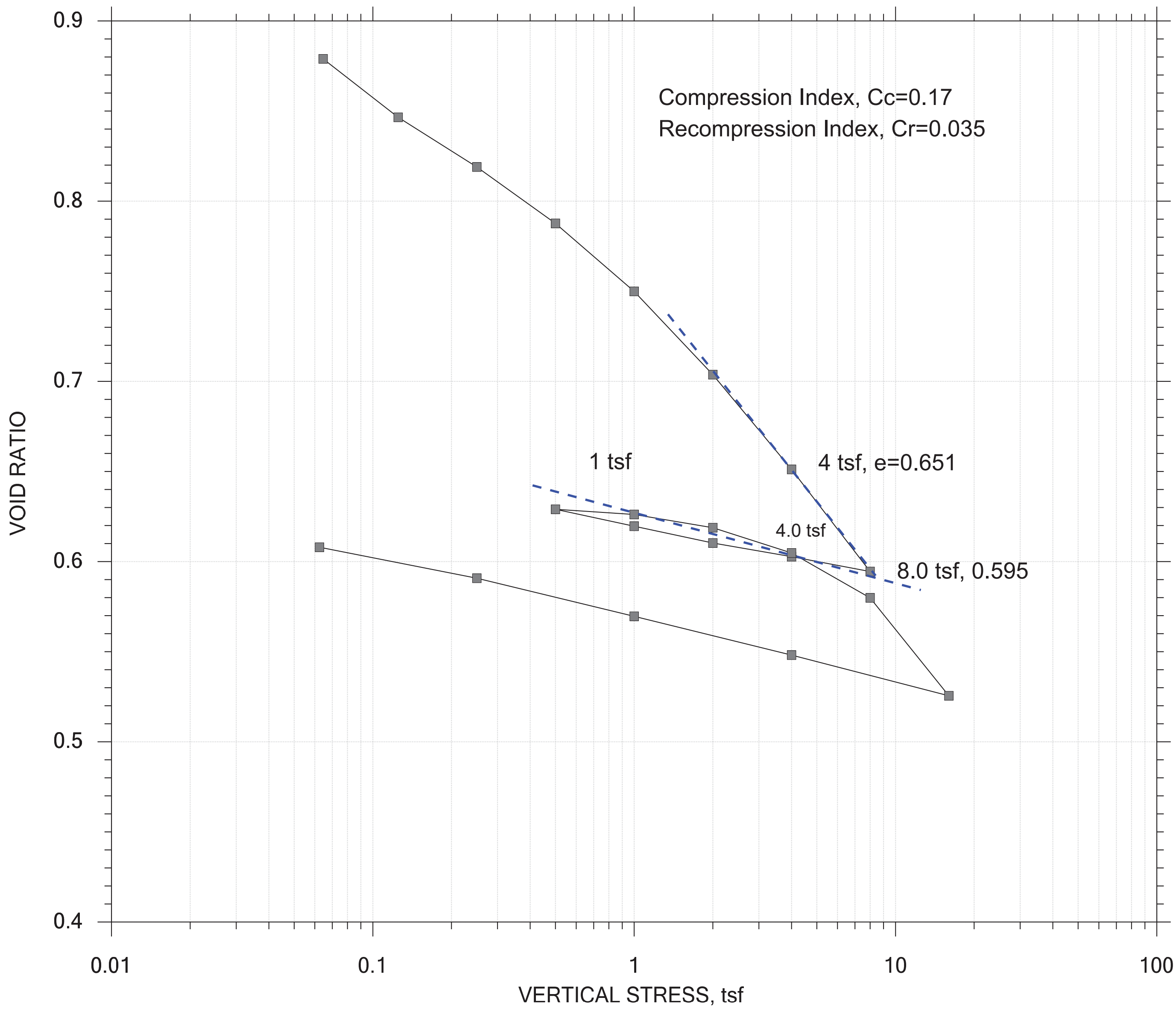
Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0630	0.01324	1.15	1.33	8.125	2.98e-006	2.10e-001	1.69e-003
2	0.125	0.02078	1.13	2.08	3.876	6.11e-006	1.22e-001	2.01e-003
3	0.250	0.03090	1.11	3.09	4.077	5.71e-006	8.10e-002	1.25e-003
4	0.500	0.04356	1.08	4.36	4.635	4.90e-006	5.06e-002	6.69e-004
5	1.00	0.06197	1.04	6.20	5.087	4.32e-006	3.68e-002	4.30e-004
6	2.00	0.08889	0.985	8.89	5.501	3.81e-006	2.69e-002	2.77e-004
7	4.00	0.1279	0.900	12.8	7.259	2.68e-006	1.95e-002	1.41e-004
8	8.00	0.1874	0.771	18.7	8.798	1.98e-006	1.49e-002	7.93e-005
9	4.00	0.1785	0.790	17.9	1.851	8.84e-006	2.22e-003	5.30e-005
10	2.00	0.1686	0.812	16.9	4.489	3.73e-006	4.93e-003	4.96e-005
11	1.00	0.1585	0.834	15.9	8.600	1.99e-006	1.01e-002	5.45e-005
12	0.500	0.1487	0.855	14.9	19.656	8.93e-007	1.96e-002	4.72e-005
13	1.00	0.1536	0.844	15.4	5.309	3.33e-006	9.80e-003	8.79e-005
14	2.00	0.1630	0.824	16.3	6.039	2.87e-006	9.43e-003	7.31e-005
15	4.00	0.1765	0.794	17.7	4.358	3.88e-006	6.76e-003	7.07e-005
16	8.00	0.2021	0.739	20.2	5.612	2.87e-006	6.40e-003	4.95e-005
17	16.0	0.2604	0.612	26.1	5.419	2.67e-006	7.29e-003	5.25e-005
18	4.00	0.2421	0.651	24.2	6.065	2.27e-006	1.53e-003	9.32e-006
19	1.00	0.2197	0.700	22.0	10.102	1.43e-006	7.45e-003	2.88e-005
20	0.250	0.2001	0.743	20.0	38.962	3.93e-007	2.62e-002	2.78e-005
21	0.0625	0.1772	0.793	17.7	198.778	8.12e-008	1.22e-001	2.67e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0630	0.01324	1.15	1.33	1.661	3.38e-006	2.10e-001	1.92e-003	0.00e+000
2	0.125	0.02078	1.13	2.08	0.603	9.13e-006	1.22e-001	3.00e-003	0.00e+000
3	0.250	0.03090	1.11	3.09	0.534	1.01e-005	8.10e-002	2.21e-003	0.00e+000
4	0.500	0.04356	1.08	4.36	0.510	1.04e-005	5.06e-002	1.41e-003	0.00e+000
5	1.00	0.06197	1.04	6.20	0.684	7.47e-006	3.68e-002	7.43e-004	0.00e+000
6	2.00	0.08889	0.985	8.89	1.489	3.27e-006	2.69e-002	2.37e-004	0.00e+000
7	4.00	0.1279	0.900	12.8	1.069	4.23e-006	1.95e-002	2.23e-004	0.00e+000
8	8.00	0.1874	0.771	18.7	0.000	0.00e+000	1.49e-002	0.00e+000	0.00e+000
9	4.00	0.1785	0.790	17.9	0.434	8.76e-006	2.22e-003	5.25e-005	0.00e+000
10	2.00	0.1686	0.812	16.9	1.147	3.39e-006	4.93e-003	4.51e-005	0.00e+000
11	1.00	0.1585	0.834	15.9	2.307	1.73e-006	1.01e-002	4.72e-005	0.00e+000
12	0.500	0.1487	0.855	14.9	6.711	6.08e-007	1.96e-002	3.21e-005	0.00e+000
13	1.00	0.1536	0.844	15.4	1.718	2.39e-006	9.80e-003	6.32e-005	0.00e+000
14	2.00	0.1630	0.824	16.3	2.022	1.99e-006	9.43e-003	5.07e-005	0.00e+000
15	4.00	0.1765	0.794	17.7	1.113	3.52e-006	6.76e-003	6.43e-005	0.00e+000
16	8.00	0.2021	0.739	20.2	1.759	2.13e-006	6.40e-003	3.67e-005	0.00e+000
17	16.0	0.2604	0.612	26.1	1.912	1.76e-006	7.29e-003	3.46e-005	0.00e+000
18	4.00	0.2421	0.651	24.2	1.111	2.87e-006	1.53e-003	1.18e-005	0.00e+000
19	1.00	0.2197	0.700	22.0	3.853	8.74e-007	7.45e-003	1.76e-005	0.00e+000
20	0.250	0.2001	0.743	20.0	0.000	0.00e+000	2.62e-002	0.00e+000	0.00e+000
21	0.0625	0.1772	0.793	17.7	0.000	0.00e+000	1.22e-001	0.00e+000	0.00e+000

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

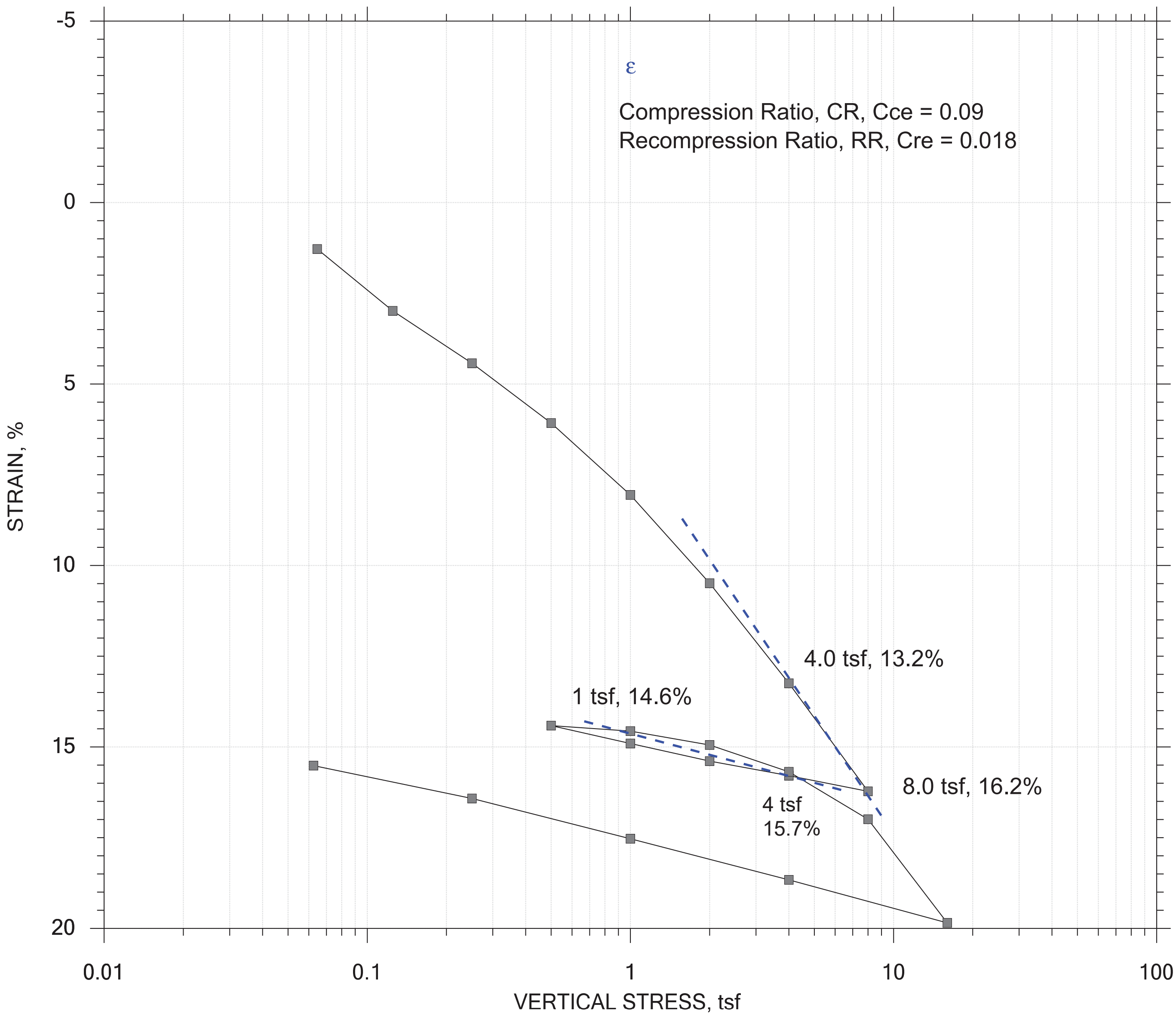


					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	34.99	23.09
Preconsolidation Stress: ---				Dry Unit Weight, pcf	88.56	103.83
Compression Ratio: ---				Saturation, %	104.57	100.00
Diameter: 2.495 in		Height: 0.9946 in		Void Ratio	0.90	0.62
LL: 27	PL: 21	PI: 6	GS: 2.70			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 2U	Test Date: 3/18/2020	Test No.: 300267
	Depth: 50.0-52.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: Grey SoftSensitive Clay		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	34.99	23.09
Preconsolidation Stress: ---				Dry Unit Weight, pcf	88.56	103.83
Compression Ratio: ---				Saturation, %	104.57	100.00
Diameter: 2.495 in		Height: 0.9946 in		Void Ratio	0.90	0.62
LL: 27	PL: 21	PI: 6	GS: 2.70			

	Project: Lisbon	Location: --	Project No.: 023118.00
	Boring No.: BB-LFB-103	Tested By: GSL	Checked By: --
	Sample No.: 2U	Test Date: 3/18/2020	Test No.: 300267
	Depth: 50.0-52.0 FT	Sample Type: UNDISTURBED	Elevation: --
	Description: Grey SoftSensitive Clay		
	Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test		
	Displacement at End of Increment		

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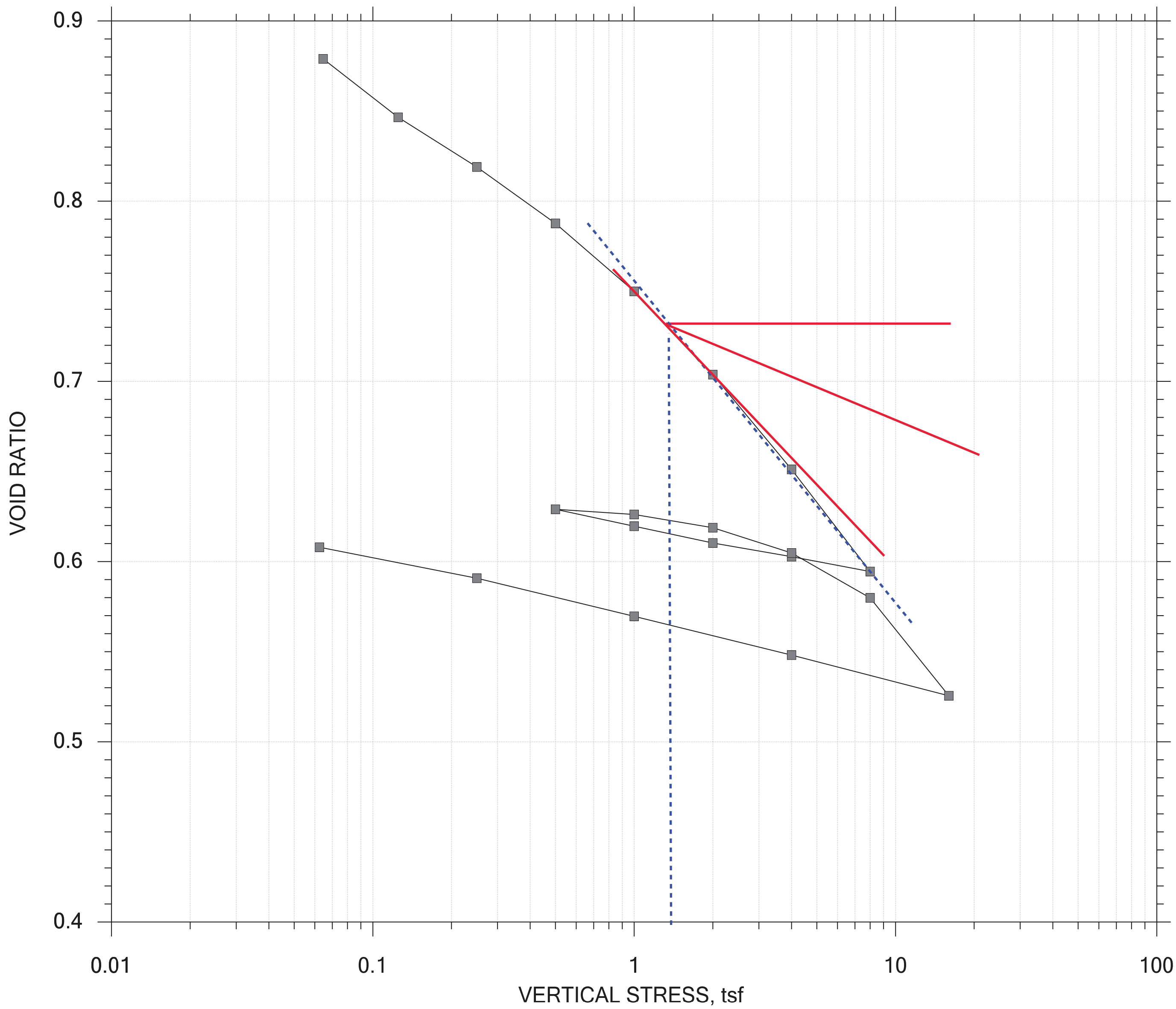
SUMMARY REPORT



Maximum Past Pressure, Pc,
1.3 tsf = 2,600 psf

					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	34.99	23.09
Preconsolidation Stress: ---				Dry Unit Weight, pcf	88.56	103.83
Compression Ratio: ---				Saturation, %	104.57	100.00
Diameter: 2.495 in		Height: 0.9946 in		Void Ratio	0.90	0.62
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Boring No.: BB-LFB-103
Sample No.: 2U
Test No.: 300267

Location: --
Tested By: GSL
Test Date: 3/18/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 50.0-52.0 FT
Elevation: --

Soil Description: Grey SoftSensitive Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

~~Estimated Specific Gravity~~: 2.70
$\boxed{\text{Initial Void Ratio: 0.903}}$
Final Void Ratio: 0.623

Liquid Limit: 27
Plastic Limit: 21
Plasticity Index: 6

Specimen Diameter: 2.50 in
Initial Height: 0.99 in
Final Height: 0.85 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	154	RING	RING+BASE	29
Wt. Container + Wet Soil, gm	115.74	414.63	401.18	202.13
Wt. Container + Dry Soil, gm	103.32	375.08	375.08	175.70
Wt. Container, gm	60.210	262.04	262.04	61.220
Wt. Dry Soil, gm	43.110	113.04	113.04	114.48
Water Content, %	28.81	34.99	23.09	23.09
Void Ratio	---	0.903	0.623	---
Degree of Saturation, %	---	104.57	100.00	---
Dry Unit Weight, pcf	---	88.560	103.83	---

One-Dimensional Consolidation by ASTM D2435 - Method B

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Boring No.: BB-LFB-103
Sample No.: 2U
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Location: --
Tested By: GSL
Test Date: 3/18/2020
Sample Type: UNDISTURBED

Project No.: 023118.00
Checked By: --
Depth: 50.0-52.0 FT
Elevation: --

Soil Description: Grey SoftSensitive Clay
Remarks: Maine Sensitive Load/Unload/Reload/Unload Consolidation Test

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0646	0.01271	0.879	1.28	46.528	5.15e-007	1.98e-001	2.75e-004
2	0.125	0.02967	0.847	2.98	75.359	3.09e-007	2.82e-001	2.35e-004
3	0.250	0.04404	0.819	4.43	24.853	9.06e-007	1.16e-001	2.82e-004
4	0.500	0.06041	0.788	6.07	16.498	1.32e-006	6.58e-002	2.34e-004
5	1.00	0.08011	0.750	8.05	7.761	2.70e-006	3.96e-002	2.89e-004
6	2.00	0.1043	0.704	10.5	5.775	3.46e-006	2.43e-002	2.27e-004
7	4.00	0.1317	0.651	13.2	5.001	3.77e-006	1.38e-002	1.40e-004
8	8.00	0.1614	0.595	16.2	4.271	4.13e-006	7.44e-003	8.29e-005
9	4.00	0.1571	0.603	15.8	0.855	2.00e-005	1.08e-003	5.85e-005
10	2.00	0.1531	0.610	15.4	0.949	1.82e-005	2.00e-003	9.84e-005
11	1.00	0.1482	0.620	14.9	2.568	6.80e-006	4.87e-003	8.94e-005
12	0.500	0.1433	0.629	14.4	4.397	4.02e-006	9.85e-003	1.07e-004
13	1.00	0.1448	0.626	14.6	4.985	3.56e-006	3.03e-003	2.91e-005
14	2.00	0.1487	0.619	14.9	1.687	1.05e-005	3.84e-003	1.08e-004
15	4.00	0.1560	0.605	15.7	1.443	1.21e-005	3.68e-003	1.20e-004
16	8.00	0.1690	0.580	17.0	1.891	8.99e-006	3.28e-003	7.94e-005
17	16.0	0.1974	0.526	19.8	2.564	6.30e-006	3.57e-003	6.06e-005
18	4.00	0.1856	0.548	18.7	0.527	3.00e-005	9.88e-004	8.00e-005
19	1.00	0.1743	0.570	17.5	1.676	9.72e-006	3.77e-003	9.88e-005
20	0.250	0.1633	0.591	16.4	9.035	1.85e-006	1.48e-002	7.39e-005
21	0.0625	0.1543	0.608	15.5	25.056	6.84e-007	4.82e-002	8.90e-005

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0646	0.01271	0.879	1.28	0.000	0.00e+000	1.98e-001	0.00e+000	0.00e+000
2	0.125	0.02967	0.847	2.98	0.000	0.00e+000	2.82e-001	0.00e+000	0.00e+000
3	0.250	0.04404	0.819	4.43	0.000	0.00e+000	1.16e-001	0.00e+000	0.00e+000
4	0.500	0.06041	0.788	6.07	4.556	1.11e-006	6.58e-002	1.97e-004	0.00e+000
5	1.00	0.08011	0.750	8.05	2.199	2.21e-006	3.96e-002	2.37e-004	0.00e+000
6	2.00	0.1043	0.704	10.5	1.915	2.42e-006	2.43e-002	1.59e-004	0.00e+000
7	4.00	0.1317	0.651	13.2	1.211	3.62e-006	1.38e-002	1.35e-004	0.00e+000
8	8.00	0.1614	0.595	16.2	1.149	3.57e-006	7.44e-003	7.16e-005	0.00e+000
9	4.00	0.1571	0.603	15.8	0.000	0.00e+000	1.08e-003	0.00e+000	0.00e+000
10	2.00	0.1531	0.610	15.4	0.000	0.00e+000	2.00e-003	0.00e+000	0.00e+000
11	1.00	0.1482	0.620	14.9	0.438	9.28e-006	4.87e-003	1.22e-004	0.00e+000
12	0.500	0.1433	0.629	14.4	1.012	4.06e-006	9.85e-003	1.08e-004	0.00e+000
13	1.00	0.1448	0.626	14.6	0.318	1.30e-005	3.03e-003	1.06e-004	0.00e+000
14	2.00	0.1487	0.619	14.9	0.000	0.00e+000	3.84e-003	0.00e+000	0.00e+000
15	4.00	0.1560	0.605	15.7	0.322	1.26e-005	3.68e-003	1.25e-004	0.00e+000
16	8.00	0.1690	0.580	17.0	0.441	8.95e-006	3.28e-003	7.91e-005	0.00e+000
17	16.0	0.1974	0.526	19.8	0.690	5.44e-006	3.57e-003	5.23e-005	0.00e+000
18	4.00	0.1856	0.548	18.7	0.000	0.00e+000	9.88e-004	0.00e+000	0.00e+000
19	1.00	0.1743	0.570	17.5	0.607	6.23e-006	3.77e-003	6.33e-005	0.00e+000
20	0.250	0.1633	0.591	16.4	2.219	1.75e-006	1.48e-002	6.99e-005	0.00e+000
21	0.0625	0.1543	0.608	15.5	6.497	6.13e-007	4.82e-002	7.97e-005	0.00e+000

Determination of Compression Index & Recompression Index for Clayey Silt Units and OCR

BB-LFB-103 Sample 1U

Determine insitu overburden stress

Sample depth $z := 45 \cdot ft$

Groundwater table $dw := 4.5 \cdot ft$

Initial void ratio $e_o := 1.18$

Effective overburden stress:

5.5 feet of granular fill over 4 feet of loose sandy silt (alluvium), then 15.5 feet of loose to medium dense sand, then 14.5 feet of loose silty sand, then 5 feet of soft clayey silt.

$\gamma_{fill} := 125 \cdot pcf$ $\quad \gamma_{sand} := 102 \cdot pcf$ $\quad \gamma_{silt} := 108 \cdot pcf$ $\quad \gamma_w := 62.4 \cdot pcf$

$$\sigma'_{vo} := 4.5 \cdot ft \cdot \gamma_{fill} + 1ft \cdot (\gamma_{fill} - \gamma_w) + 4 \cdot ft \cdot (\gamma_{silt} - \gamma_w) + 15.5 \cdot ft \cdot (\gamma_{sand} - \gamma_w) + (14.5 \cdot ft) \cdot (\gamma_{sand} - \gamma_w) + 5ft \cdot (\gamma_{silt} - \gamma_w)$$

$\sigma'_{vo} = 2.224 \cdot ksf$

Maximum past pressure from consolidation curve using Casagrande Construction (1936) - <u>this procedure is also applicable to ε vs log-p curves per Holtz & Kovacs pg. 295</u>

$\sigma'_{vm} := 4 \cdot ksf$

Overconsolidation ratio

$OCR := \frac{\sigma'_{vm}}{\sigma'_{vo}}$ $\quad OCR = 1.799$

This indicates the deposit is slightly preconsolidated
Use Shansep Method to backcalculate OCR in this clay silt deposit

Determine Compression Ratio = Modified Compression Index (Ccε) from lab consolidation Strain vs log-p curve

$s_1 := .128$ $\quad s_2 := 0.187$ $\quad p_1 := 4.0 \cdot 2 \cdot ksf$ $\quad p_2 := 8.0 \cdot 2 \cdot ksf$

$$C_{ce} := \frac{s_2 - s_1}{\log\left(\frac{p_2}{p_1}\right)}$$

Holtz & Kovacs Eq. 8-8

$C_{ce} = 0.196$ Use this value

Determine Compression Index using correlation

$$C_c := C_{ce} \cdot (1 + e_o)$$

Holtz & Kovacs Eq. 8-9

$$C_c = 0.427$$

Determine Recompression Ratio = Modified Recompression Index (Crε) from lab consolidation Strain (ε) vs log-p curve

$s_1 := .149$ $\quad s_2 := 0.177$ $\quad p_1 := .5 \cdot 2 \cdot ksf$ $\quad p_2 := 4.0 \cdot 2 \cdot ksf$

$$C_{re} := \frac{s_2 - s_1}{\log\left(\frac{p_2}{p_1}\right)}$$

$$C_{re} = 0.031$$

Determine Recompression Index using correlation

$$C_r := C_{re} \cdot (1 + e_o)$$

$$C_r = 0.068$$

Determine Compression Index (Cc) for lab Void Ratio, e vs log-p consolidation curve

$e_1 := 0.9$ $\quad e_2 := 0.771$ $\quad p_1 := 4.0 \cdot 2 \cdot ksf$ $\quad p_2 := 8.0 \cdot 2 \cdot ksf$

$$C_{c\_2} := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$$

$$C_{c\_2} = 0.429$$

Determine Compression Ratio = Modified Compression Index (Ccε)

$$Cce\_2 := \frac{C_{c\_2}}{1 + e_o}$$

$$Cce\_2 = 0.197$$

Determine Recompression Index (Cr) from Void Ratio (e) vs log-p curve

$e_{r1} := 0.855 \quad e_{r2} := 0.794 \quad p_{r1} := 0.5 \cdot 2 \cdot ksf \quad p_{r2} := 4.0 \cdot 2 \cdot ksf$

$$C_{r\_2} := \frac{e_{r1} - e_{r2}}{\log\left(\frac{p_{r2}}{p_{r1}}\right)}$$

$C_{r\_2} = 0.068$

Determine Recompression Ratio = Modified Recompression Index (Ccε)

$$Cre\_2 := \frac{C_{r\_2}}{1 + e_o}$$

$Cre\_2 = 0.031$

<u>Shansep Method to Backcalculate OCR - have vane tests in upper soft to medium stiff clayey silt in BB-LFB-101</u>

Range of undrained shear strengths above and below 1U

$$Su := \frac{893 + 625 + 915 + 982}{4} \cdot psf \qquad Su = 853.75 \cdot psf$$

Shansep Method - Reference Ladd (1991) for S and m variables:

Su / σ'vo = S x OCR^m

$S := 0.22$ for maine silt clays

$$m := 0.88 \cdot \left(1 - \frac{C_r}{C_c}\right) \qquad m = 0.741$$

$$OCR_{shan2} := \left(\frac{Su}{0.22 \cdot \sigma'_{vo}}\right)^{\frac{1}{m}} \qquad OCR_{shan2} = 2.121$$

OCR is slighly preconsolidated

Determination of Compression Index & Recompression Index for Clayey Silt Units and OCR

BB-LFB-103 Sample 2U

Determine insitu overburden stress

Sample depth $z := 50 \cdot ft$

Groundwater table $dw := 4.5 \cdot ft$

Initial void ratio $e_o := 0.90$

Effective overburden stress:

5.5 feet of granular fill over 4 feet of loose sandy silt (alluvium), then 15.5 feet of loose to medium dense sand, then 14.5 feet of loose silty sand, then 10 feet of soft clayey silt.

$\gamma_{fill} := 125 \cdot pcf$ $\gamma_{sand} := 102 \cdot pcf$ $\gamma_{silt} := 108 \cdot pcf$ $\gamma_w := 62.4 \cdot pcf$

$$\sigma'_{vo} := 4.5 \cdot ft \cdot \gamma_{fill} + 1ft \cdot (\gamma_{fill} - \gamma_w) + 4 \cdot ft \cdot (\gamma_{silt} - \gamma_w) + 15.5 \cdot ft \cdot (\gamma_{sand} - \gamma_w) + (14.5 \cdot ft) \cdot (\gamma_{sand} - \gamma_w) + 10ft \cdot (\gamma_{silt} - \gamma_w)$$

$$\sigma'_{vo} = 2.452 \cdot ksf$$

Maximum past pressure from consolidation curve using Casagrande Construction (1936) - <u>this procedure is also applicable to ε vs log-p curves per Holtz & Kovacs pg. 295</u>

$$\sigma'_{vm} := 2.6 \cdot ksf$$

Overconsolidation ratio

$$OCR := \frac{\sigma'_{vm}}{\sigma'_{vo}} \qquad OCR = 1.061$$

This indicates the deposit is normally consolidated
Use Shansep Method to backcalculate OCR in this clay silt deposit

Determine Compression Ratio = Modified Compression Index (Ccε) from lab consolidation Strain vs log-p curve

$s_1 := .105$ $s_2 := 0.132$ $p_1 := 2.0 \cdot 2 \cdot ksf$ $p_2 := 4.0 \cdot 2 \cdot ksf$

$$C_{ce} := \frac{s_2 - s_1}{\log\left(\frac{p_2}{p_1}\right)}$$

Holtz & Kovacs Eq. 8-8

$C_{ce} = 0.09$ Use this value

Determine Compression Index using correlation

$$C_c := C_{ce} \cdot (1 + e_o)$$ Holtz & Kovacs Eq. 8-9

$$C_c = 0.17$$

Determine Recompression Ratio = Modified Recompression Index (Crε) from lab consolidation Strain (ε) vs log-p curve

$s_1 := .146$ $s_2 := 0.157$ $p_1 := 1.0 \cdot 2 \cdot ksf$ $p_2 := 4.0 \cdot 2 \cdot ksf$

$$C_{re} := \frac{s_2 - s_1}{\log\left(\frac{p_2}{p_1}\right)}$$

$$C_{re} = 0.018$$

Determine Recompression Index using correlation

$$C_r := C_{re} \cdot (1 + e_o)$$

$$C_r = 0.035$$

Determine Compression Index (Cc) for lab Void Ratio, e vs log-p consolidation curve

$e_1 := 0.704$ $e_2 := 0.651$ $p_1 := 2.0 \cdot 2 \cdot ksf$ $p_2 := 4.0 \cdot 2 \cdot ksf$

$$C_{c\_2} := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$$

$$C_{c\_2} = 0.176$$

Determine Compression Ratio = Modified Compression Index (Ccε)

$$Cce\_2 := \frac{C_{c\_2}}{1 + e_o}$$

$$Cce\_2 = 0.093$$

Determine Recompression Index (Cr) from Void Ratio (e) vs log-p curve

$e_{r1} := 0.626 \quad e_{r2} := 0.605 \qquad p_{r1} := 1.0 \cdot 2 \cdot ksf \qquad p_{r2} := 4.0 \cdot 2 \cdot ksf$

$$C_{r\_2} := \frac{e_{r1} - e_{r2}}{\log\left(\frac{p_{r2}}{p_{r1}}\right)}$$

$C_{r\_2} = 0.035$

Determine Recompression Ratio = Modified Recompression Index (Ccε)

$$Cre\_2 := \frac{C_{r\_2}}{1 + e_o}$$

$Cre\_2 = 0.018$

<u>Shansep Method to Backcalculate OCR - have vane tests in upper soft to medium stiff clayey silt in BB-LFB-102</u>

Range of undrained shear strengths above and below 2U

$$Su := \frac{893 + 625 + 915 + 982}{4} \cdot psf \qquad Su = 853.75 \cdot psf$$

Shansep Method - Reference Ladd (1991) for S and m variables:

Su / σ'vo = S x OCR^m

$S := 0.22$ for maine silt clays

$$m := 0.88 \cdot \left(1 - \frac{C_r}{C_c}\right) \qquad m = 0.701$$

$$OCR_{shan2} := \left(\frac{Su}{0.22 \cdot \sigma'_{vo}}\right)^{\frac{1}{m}} \qquad OCR_{shan2} = 1.926$$

OCR is slighly preconsolidated

By: J. Manahan
Date: 11/17/2020
Checked by: LK 12/10/2020

	BB-LFB-101			BB-LFB-102		Sensitivity	
50							
51							
52							
53							
54							
55	893	112	8.0				Very Sensitive
56	625	89	7.0				Sensitive
57							
58							
59							
60	937	179	5.2				Sensitive
61	982	179	5.5				Sensitive
62							
63							
64							
65	915	179	5.1				Sensitive
66	1027	223	4.6				Sensitive

< 2 = insensitive
2 to 4 = moderately sensitive
4 to 8 = sensitive
8 to 16 = very sensitive
16 to 32 = slightly quick

By: J. Manahan
Date: 11/17/2020
Checked by: LK 12/10/2020

Lisbon Frazier Bridge Atterberg Data

Sample	Soil type	WC	LL	PL	PI	LI	
BB-LFB-101/7D	Silt	26.9	NP	NP	NP	NP	Non-plastic
BB-LFB-101/9D	Silt	26.6	NP	NP	NP	NP	Non-plastic
BB-LFB-101/10D	Clayey Silt	36	32	23	9	1.4	Potential to be Viscous
BB-LFB-101/12D	Silty Clay	33.3	34	22	12	0.9	Normally Consolidated
BB-LFB-102/8D	Silt	22.2	NP	NP	NP	NP	Non-plastic
BB-LFB-103/1U	Silt	31.2	NP	NP	NP	NP	Non-plastic
BB-LFB-103/2U	Silt, some clay	34.7	26	21	5	2.2	Potential to be Viscous