MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Construction of:

GRAY FARM BRIDGE
U.S. ROUTE 2/STATE ROUTE 27 OVER GRAY FARM STREAM
NEW SHARON, MAINE

Prepared by:

Brandon Slaven
Assistant Geotechnical Engineer

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Franklin County
WIN 19178.00

Soils Report 2018-34
Bridge No. 6486

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Table of Contents

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of a large culvert with proposed Gray Farm Bridge which carries U.S. Route 2 and State Route 27 over Gray Farm Stream in New Sharon, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design parameters, construction recommendations and considerations for the proposed bridge.

The existing culvert was constructed in 1955 and is an 88-foot-long, 6-foot diameter single corrugated metal pipe. The existing pipe is not a bridge now, therefore is not in the Maine Department of Transportation (MaineDOT) Bridge Maintenance inventory. The culvert has rusted and distorted over time. There is a large scour hole at the downstream end. The culvert is considered in poor condition and in need of replacement. There is a 5-foot corrugated metal overflow culvert located approximately 200 feet west on Route 2. This overflow pipe will be removed.

The proposed replacement structure will be a 15-foot span by 8-foot rise precast concrete box culvert. The concrete box culvert shall have 1-foot tall precast headwalls and 2-foot toe walls. The upstream and downstream ends of the culvert will be slope-tapered to match the 2H:1V (horizontal:vertical) sideslopes. The box culvert will be embedded approximately 3 feet into the streambed and 2 feet of special fill and rock bands 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 will be specified.

The box invert elevation will stay at or above Elev. 312 which will ensure minimum cover of the soft, weak marine clay deposit during construction. Maintaining the minimum cover over the clay layer will protect the stability of the clay. Disturbing the clay layer would lead to unstable soils during construction and should be avoided.

The proposed roadway alignment will match existing horizontal alignment. The vertical profile will also roughly match the existing vertical profile across the culvert. Staged construction will be utilized to allow the maintenance of one-way, alternating traffic over a 12-foot travel lane during construction. The major cost consideration for the proposed box alternative are construction phasing and temporary excavation support.

2.0 GEOLOGIC SETTING

The existing pipe carries U.S. Route 2 and State Route 27 over Gray Farm Stream in New Sharon, Maine approximately 1.5 miles west of the intersection of Route 134 with US Route 2 as shown on Sheet 1 – Location Map. Gray Farm Stream drains a portion of the Sandy River floodplain. The Sandy River is located approximately 750 feet southeasterly of Route 2.

The Maine Geological Survey (MGS) Surficial Geology Map of the Farmington

Quadrangle, Maine, Open-file No. 86-29 (1986), indicates the surficial soils in the vicinity of the bridge project consist of recent stream alluvium along the river channel surrounded by glacial outwash stream deposits over the floodplain. Stream alluvium generally consists of sands, gravel and silt. Glacial stream deposits consist of sand and gravel and were deposited by meltwater streams during the melting of the Late Wisconsinan Glacier.

According to the Bedrock Geology Map of the Farmington Quadrangle, MGS, Open File 78-16 (1977), bedrock in the project area is mapped as the Anasagunticook Member of the Sangerville Formation. The Anasagunticook Member is described as a thinly interbedded siltstone and pelite (siltstone) that has undergone moderate metamorphism due in part to the local granitic intrusion.

3.0 SUBSURFACE INVESTIGATION

The MaineDOT drilled two test borings, designated BB-NS-101 and BB-NS-102, at the project site. Boring BB-NS-101 was drilled in the eastbound travel lane on the south side of the existing pipe and was terminated in the very soft marine clay layer. Test boring BB-NS-102 was subsequently drilled diagonally across Route 2 at the existing pipe to characterize the thickness and engineering properties of the marine clay. Test boring BB-NS-102 penetrated the marine clay layer in its entirety and was terminated approximately 38 feet into glacial till. The boring locations are shown on Sheet 2 – Boring Location Plan. An interpretive subsurface profile across the site is shown on Sheet 3 – Interpretive Subsurface Profile.

Test boring BB-NS-101 was drilled on September 18, 2014. Test boring BB-NS-102 was drilled on December 22 through 30, 2014. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

Borings were performed by using a combination of solid stem auger and cased wash boring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The 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" on July 31, 2013 and October 16, 2014. All N-values discussed in this report are corrected values computed by applying the average energy transfer ratios of 0.867 and 0.908 to the raw field N-values in borings BB-NS-101 and BB-NS-102, respectively. The hammer efficiency factors (0.867 and 0.908) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Undisturbed tube samples were obtained in the soft soil deposits where possible. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata.

A geotechnical engineer logged the subsurface conditions encountered and selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified lab testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program and were later located by MaineDOT Region 3 survey.

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, seven grain size analyses with hydrometer and natural moisture content, and six Atterberg limits tests. Soil test results are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Granular (embankment) Fill, over silty fine sand (Recent Alluvium). The Recent Alluvium was underlain in by a thick deposit of very soft, sensitive silty clay (Marine Clay). The Marine Clay deposit was underlain by Glacial Till that was in excess of 38 feet in boring BB-NS-102. Boring BB-NS-102 was terminated at a depth of 130 feet below the ground surface (bgs) without encountering bedrock. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

5.1 Granular (Embankment) Fill

Encountered in both borings was a layer of Granular (Embankment) Fill. The combined thickness of the embankment Fill encountered ranged from approximately 16 to 20 feet at the boring locations. The Fill generally consisted of:

- Brown, moist to wet, fine to medium sand, with varying amounts of coarse sand, gravel and silt.

Corrected SPT N-values in the Fill ranged from 4 to 33 blows per foot (bpf) indicating the fill layer is loose to dense in consistency. Two grain size analyses resulted in material classifications of A-2-4 and A-3 under the AASHTO Soil Classification System and SM and SP-SM under the Unified Soil Classification System (USCS). The natural water content of the fill samples tested ranged from approximately 6 to 11 percent.

5.2 Recent Alluvium

Both borings encountered Recent Alluvium below the Embankment Fill. The Recent Alluvium was typically saturated, loose, silty fine to medium sand, with varying amounts of organics and wood. The thickness of the deposit encountered was approximately 9 feet at the boring locations. SPT N-values were 6 and 7 bpf in indicating that the deposit is loose in consistency.

5.3 Marine Clay Deposit

Both test borings encountered Marine Clay (Presumpscot Formation) below the Recent Alluvium. The Marine Clay deposit encountered was variable and generally consisted of:

- Grey, wet, silt, some silt, trace sand;
- Dark grey, wet, silt, some clay, trace to some sand, trace gravel
- Dark grey, clayey silt, trace sand

Boring BB-NS-101 was terminated 8.5 feet into the Marine Clay deposit. The bottom of the Marine Clay in boring BB-NS-102 was encountered at approximately 92 feet bgs, for a total thickness of 66.7 feet.

Two SPT N-values in the Marine Clay were weight-of-rod (WOR) indicating the deposit is very soft in consistency. The remainder of SPT tests were conducted though soils disturbed by in-situ vane shear tests and therefore are invalid.

Seven grain size analyses with hydrometer resulted in the glaciomarine deposit being classified as A-4 under the AASHTO Soil Classification System and ML, CL and ML-CL under the USCS.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the Marine Clay deposit. A 55 x 110 vane was used. Occasionally vane shear tests could not be completed because the vane was unable to be pushed by hand to test depth or the vane would not turn due to the presence of gravel; this is noted on the boring logs. Nine of fourteen successful vane shear tests conducted within the silty clay layers showed measured undisturbed undrained shear strengths ranging from approximately 313 psf to 447 psf, indicating that the Marine Clay deposit is primarily soft in consistency. The remaining five vane shear tests measured undrained shear strengths ranging from approximately 580 to 1049 psf, indicating those subunits are medium stiff to stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 22 to 89 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the silty clay has a sensitivity ranging from 7 to 26 and is classified as moderately sensitive to slightly quick.

Atterberg limits tests were conducted six samples of the Marine Clay deposit, and are summarized in Table 1:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-NS-101, 7D	SILT, some clay	26.3	28	27	1	-0.7
BB-NS-101, 8D	SILT, some clay	26.4	27	26	1	0.4
BB-NS-102, 4D	SILT, some clay	28.7	25	22	3	2.2
BB-NS-102, 5D	SILT, some clay	29.4	25	22	3	2.5
BB-NS-102, 6D	SILT, some clay	30.5	27	20	7	1.5
BB-NS-102, 7D	Clayey SILT	30.1	33	22	11	0.7

Table 1 – Summary of Atterberg Limits Test Results

The plasticity indices of the samples indicate that the soils are slightly plastic to having medium plasticity (Burmister, 1949). The natural water contents of the tested samples ranged from approximately 26 to 31 percent and liquid limits ranged from 25 to 33. The liquidity indices range from -0.7 to 2.5. Interpretation of these results indicates that the soils with liquidity indices of 1 or less are normally consolidated or 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”.

5.4 Glacial Till

A Glacial Till deposit was encountered beneath the Marine Clay deposit in boring BB-NS-102. The boring did not penetrate the entire thickness of the deposit. The thickness of the deposit encountered was approximately 38 feet. The Glacial Till generally consisted of:

- Dark grey, dense, fine sand, some silt, little gravel, trace clay;
- Gravel layers; and
- Cobbles.

One SPT N-value in the Glacial Till resulted in a corrected N-value of 42 bpf indicating this layer is dense. One grain size analysis conducted on a sample of the glacial till resulted in the sample being classified as A-4 under the AASHTO Soil Classification System and SC-SM under the USCS. The moisture content of the tested sample was approximately 11 percent.

Boring BB-NS-102 was terminated in the Glacial Till at a depth of 130 feet bgs, without encountering bedrock.

5.5 Groundwater

Groundwater measurements performed at the completion of the borings indicated groundwater levels ranging between 13.5 and 15 feet bgs. Groundwater observations are provided on the boring logs in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. Note that because water was introduced into the borehole during drilling operations, the groundwater measurements may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report[1] (PDR) evaluated a 1.2 bankfull width, 15-foot wide by 8-foot tall precast concrete box culvert and a 75-foot span, pile-supported integral abutment bridge as potential replacement structures. To provide fish passage, both options included filling the scour pool downstream and additional measures to mitigate the streambed headcut.

It was critical that the invert elevation and bedding of the precast box alternative be above Elev. 309 to provide 3 to 6 feet of separation between the bottom of the excavation for the box culvert and the underlying Marine Clay. During PDR development, disturbing the sensitive clay layer was strongly advised against as it would leave to unstable soils, constructability issues and long-term performance issues. While the pile-supported bridge option would reduce the depth of excavation required and reduce the stability risks associated with the sensitive marine clay layer, the precast concrete box culvert was the preferred alternative because its overall lower cost.

The selected alternative is, therefore, a 1.2 bankfull width, precast concrete box culvert. The precast box culvert will be constructed with a minimum of 3 feet of separation between the box culvert bedding material and the top of the Marine Clay. To improve fish passage, the box will have 2 feet of riprap, special fill and rock bands. A portion of the stream upstream and downstream will be reconstructed with special fill and rock bands. Riprap will be placed on the sideslopes of the reconstructed stream channel. The project will be constructed using staged construction. The overflow culvert located 200 feet to the west on Route 2 will be filled with flowable fill.

[1] Preliminary Design Report, Gray Farm Bridge #6486 over Gray Farm Stream, New Sharon, Maine, April 16, 2018.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 15-foot span by 8-foot high precast concrete box culvert with slope-tapered inlet and outlet walls. The box culvert will have 1 foot tall precast headwalls. To prevent undermining, the box culvert will have inlet and outlet toe walls.

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 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 C Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor (method of compaction approximating 97 percent of AASHTO T-108 maximum dry density).

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

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2018 (LRFD).

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

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1 foot by 1 foot concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

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

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

7.1.3 Precast Concrete Toe Walls

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

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 2-foot-thick layer of crushed stone that is reinforced with geogrid and wrapped in stabilization/reinforcement geogrid place on the native soil subgrade. The bearing elevation of the crushed stone mat will be approximately elevation 312 at the inlet and Elevation 309 at the outlet. The subgrade soils at this elevation are expected to be loose, saturated silty sands (Recent Alluvium). The Recent Alluvium is underlain by soft, sensitive, Marine Clay. It is anticipated that the precast concrete box culvert will be adequately supported by the Recent Alluvium and underlying Marine Clay, but will require a bearing pad consisting of a 2-foot thick crushed stone mat wrapped in a heavy non-woven geotextile.

For a precast concrete box culvert with a base width of 17 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 1 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 2 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as 0.3*f'c*. See Appendix C – Calculations for supporting calculations.

7.1.5 Modulus of Subgrade Reaction

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

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 30 pounds per cubic inch (pci). See Appendix C – Calculations for supporting calculations.

7.2 Settlement

Constructing the proposed precast box culvert will result in a net reduction in stress at the bearing elevation due to the increase in the culvert opening and distribution of the imposed loads over the large base slab of the box. The project calls for the vertical alignment over the new structure to match the existing. As such, post-construction consolidation settlement of the precast concrete box culvert should be negligible with proper subgrade preparation.

7.3 Subgrade Excavation and Subgrade Preparation

The silty fine sand below the groundwater level is saturated and loose and will easily become disturbed by construction activities. Furthermore, as great a thickness as possible of the silty fine sand (Recent Alluvium) needs to remain in place and undisturbed between the box culvert bearing pad and the underlying Marine Clay. The excavation will require care to maintain bottom stability and bearing capacity. The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to allow the bearing pad construction in the dry;
- Use a smooth-edged bucket to avoid disturbance of the subgrade;
- Careful grade control will be necessary to avoid over excavation;
- Maintain as much of the Recent Alluvium over the Marine Clay as possible;
- Construct the box on a 2-foot thick layer of crushed stone layer reinforced with geogrid and wrapped in a heavy non-woven stabilization/ reinforcement geotextile.
- Hand-deploy the geotextile 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 C Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor.

7.4 Frost Protection

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, New Sharon has a design freezing index (DFI) of approximately 1750 F-degree days. The anticipated coarse grained alluvial soils were assigned a water content of 20%. These components correlate to a frost depth of 6.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Madison, Maine has an air DFI from the Modberg database of approximately 1847 F-degree days. Madison was selected because it lies near the same isoline as New Sharon and New Sharon is not available in the Modberg database. A water content of 20% was assumed. These components correlate to a frost depth of approximately 6.9 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.1 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.5 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and wingwalls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1 foot below the maximum depth of scour. Inlet and outlet toe walls will be protected by streambed armoring which will consist of 2 feet of plain riprap overlain by special fill.

The PDR states that in addition to reconstructing the streambed with 2-feet of plain riprap, the sideslopes be armored with a 3-foot-thick layer of riprap. Riprap shall conform to MaineDOT Standard Specification 703.26 – Plain 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 top of the riprap toe sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 2H:1V extending from the edge of the roadway down to the existing ground surface.

7.6 Seismic Design Considerations

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

7.7 Construction Considerations

The box culvert will be constructed on a 2-foot thick layer of crushed stone reinforced with geogrid 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 excavation for the box culvert and its bearing pad shall stay above Elev. 311 (at the inlet) and Elev. 309 (at the outlet) to ensure minimum cover of the soft Marine Clay deposit during construction. Maintaining a minimum thickness of the Recent Alluvium over the clay layer will protect the stability of the clay. Disturbing the clay layer may lead to unstable soils during construction and should be avoided.

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

- Construction phase dewatering to limit disturbance of the Recent Alluvium and limit the risk of excavation bottom heave, is recommended to allow the bearing pad construction in the dry. Cofferdams may be required to divert flow away from the new culvert location during construction.
- As great a thickness as possible of the silty fine sand (Recent Alluvium) needs to remain in place and undisturbed between the box culvert bedding pad and the underlying Marine Clay.
- The contractor shall not operate equipment over the excavated subgrade to minimize subgrade disturbance.
- Use a smooth-edged bucket to avoid disturbance of the subgrade.
- Careful grade control will be necessary to avoid over excavation.
- The stabilization/reinforcement geotextile shall be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat.

The saturated loose sands have potential to "flow" when disturbed. The Marine Clay is soft and sensitive and will experience strength loss and become viscous when disturbed. It is possible there may be subsidence associated with the withdrawal of the temporary sheeting and shoring required for staged construction. This may require final shimming and repair of each phase of the roadway.

The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The precast concrete box culvert shall be installed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification Section 534.

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

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

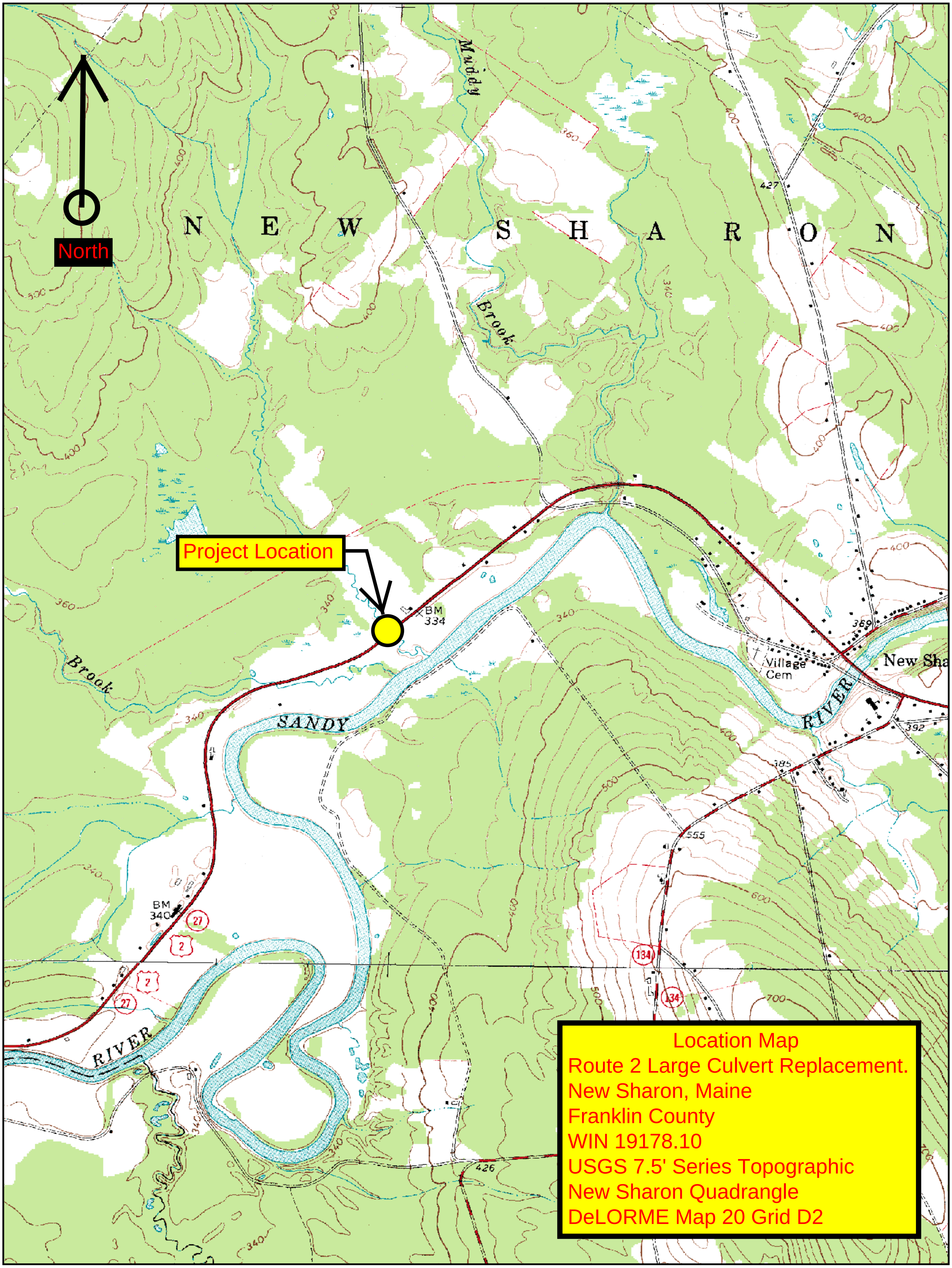
8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application to the proposed construction of Gray Farm Bridge in New Sharon, 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 in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Sheets



Map Scale 1:24000

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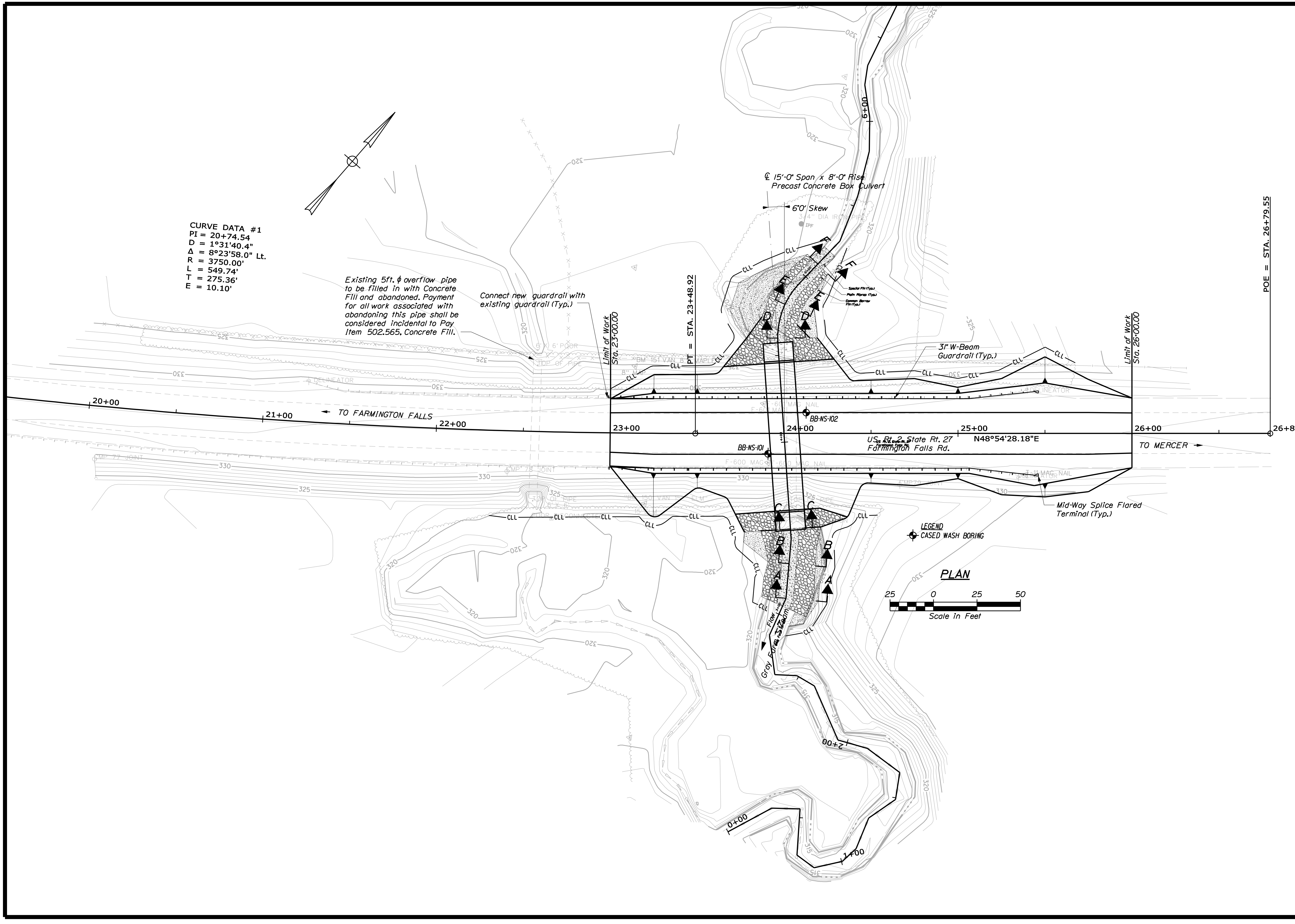


STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

19178.10

BRIDGE NO. 6486 | WIN 19178.10 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. WIGHT	
DESIGN-DETAILED	A. SHKARA	A. PARADIS	SEP 2018
CHECKED-REVIEWED	A. SHKARA	T. WHITE	SEP 2018
DESIGN2-DETAILED2	K.KRUSINSKI	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

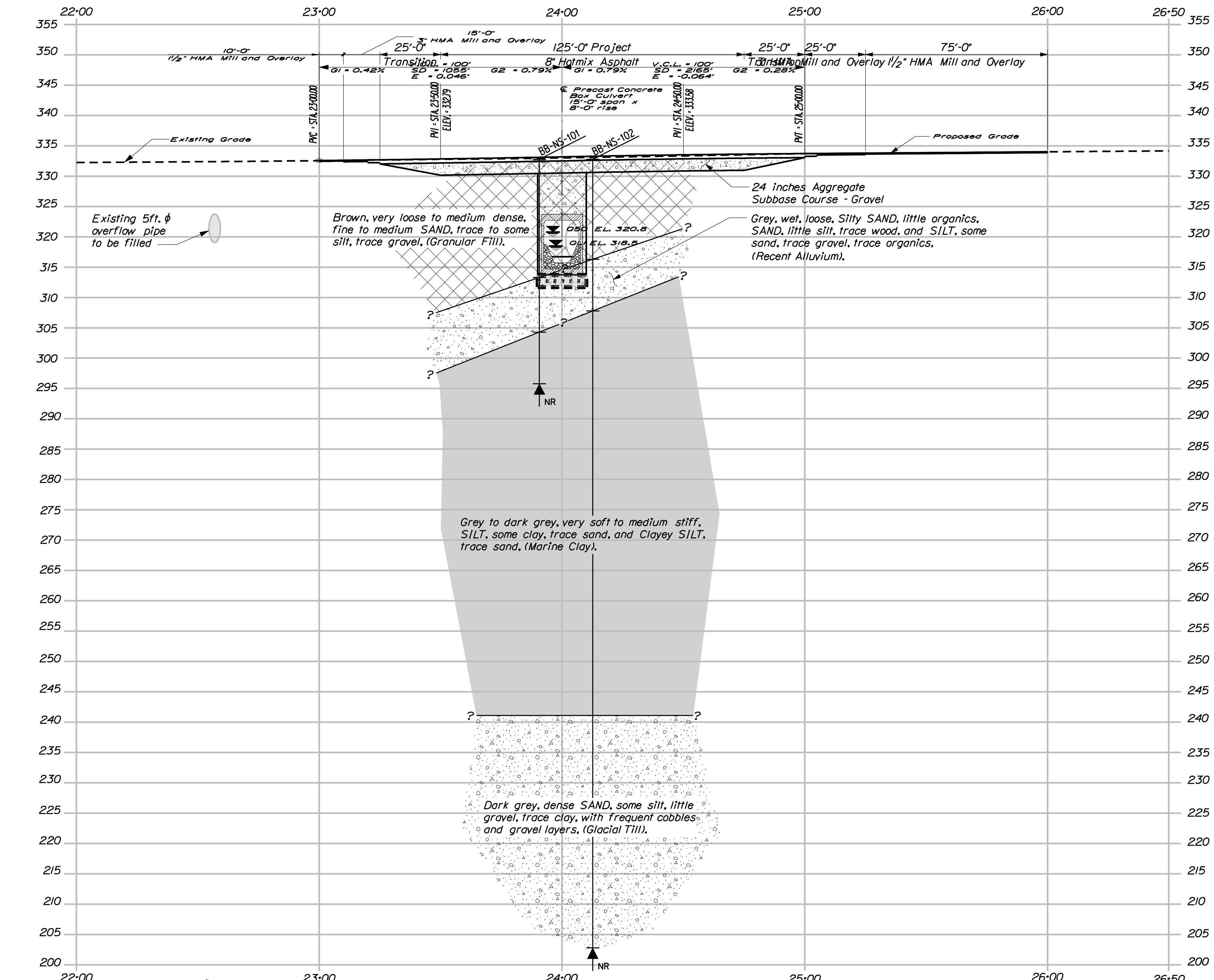
GRAY FARM BRIDGE
GRAY FARM STREAM
NEW SHARON FRANKLIN COUNTY

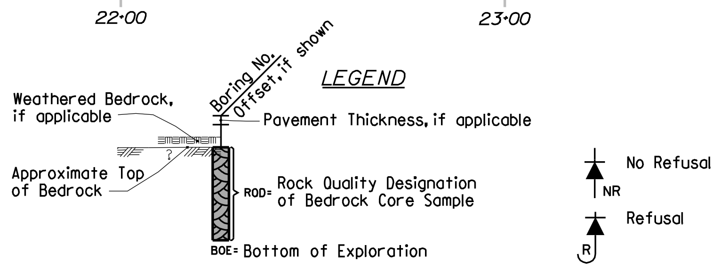
BORING LOCATION PLAN

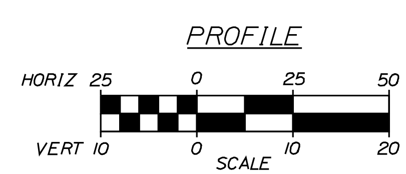
SHEET NUMBER

2

OF 4







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

19178.10

WIN 19178.10

BRIDGE PLANS

BRIDGE NO. 6486

PROJ. MANAGER	M. WIGHT	BY	DATE
DESIGN-DETAILED	A. SHKARA	A. PARADIS	SEP 2018
CHECKED-REVIEWED			
DESIGN2-DETAILED2	L.KRUSINSKI	T.WHITE	SEP 2018
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GRAY FARM BRIDGE
GRAY FARM STREAM
NEW SHARON FRANKLIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\10\geotech\msta\006_ISP1.dgn Division: GEOTECH Username: Terry.White Date:9/29/2018

Date:9/29/2018 Username: Terry.White Division: GEOTECH Filename: ...\msta\007_BORING LOGS1.dgn

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

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-101
WIN: 19178.10

Driller:	MaineDOT	Elevation (ft.)	332.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Daggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/18/2014; 07:30-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	23+90.7, 11.7 ft Rt.	Casing ID/OD:	HW	Water Level*:	15.0 ft bgs.
Hammer Efficiency Factor: 0.867		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	332.3	5.5" PAVEMENT. —0.5	
									Brown, fine to medium SAND, trace coarse sand, trace gravel, trace silt, (Fill).	
								330.8	—2.0	
	1D	24/20	2.00 - 4.00	10/11/12/13	23	33			Brown, damp, dense, fine to medium SAND, trace gravel, trace silt, (Fill).	
								328.8	—4.0	
5	2D	24/22	5.00 - 7.00	4/8/13/19	21	30			Brown, damp, medium dense, fine to medium SAND, some silt, (Fill).	G#243022 A-2-4, SM WC=11.1%
								324.3	—8.5	
10	3D	24/24	10.00 - 12.00	4/6/6/5	12	17			Brown, damp, medium dense, fine to medium SAND, trace silt, (Fill).	G#243023 A-3, SP-SM WC=6.1%
15	4D	24/18	15.00 - 17.00	1/1/2/2	3	4	16		Similar to above, except very loose, wet, (Fill).	
							26			
							42			
							35			
							43	313.3	—19.5	
20	5D	24/17	20.00 - 22.00	3/2/2/2	4	6	37		Dark grey, wet, loose, Silty, fine to medium SAND, little organics, trace wood, (Recent Alluvium).	
							41			
							57			
							63	308.8	—24.0	
							69			
25	6D	24/18	25.00 - 27.00	3/3/2/3	5	7	OPEN HOLE		Grey, wet, loose, fine to coarse SAND, little silt, trace wood, (Recent Alluvium).	
								304.3	—28.5	
30	7D	24/20	30.00 - 32.00	WOH/WOR/WOR/WOR	---				Grey, wet, very soft, SILT, some clay, trace fine sand, (Marine Clay).	G#243024 A-4, ML WC=26.3% LL=28 PL=27 PI=1
35	8D	24/22	35.00 - 37.00	WOR/WOR/WOR/WOR	---				Grey, wet, very soft, SILT, some clay, trace sand, (Marine Clay).	G#243025 A-4, ML WC=26.4% LL=27 PL=26 PI=1
								295.8	—37.0 Bottom of Exploration at 37.0 feet below ground surface. No refusal encountered.	
40										
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-NS-101

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

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102
WIN: 19178.10

Driller:	MaineDOT	Elevation (ft.)	332.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	13.5 ft bgs.
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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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		Auger cuttings 0 to 15 ft.: Brown, fine to medium SAND, trace to little silt.	
5										
10										
15	1D	24/12	15.00 - 17.00	1/2/4/2	6	9	26		Brown, wet, loose, fine to medium SAND, trace to little silt, trace coarse sand, (Fill).	
							45	316.3	—16.5 Changing at 16.5 ft to 1D: Grey, wet, SILT, some sand, trace gravel, trace organics; strong organic odor, fine laminations, (Recent Alluvium).	G#175420 A-4, ML WC=22.9%
							38			
							43			
							40			
20							66			
							58			
							68		Wood in wash water at 22.5 ft bgs.	
							71			
							73			
25	2D	24/3	25.00 - 27.00	4/5/2/2	7	11	69	307.8	—25.0 Dark grey, saturated, medium stiff, SILT, some clay, (Marine Clay) Significant fine to coarse sand wash in top of spoon that may have affected blow counts.	
							60			
							54			
							56			
							54			
30	3D V1		30.00 - 32.00 30.63 - 31.00	vane interval Su=580/22 psf			52		Dark grey, soft to medium stiff, SILT, some clay, (Marine Clay). 55x110 mm vane raw torque readings: V1: 13.0/0.5 ft-lbs V2: 8.0/1.0 ft-lbs	
	V2		31.63 - 32.00	Su=357/45 psf			59			
							56			
							56			
							56			
35	4D V3		35.00 - 37.00 35.63 - 36.00	vane interval Su=335/45 psf			53		Dark grey, soft, SILT, some clay, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V3: 7.5/1.0 ft-lbs V4: 8.0/0.5 ft-lbs	G#175421 A-4, ML WC=28.7% LL=25 PL=22 PI=3
	V4		36.63 - 37.00	Su=357/22 psf			51			
							53			
							53			
							56			
40	5D/MU		40.00 - 42.00	tube interval			OPEN HOLE		Dark grey, SILT, some clay, trace sand, trace gravel, (Marine Clay) Failed tube attempt.	G#175422 A-4, ML WC=29.4% LL=25 PL=22 PI=3
45	6D V5		45.00 - 47.00 45.63 - 46.00	Su=313/45 psf					Dark grey, soft, SILT, some clay, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V5: 7.0/1.0 ft-lbs V6: 7.0/1.0 ft-lbs	G#175423 A-4, CL-ML WC=30.5% LL=27 PL=20 PI=7
	V6		46.63 - 47.00	Su=313/45 psf						
50	1U	24/24	50.00 - 52.00						Dark grey, soft, Clayey SILT, trace sand (Marine Clay).	
	V7		52.63 - 53.00	Su=313/22 psf					55x110 mm vane raw torque readings: V7: 7.0/0.5 ft-lbs V8: 10.0/1.0 ft-lbs	
	V8		53.63 - 54.00	Su=447/45 psf						
55										
60	V9		60.63 - 61.00	Su=447/22 psf					55x110 mm vane raw torque readings: V9: 10.0/0.5 ft-lbs V10: 10.0/1.0 ft-lbs	
	V10		61.63 - 62.00	Su=447/45 psf						
							WOC			
65										
70	7D V11		70.00 - 72.00 70.63 - 71.00	Su=647/89 psf					Dark grey, medium stiff, Clayey SILT, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V11: 14.5/2.0 ft-lbs V12: 15.5/2.0 ft-lbs	G#175424 A-6, CL WC=30.1% LL=33 PL=22 PI=11
	V12		71.63 - 72.00	Su=692/89 psf						
75										

Remarks:

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Page 1 of 2
Boring No.: BB-NS-102

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

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102
WIN: 19178.10

Driller:	MaineDOT	Elevation (ft.)	332.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	13.5 ft bgs.
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
75										
80	V13		80.63 - 81.00	Su=848/67 psf					55x110 mm vane raw torque readings: V13: 19.0/1.5 ft-lbs V14: 23.5/1.0 ft-lbs	
	V14		81.63 - 82.00	Su=1049/45 psf			HYD			
							PUSH			
85										
90										
							a12	241.1	—91.7 NW casing fetches up, probable top of TILL. a12 blows for 0.3 ft.	
							57			
							75			
							93			
95	8D	24/10	95.00 - 97.00	14/15/13/21	28	42	ROLL CONE		Dark grey, dense, fine SAND, some silt, little rounded gravel, trace clay, (Glacial Till).	G#175425 A-4, SC-SM WC=11.0%
							AHEAD			
100										
									Roller Cone refusal at 103.7 FT BGS. Casing refusal at 103.8 ft bgs. Broke through cobble at approx. 104.1 ft bgs.	
105										
110									Wash: fine sand (Glacial Till). Drilling behavior: occasional gravel seams from 104 to 116 ft bgs.	
115									NW refusal, possible top of rock. Broke through at 116.4 ft bgs.	
120									Drilling behavior: cobbles and gravel layers more common with depth, typically 2-4" thick.	
125									Boney Glacial Till: frequent gravel and cobbles.	
130								202.8	—130.0 Bottom of Exploration at 130.0 feet below ground surface. No refusal encountered.	
135										
140										
145										
150										

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 2 of 2
Boring No.: BB-NS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

19178.10

WIN 19178.10
BRIDGE PLANS
BRIDGE NO. 6486

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	A. SHKARA	A. PARADIS	SEP 2018
CHECKED-REVIEWED	L.KRUSINSKI	T.WHITE	SEP 2018
DESIGN2-DETAILED2			
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REVISIONS 1			
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SIGNATURE
P.E. NUMBER
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GRAY FARM BRIDGE
GRAY FARM STREAM
NEW SHARON FRANKLIN 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

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):

RQD (%) = sum of the lengths of intact pieces of core* > 4 inches / length of core advance

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

Correlation of RQD to Rock Mass Quality

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	332.3		5½" PAVEMENT. 0.5	
										Brown, fine to medium SAND, trace coarse sand, trace gravel, trace silt, (Fill).	
	1D	24/20	2.00 - 4.00	10/11/12/13	23	33				Brown, damp, dense, fine to medium SAND, trace gravel, trace silt, (Fill).	
5	2D	24/22	5.00 - 7.00	4/8/13/19	21	30				Brown, damp, medium dense, fine to medium SAND, some silt, (Fill).	G#243022 A-2-4, SM WC=11.1%
10	3D	24/24	10.00 - 12.00	4/6/6/5	12	17				Brown, damp, medium dense, fine to medium SAND, trace silt, (Fill).	G#243023 A-3, SP-SM WC=6.1%
15	4D	24/18	15.00 - 17.00	1/1/2/2	3	4	16			Similar to above, except very loose, wet. (Fill).	
							26				
							42				
							35				
							43	313.3		19.5	
20	5D	24/17	20.00 - 22.00	3/2/2/2	4	6	37			Dark grey, wet, loose, Silty, fine to medium SAND, little organics, trace wood, (Recent Alluvium).	
							41				
							57				
							63				
25							69				

Remarks:

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Boring No.: BB-NS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gray Farm Bridge #6486 carries Routes 2/ 27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-101
WIN: 19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/18/2014; 07:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	23+90.7, 11.7 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	15.0 ft bgs.
Hammer Efficiency Factor:	0.867	**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	6D	24/18	25.00 - 27.00	3/3/2/3	5	7	OPEN HOLE			Grey, wet, loose, fine to coarse SAND, little silt, trace wood, (Recent Alluvium).	
								304.3		28.5	
30	7D	24/20	30.00 - 32.00	WOH/WOR/WOR/ WOR	---					Grey, wet, very soft, SILT, some clay, trace fine sand, (Marine Clay).	G#243024 A-4, ML WC=26.3% LL=28 PL=27 PI=1
35	8D	24/22	35.00 - 37.00	WOR/WOR/WOR/ WOR	---					Grey, wet, very soft, SILT, some clay, trace sand, (Marine Clay).	G#243025 A-4, ML WC=26.4% LL=27 PL=26 PI=1
								295.8		37.0 **Bottom of Exploration at 37.0 feet below ground surface.** No refusal encountered.	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gray Farm Bridge #6486 carries Routes 2/ 27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102
WIN: 19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Auger cuttings 0 to 15 ft.: Brown, fine to medium SAND, trace to little silt.	
5											
10											
15	1D	24/12	15.00 - 17.00	1/2/4/2	6	9	26			Brown, wet, loose, fine to medium SAND, trace to little silt, trace coarse sand, (Fill).	
							45	316.3		16.5 Changing at 16.5 ft to 1D: Grey, wet, SILT, some sand, trace gravel, trace organics; strong organic odor, fine laminations, (Recent Alluvium).	G#175420 A-4, ML WC=22.9%
							38				
							43				
							40				
20							66				
							58				
							68			Wood in wash water at 22.5 ft bgs.	
							71				
							73				
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102
WIN: 19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
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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.
25	2D	24/3	25.00 - 27.00	4/5/2/2	7	11	69	307.8		25.0 Dark grey, saturated, medium stiff, SILT, some clay, (Marine Clay) Significant fine to coarse sand wash in top of spoon that may have affected blow counts.	
							60				
							54				
							56				
							54				
30	3D		30.00 - 32.00	vane interval			52			Dark grey, soft to medium stiff, SILT, some clay, (Marine Clay). 55x110 mm vane raw torque readings: V1: 13.0/0.5 ft-lbs V2: 8.0/1.0 ft-lbs	
	V1		30.63 - 31.00	Su=580/22 psf							
	V2		31.63 - 32.00	Su=357/45 psf			59				
							56				
							56				
							56				
35	4D		35.00 - 37.00	vane interval			53			Dark grey, soft, SILT, some clay, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V3: 7.5/1.0 ft-lbs V4: 8.0/0.5 ft-lbs	G#175421 A-4, ML WC=28.7% LL=25 PL=22 PI=3
	V3		35.63 - 36.00	Su=335/45 psf							
	V4		36.63 - 37.00	Su=357/22 psf			51				
							53				
							53				
							56				
40	5D/MU		40.00 - 42.00	tube interval			OPEN HOLE			Dark grey, SILT, some clay, trace sand, trace gravel, (Marine Clay) Failed tube attempt.	G#175422 A-4, ML WC=29.4% LL=25 PL=22 PI=3
45	6D		45.00 - 47.00	Su=313/45 psf						Dark grey, soft, SILT, some clay, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V5: 7.0/1.0 ft-lbs V6: 7.0/1.0 ft-lbs	G#175423 A-4, CL-ML WC=30.5% LL=27 PL=20 PI=7
	V5		45.63 - 46.00								
	V6		46.63 - 47.00	Su=313/45 psf							
50											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream Location: New Sharon, Maine	Boring No.: BB-NS-102 WIN: 19178.10

Driller:	MaineDOT	Elevation (ft.)	332.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Wilder/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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
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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.
50	1U	24/24	50.00 - 52.00							Dark grey, soft, Clayey SILT, trace sand (Marine Clay).	
	V7		52.63 - 53.00	Su=313/22 psf						55x110 mm vane raw torque readings: V7: 7.0/0.5 ft-lbs V8: 10.0/1.0 ft-lbs	
	V8		53.63 - 54.00	Su=447/45 psf							
55											
60	V9		60.63 - 61.00	Su=447/22 psf						55x110 mm vane raw torque readings: V9: 10.0/0.5 ft-lbs V10: 10.0/1.0 ft-lbs	
	V10		61.63 - 62.00	Su=447/45 psf							
							WOC				
65											
70	7D		70.00 - 72.00	Su=647/89 psf						Dark grey, medium stiff, Clayey SILT, trace sand, (Marine Clay). 55x110 mm vane raw torque readings: V11: 14.5/2.0 ft-lbs V12: 15.5/2.0 ft-lbs	G#175424 A-6, CL WC=30.1% LL=33 PL=22 PI=11
	~~V11~~		~~70.63 - 71.00~~								
	V12		71.63 - 72.00	Su=692/89 psf							
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102
WIN: 19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N-uncorrected = Raw Field SPT N-value
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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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.
75											
80	V13		80.63 - 81.00	Su=848/67 psf						55x110 mm vane raw torque readings: V13: 19.0/1.5 ft-lbs V14: 23.5/1.0 ft-lbs	
	V14		81.63 - 82.00	Su=1049/45 psf			HYD				
							PUSH				
85											
90											
							a12	241.1		91.7 NW casing fetches up, probable top of TILL. a12 blows for 0.3 ft.	
							57				
							75				
							93				
95	8D	24/10	95.00 - 97.00	14/15/13/21	28	42	ROLL CONE AHEAD			Dark grey, dense, fine SAND, some silt, little rounded gravel, trace clay, (Glacial Till).	G#175425 A-4, SC-SM WC=11.0%
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream
Location: New Sharon, Maine

Boring No.: BB-NS-102

WIN: 19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100											
										Roller Cone refusal at 103.7 FT BGS. Casing refusal at 103.8 ft bgs. Broke through cobble at approx. 104.1 ft bgs.	
105											
110										Wash: fine sand (Glacial Till). Drilling behavior: occasional gravel seams from 104 to 116 ft bgs.	
115											
										NW refusal, possible top of rock. Broke through at 116.4 ft bgs.	
120										Drilling behavior: cobbles and gravel layers more common with depth, typically 2-4" thick.	
125											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Gray Farm Bridge #6486 carries Routes 2/27 over Gray Farm Stream	**Location:**	New Sharon, Maine	**Boring No.:**	BB-NS-102
				WIN:	19178.10

Driller:	MaineDOT	**Elevation (ft.)**	332.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Wilder/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/22/2014-12/30/2014	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+12.7, 12.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	13.5 ft bgs.
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
125										Boney Glacial Till; frequent gravel and cobbles.	
130								202.8		130.0 **Bottom of Exploration at 130.0 feet below ground surface.** No refusal encountered.	
135											
140											
145											
150											

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): New Sharon Work Number: 19178.10

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-NS-101, 2D	23+90.7	11.7 Rt.	5.0-7.0	243022	1	11.1			SM	A-2-4	II
BB-NS-101, 3D	23+90.7	11.7 Rt.	10.0-12.0	243023	1	6.1			SP-SM	A-3	0
BB-NS-101, 7D	23+90.7	11.7 Rt.	30.0-32.0	243024	1	26.3	28	1	ML	A-4	IV
BB-NS-101, 8D	23+90.7	11.7 Rt.	35.0-37.0	243025	1	26.4	27	1	ML	A-4	IV
BB-NS-102, 1D	24+12.7	12.0 Lt.	15.0-17.0	175420	2	22.9			ML	A-4	IV
BB-NS-102, 4D	24+12.7	12.0 Lt.	35.0-37.0	175421	2	28.7	25	3	ML	A-4	IV
BB-NS-102, 5D	24+12.7	12.0 Lt.	40.0-42.0	175422	2	29.4	25	3	ML	A-4	IV
BB-NS-102, 6D	24+12.7	12.0 Lt.	45.0-47.0	175423	2	30.5	27	7	CL-ML	A-4	IV
BB-NS-102, 7D	24+12.7	12.0 Lt.	70.0-72.0	175424	2	30.1	33	11	CL	A-6	IV
BB-NS-102, 8D	24+12.7	12.0 Lt.	95.0-97.0	175425	2	11.0			SC-SM	A-4	III

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

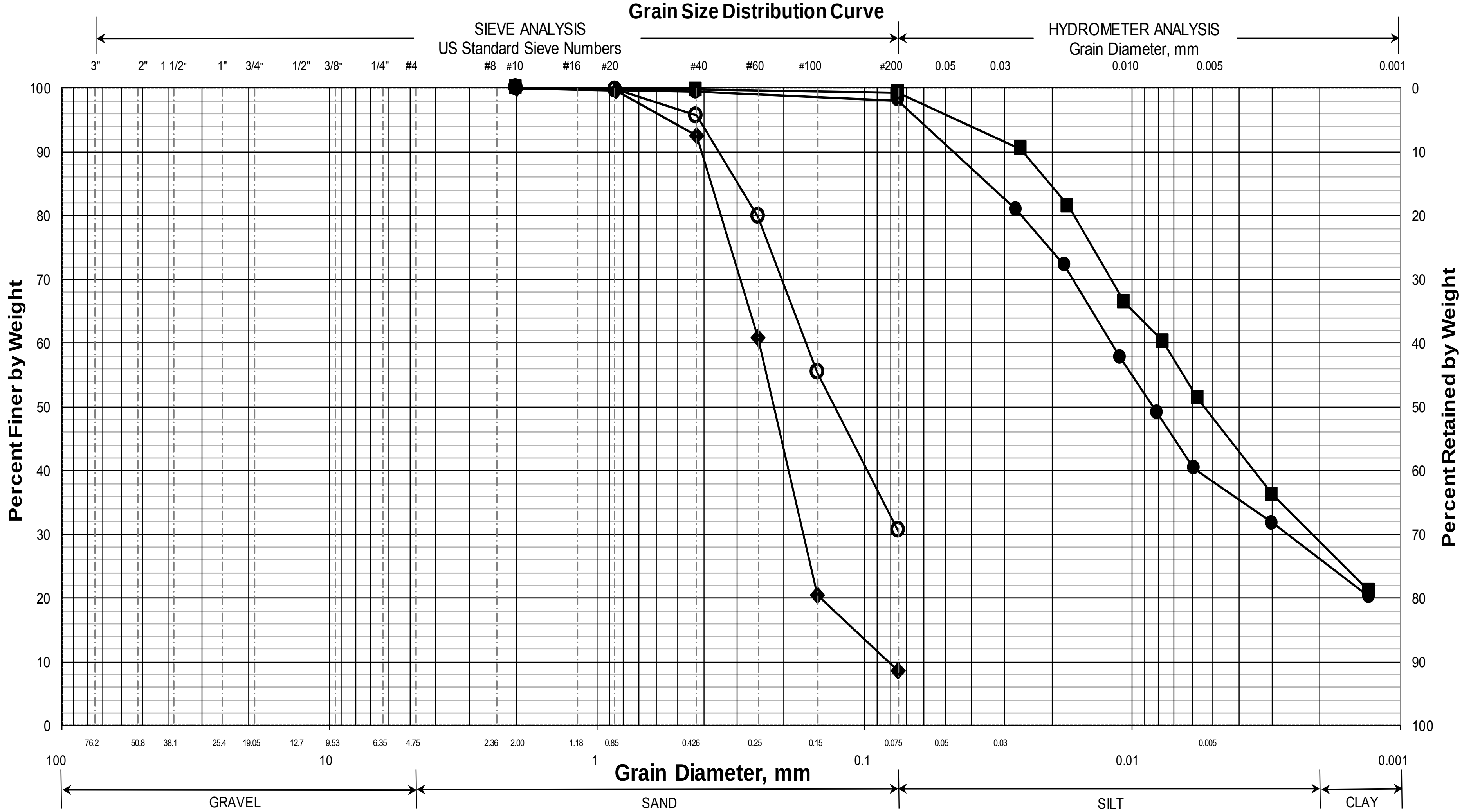
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



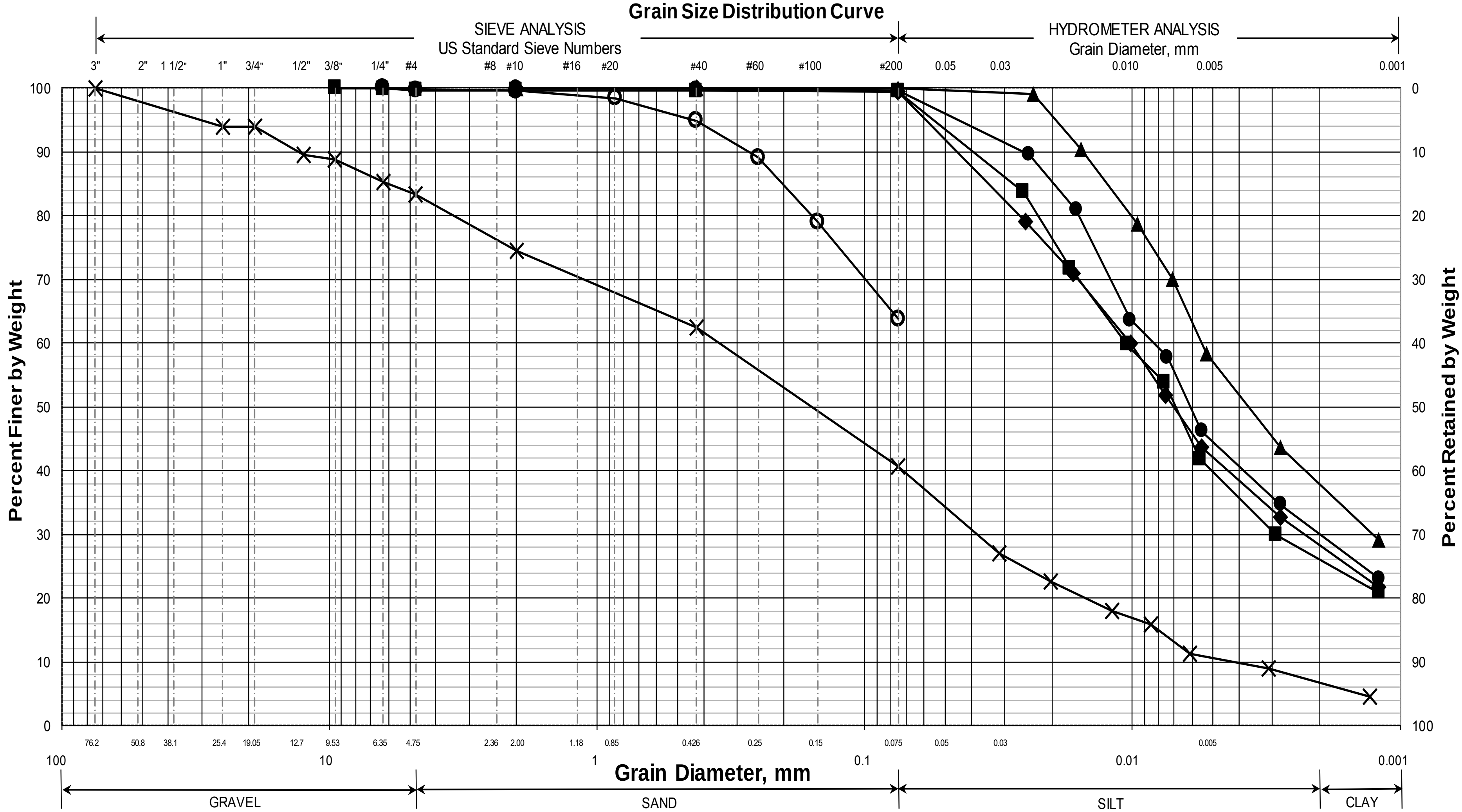
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-NS-101/2D	23+90.7	11.7 RT	5.0 - 7.0	SAND, some silt.	11.1			
◆	BB-NS-101/3D	23+90.7	11.7 RT	10.0-12.0	SAND, trace silt.	6.1			
■	BB-NS-101/7D	23+90.7	11.7 RT	30.0-32.0	SILT, some clay, trace sand.	26.3	28	27	1
●	BB-NS-101/8D	23+90.7	11.7 RT	35.0-37.0	SILT, some clay, trace sand.	26.4	27	26	1
▲									
×									

WIN	
019178.10	
Town	
New Sharon	
Reported by/Date	
WHITE, TERRY A	9/20/2018

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-NS-102/1D	24+12.7	12.0 LT	15.0-17.0	SILT, some sand, trace gravel.	22.9			
◆	BB-NS-102/4D	24+12.7	12.0 LT	35.0-37.0	SILT, some clay, trace sand.	28.7	25	22	3
■	BB-NS-102/5D	24+12.7	12.0 LT	40.0-42.0	SILT, some clay, trace sand, trace gravel.	29.4	25	22	3
●	BB-NS-102/6D	24+12.7	12.0 LT	45.0-47.0	SILT, some clay, trace sand.	30.5	27	20	7
▲	BB-NS-102/7D	24+12.7	12.0 LT	70.0-72.0	Clayey SILT, trace sand.	30.1	33	22	11
×	BB-NS-102/8D	24+12.7	12.0 LT	95.0-97.0	SAND, some silt, little gravel, trace clay.	11			

WIN	
019178.10	
Town	
New Sharon	
Reported by/Date	
WHITE, TERRY A	9/20/2018

SHEET 2

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
243024	**BB-NS-101/7D**	**GEOTECHNICAL (DISTURBED)**	**9/18/2014**	**10/17/2014**

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **23+90.7** Offset, ft: **11.7 RT** Dbfg, ft: **30.0-32.0**

WIN/Town **019178.10 - NEW SHARON** 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]	**99.8**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**99.3**
[0.0261 mm]	**90.5**
[0.0174 mm]	**81.5**
[0.0107 mm]	**66.4**
[0.0077 mm]	**60.3**
[0.0057 mm]	**51.3**
[0.0030 mm]	**36.2**
[0.0013 mm]	**21.1**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**28**
Plastic Limit (T 90), %	**27**
Plasticity Index (T 90), %	**1**
Specific Gravity, Corrected to 20°C (T 100)	**2.80**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**26.3**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

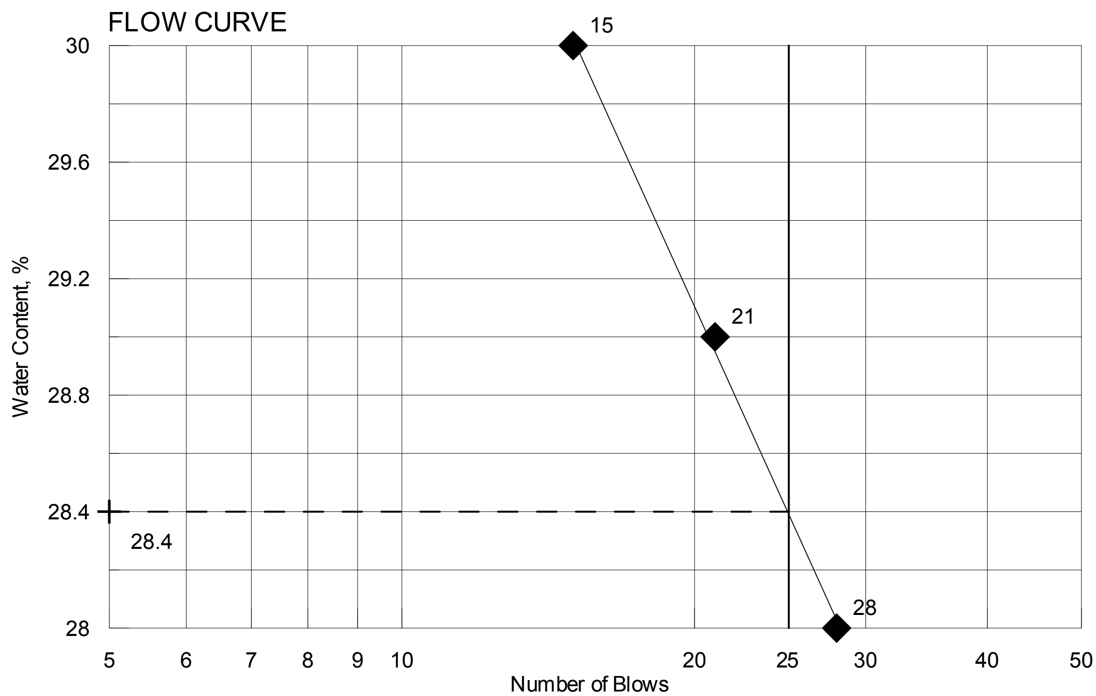
Comments:

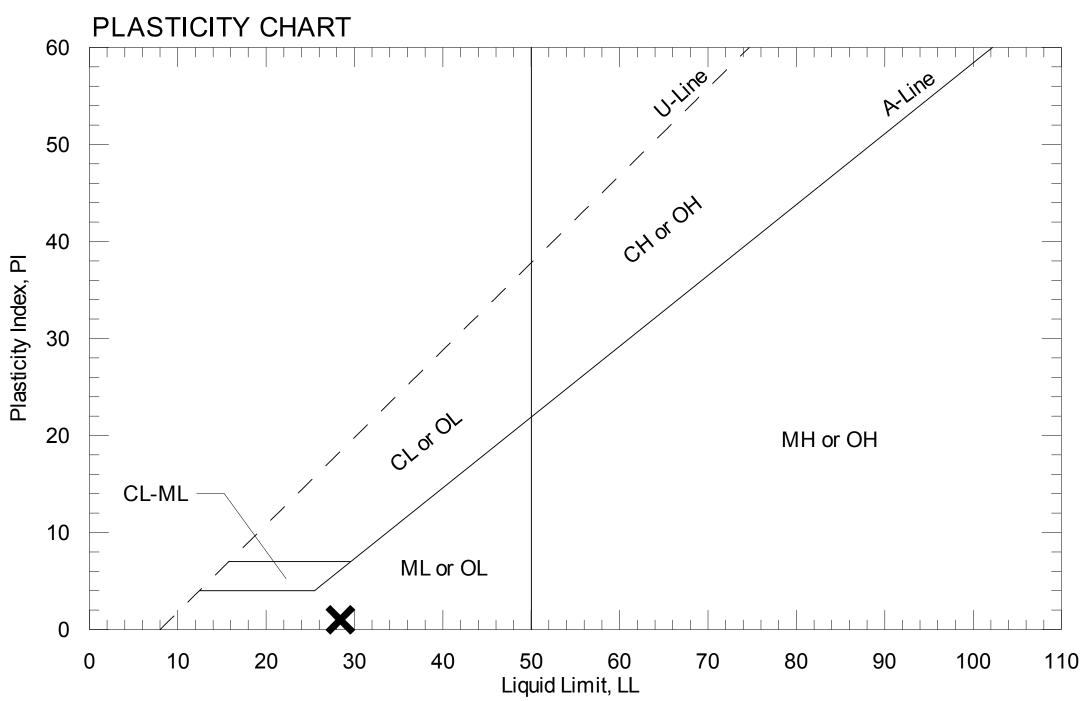
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **10/24/2014**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	243024
WIN	019178.10	Water Content, %	26.3
Sampled	9/18/2014	Liquid Limit @ 25 blows (T 89), %	28
Boring No./Sample No.	BB-NS-101/7D	Plastic Limit (T 90), %	27
Station	23+90.7	Plasticity Index (T 90), %	1
Depth	30.0-32.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
243025	BB-NS-101/8D	GEOTECHNICAL (DISTURBED)	9/18/2014	10/17/2014

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **23+90.7** Offset, ft: **11.7 RT** Dbfg, ft: **35.0-37.0**

WIN/Town **019178.10 - NEW SHARON** 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]	99.4
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	98.1
[0.0271 mm]	80.9
[0.0179 mm]	72.2
[0.0111 mm]	57.8
[0.0081 mm]	49.1
[0.0059 mm]	40.4
[0.0030 mm]	31.8
[0.0013 mm]	20.2

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	27
Plastic Limit (T 90), %	26
Plasticity Index (T 90), %	1
Specific Gravity, Corrected to 20°C (T 100)	2.79
Loss on Ignition, % (T 267)	
Water Content (T 265), %	26.4

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

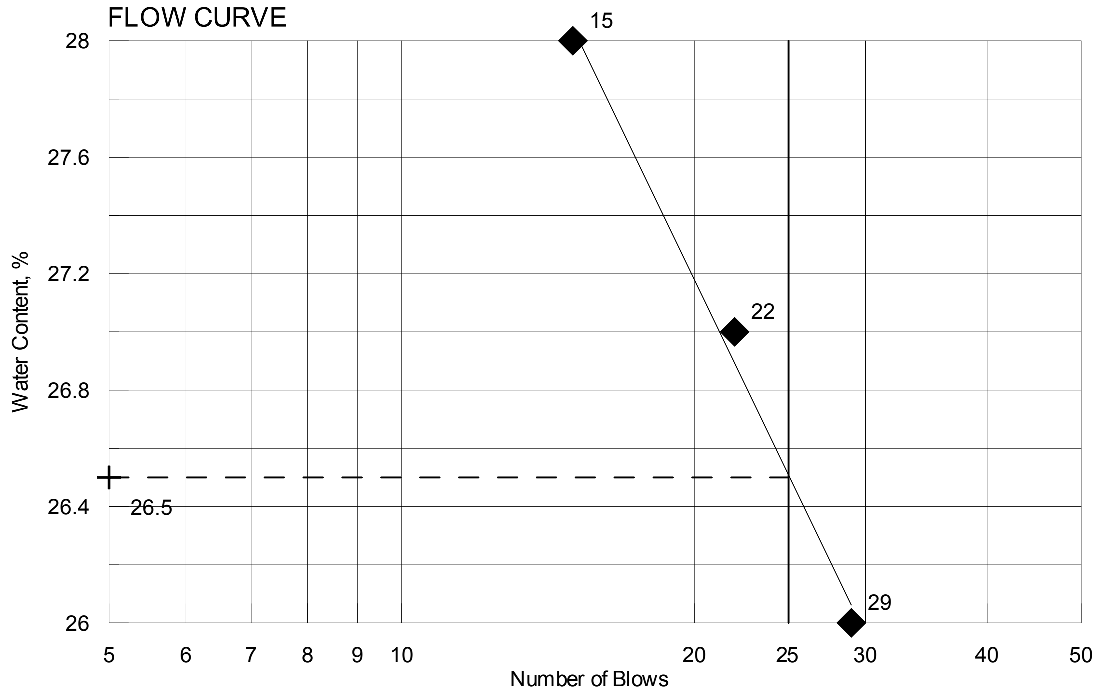
Comments:

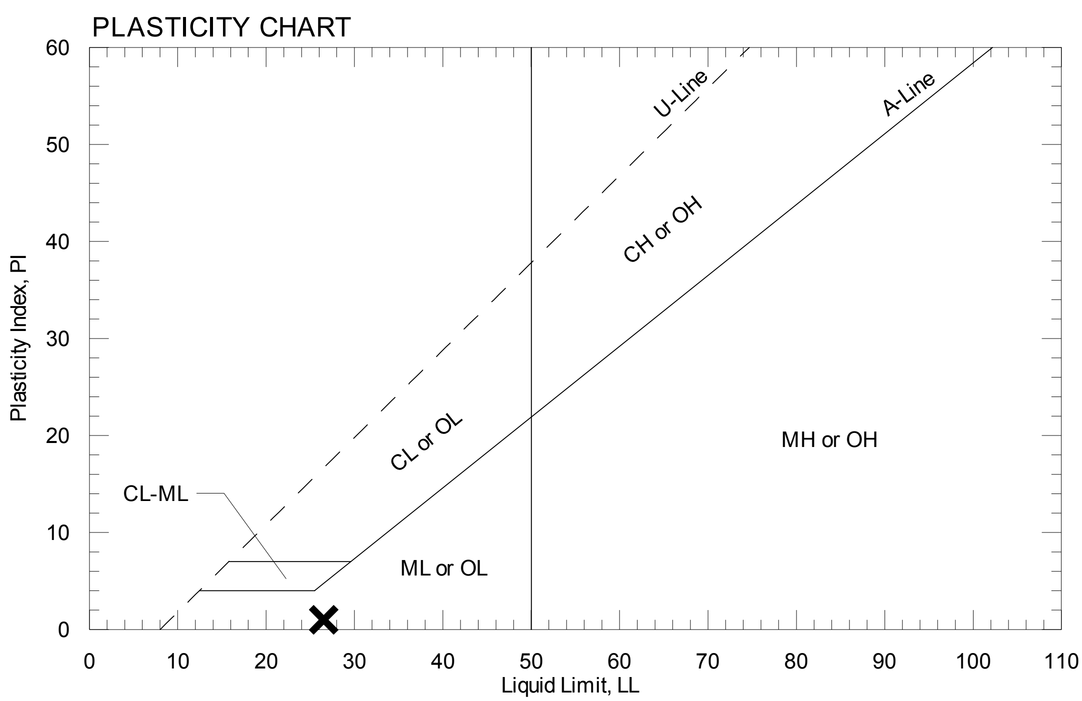
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **10/24/2014**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	243025
WIN	019178.10	Water Content, %	26.4
Sampled	9/18/2014	Liquid Limit @ 25 blows (T 89), %	27
Boring No./Sample No.	BB-NS-101/8D	Plastic Limit (T 90), %	26
Station	23+90.7	Plasticity Index (T 90), %	1
Depth	35.0-37.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
175421	BB-NS-102/4D	GEOTECHNICAL (DISTURBED)	12/22/2014	1/7/2015

Sample Type: **GEOTECHNICAL** Location: Station: **24+12.7** Offset, ft: **12.0 LT** Dbfg, ft: **35.0-37.0**

WIN/Town **019178.10 - NEW SHARON** 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]	99.5
[0.0251 mm]	79.1
[0.0167 mm]	70.9
[0.0102 mm]	60.0
[0.0075 mm]	51.8
[0.0055 mm]	43.6
[0.0028 mm]	32.7
[0.0012 mm]	21.8

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	25
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	3
Specific Gravity, Corrected to 20°C (T 100)	2.76
Loss on Ignition, % (T 267)	
Water Content (T 265), %	28.7

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. 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

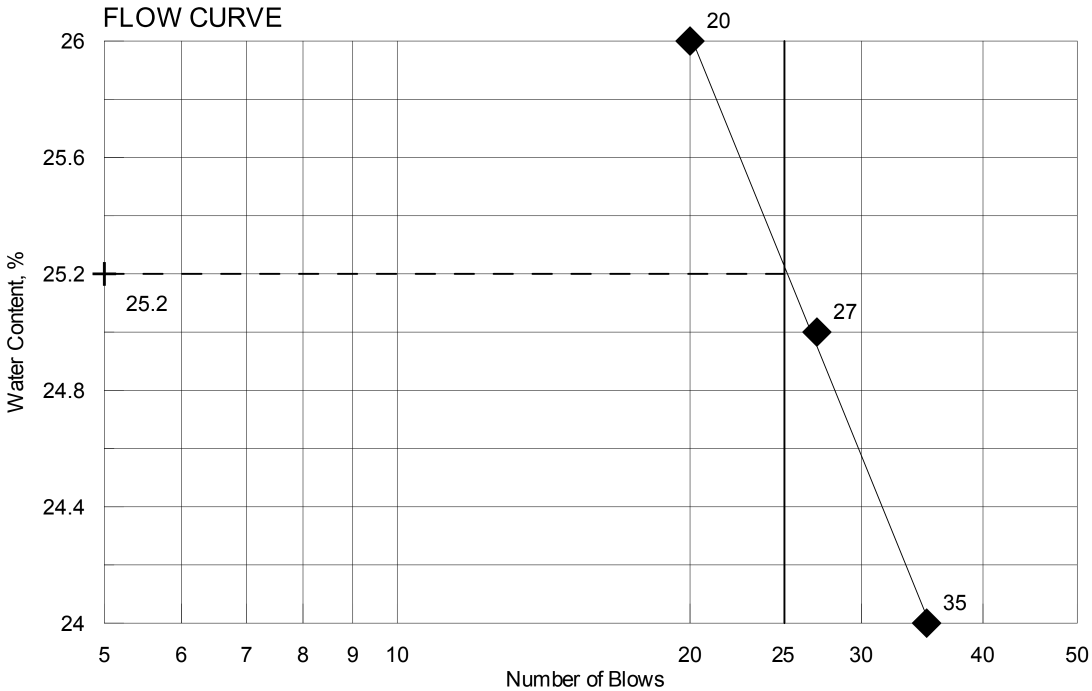
Comments:

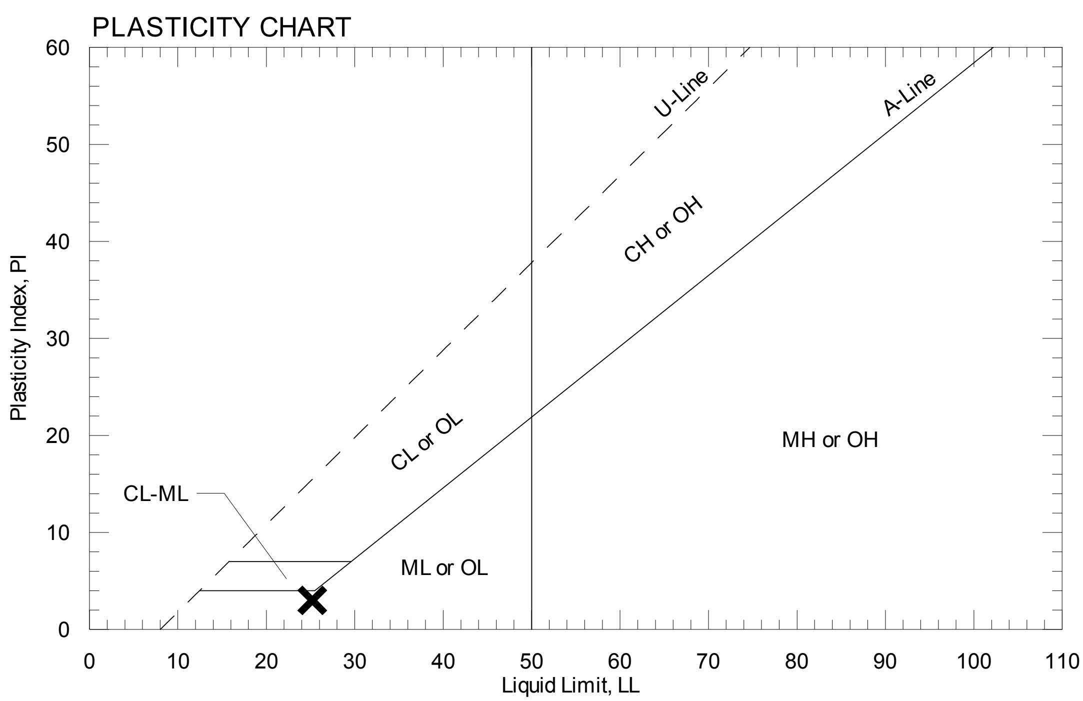
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **1/16/2015**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	175421
WIN	019178.10	Water Content, %	28.7
Sampled	12/22/2014	Liquid Limit @ 25 blows (T 89), %	25
Boring No./Sample No.	BB-NS-102/4D	Plastic Limit (T 90), %	22
Station	24+12.7	Plasticity Index (T 90), %	3
Depth	35.0-37.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
175422	BB-NS-102/5D	GEOTECHNICAL (DISTURBED)	12/22/2014	1/7/2015

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **24+12.7** Offset, ft: **12.0 LT** Dbfg, ft: **40.0-42.0**

WIN/Town **019178.10 - NEW SHARON** 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]	100.0
¼ in. [6.3 mm]	99.9
No. 4 [4.75 mm]	99.8
No. 10 [2.00 mm]	99.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.5
[0.0256 mm]	83.7
[0.0171 mm]	71.8
[0.0105 mm]	59.8
[0.0076 mm]	53.8
[0.0056 mm]	41.9
[0.0029 mm]	29.9
[0.0012 mm]	20.9

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	25
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	3
Specific Gravity, Corrected to 20°C (T 100)	2.76
Loss on Ignition, % (T 267)	
Water Content (T 265), %	29.4

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

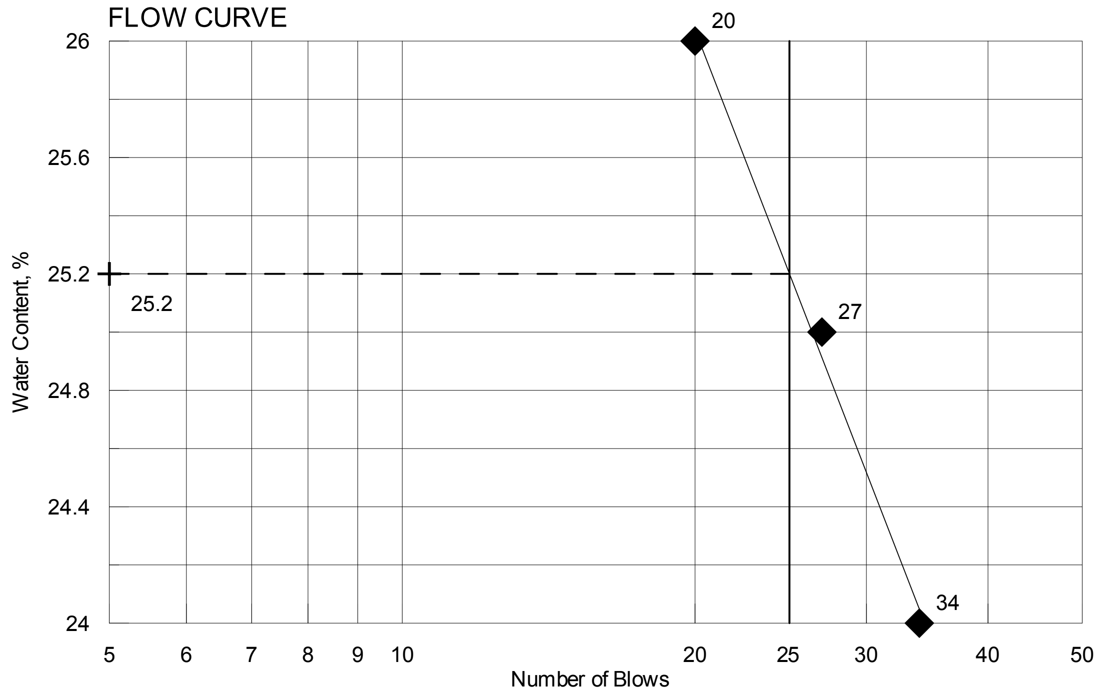
Comments:

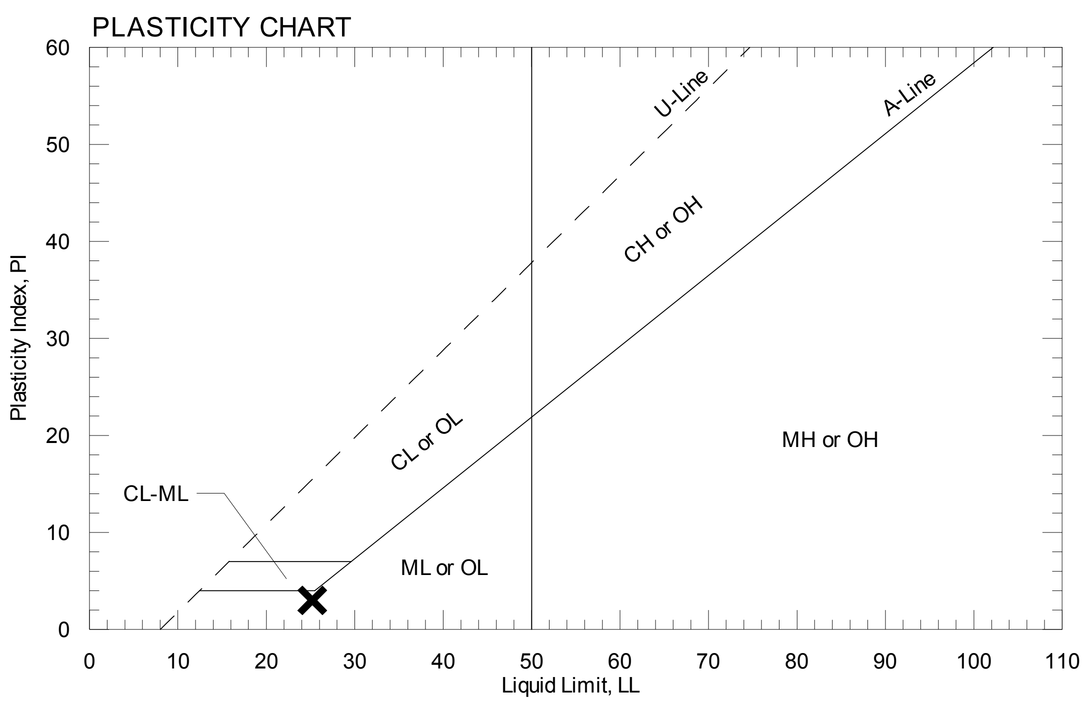
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **1/16/2015**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	175422
WIN	019178.10	Water Content, %	29.4
Sampled	12/22/2014	Liquid Limit @ 25 blows (T 89), %	25
Boring No./Sample No.	BB-NS-102/5D	Plastic Limit (T 90), %	22
Station	24+12.7	Plasticity Index (T 90), %	3
Depth	40.0-42.0	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
175423	**BB-NS-102/6D**	**GEOTECHNICAL (DISTURBED)**	**12/22/2014**	**1/7/2015**

Sample Type: **GEOTECHNICAL** Location: Station: **24+12.7** Offset, ft: **12.0 LT** Dbfg, ft: **45.0-47.0**

WIN/Town **019178.10 - NEW SHARON** 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]	**99.6**
[0.0243 mm]	**89.6**
[0.0162 mm]	**80.9**
[0.0102 mm]	**63.6**
[0.0074 mm]	**57.8**
[0.0055 mm]	**46.2**
[0.0028 mm]	**34.7**
[0.0012 mm]	**23.1**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**27**
Plastic Limit (T 90), %	**20**
Plasticity Index (T 90), %	**7**
Specific Gravity, Corrected to 20°C (T 100)	**2.77**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**30.5**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. 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

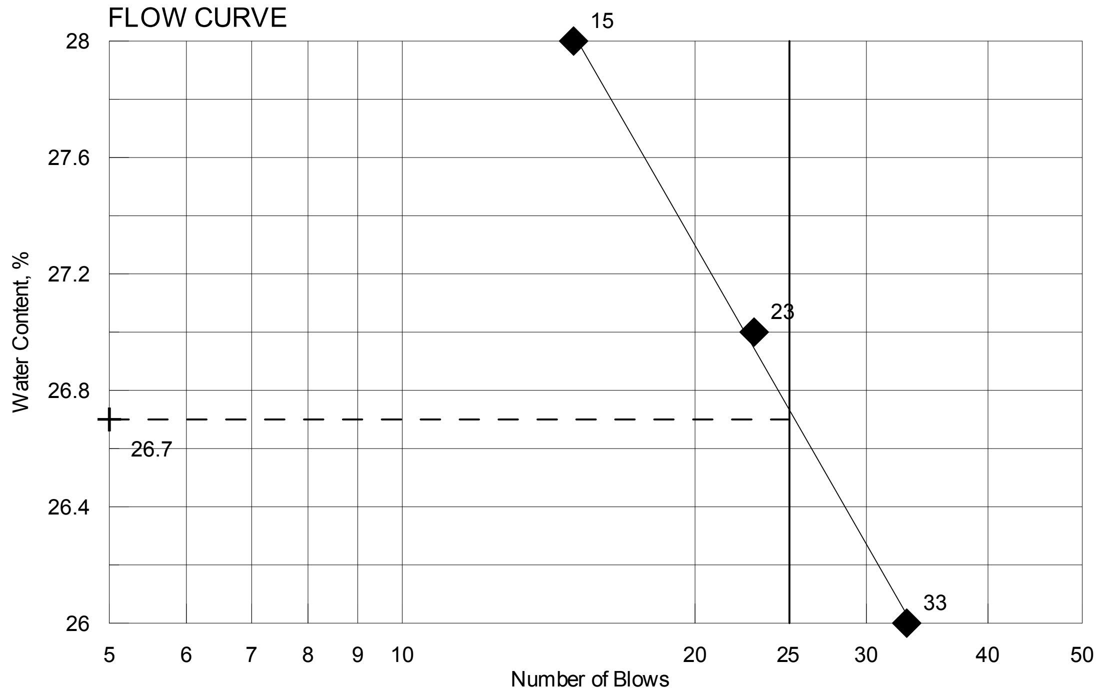
Comments:

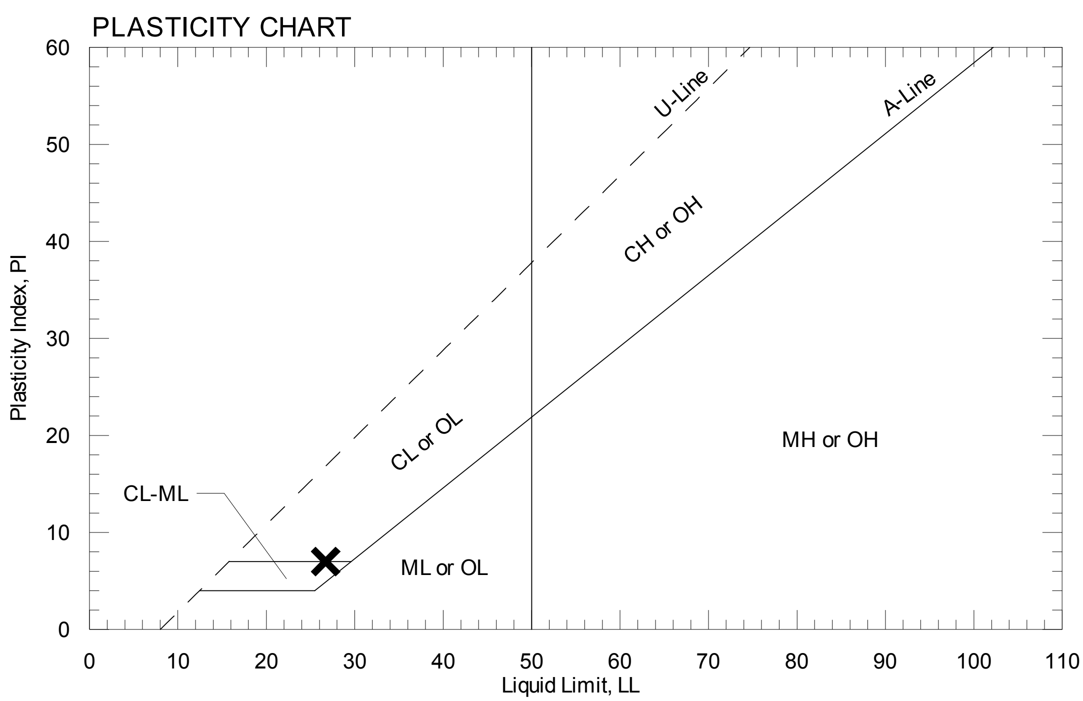
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **1/15/2015**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	175423
WIN	019178.10	Water Content, %	30.5
Sampled	12/22/2014	Liquid Limit @ 25 blows (T 89), %	27
Boring No./Sample No.	BB-NS-102/6D	Plastic Limit (T 90), %	20
Station	24+12.7	Plasticity Index (T 90), %	7
Depth	45.0-47.0	Tested By	BBURR





12 Desert Rd, Freeport   MaineDOT TESTING LABORATORIES   219 Hogan Rd, Bangor

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
175424	**BB-NS-102/7D**	**GEOTECHNICAL (DISTURBED)**	**12/29/2014**	**1/7/2015**

Sample Type: **GEOTECHNICAL** Location: Station: **24+12.7** Offset, ft: **12.0 LT** Dbfg, ft: **70.0-72.0**

WIN/Town **019178.10 - NEW SHARON** 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**
[0.0234 mm]	**99.1**
[0.0156 mm]	**90.4**
[0.0096 mm]	**78.7**
[0.0071 mm]	**70.0**
[0.0053 mm]	**58.3**
[0.0028 mm]	**43.7**
[0.0012 mm]	**29.2**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**33**
Plastic Limit (T 90), %	**22**
Plasticity Index (T 90), %	**11**
Specific Gravity, Corrected to 20°C (T 100)	**2.79**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**30.1**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. 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

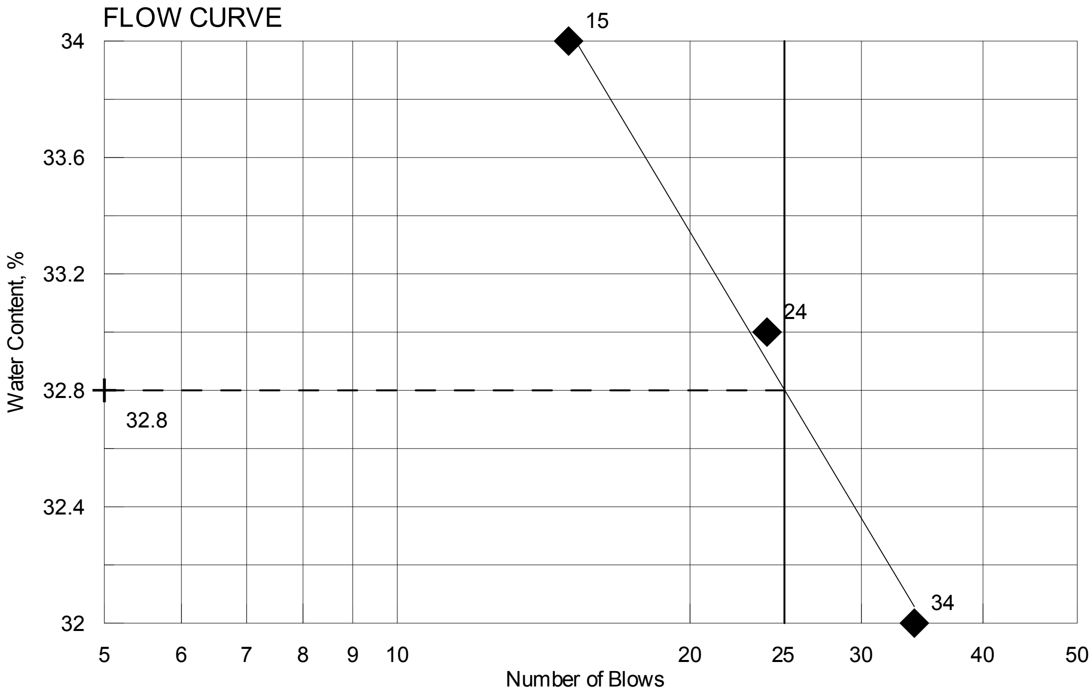
Comments:

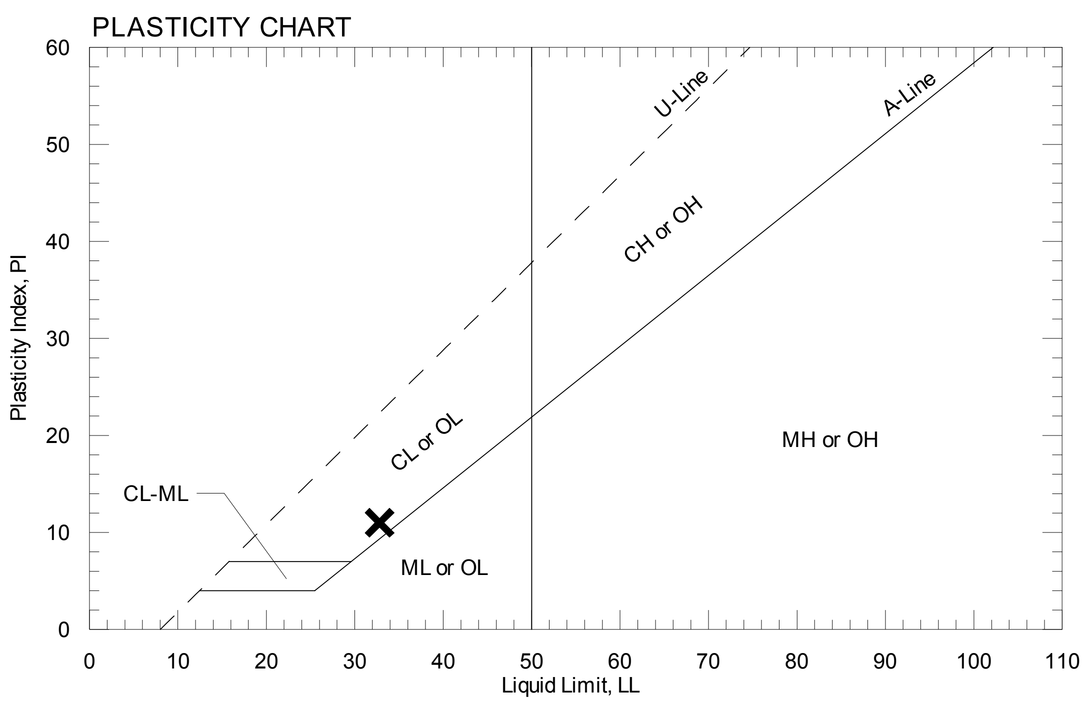
AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG** Date Reported: **1/16/2015**

Paper Copy: Lab File; Project File; Geotech File

TOWN	New Sharon	Reference No.	175424
WIN	019178.10	Water Content, %	30.1
Sampled	12/29/2014	Liquid Limit @ 25 blows (T 89), %	33
Boring No./Sample No.	BB-NS-102/7D	Plastic Limit (T 90), %	22
Station	24+12.7	Plasticity Index (T 90), %	11
Depth	70.0-72.0	Tested By	BBURR





Appendix C

Calculations

Soil Parameters:

Assume existing material removed and replaced with material with properties similar to Soil Type 4, MaineDOT BDG Section 3.6.1.

Unit weight $\gamma := 125 \cdot pcf$

Internal friction angle $\phi := 32 \cdot deg$

Cohesion $c := 0 \cdot psf$

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

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

Fang, Foundation Engineering Handbook 2nd ed. Pg. 224, Eq. 6.2

$$K_o = 0.47$$

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

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

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

For cantilver walls with horizontal backslope:

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

MaineDOT BDG Pg. 3-7

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

MaineDOT BDG Pg. 3-7

$$K_{ar\_slope} = 0.46$$

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

3.4 Construction Loads

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

3.5 Railroad Loads

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

3.6 Earth Loads

3.6.1 General

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

Table 3-3 Material Classification

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

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

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

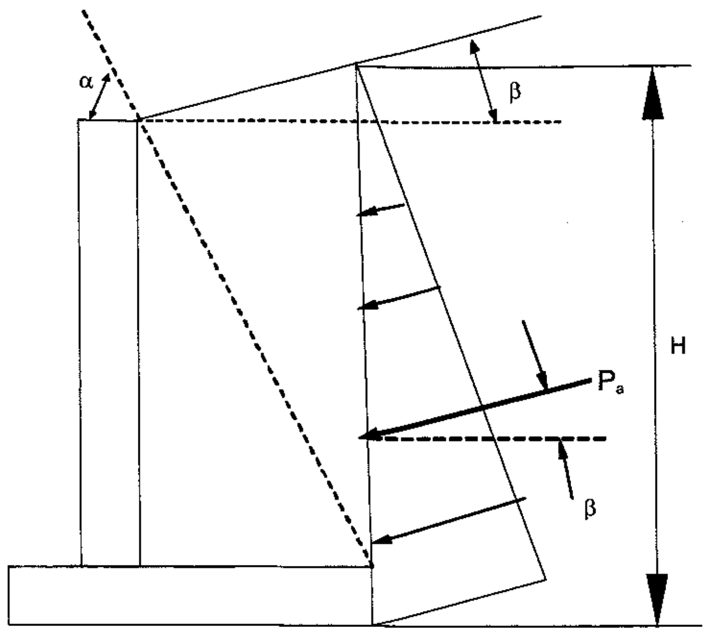


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

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

Given:
1. Limited lab data
2. Soil strength properties based on correlations to SPT N-values
3. Undrained shear strength from vanes shear tests or correlations to N-values

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 2 feet; streambed to be lined with 2 feet riprap covered by 1 foot of special fill.Include 2 foot crushed stone mat as Foundation Footing, therefore Foundation Depth is 4.0 feet
2. Crushed stone considered part of base slab
3. The proposed bearing elevation slopes from El. 313 at inlet to 311 at outlet.
4. Bearing elevation of crushed stone mat is El. 311 at inlet and 309 at outlet.
5. Proposed finish roadway grade elevation is approximately 333 feet.
6. Proposed precast concrete box has a span of 15 feet and a base of 17 feet wide. Use dimensions of geogrid reinforced crushed stone mat as the Foundation Width, 17 + 3 = 20 feet
7. Based on BB-NS-102 only 3 feet of Recent Alluvium separates bottom of crushed stone mat from the top of the Marine Clay. Model subsurface soil conditions based on upper Marine Clay deposit, Su=335 to 580 psf (BB-NS-102)and WOR (BB-NS-101)
8. The bottom of the box culvert will be submerged for the structure's design life.

1. Estimate the factored bearing resistance at the Service Limit State:

The use of presumptive values may be used when sufficient knowledge of geological conditions at or near the structure site exists. AASHTO LRFD Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2*, *May 1983*, *Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Soft to Medium Stiff	1-2, 2-4	2

The Marine Clay (ML) deposit is soft to medium stiff in consistency below the Recent Alluvium. Recommend 2 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Assumed Foundation Width, Depth, and Water Surface

$B := 20ft$

$D_f := 4.0 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

Foundation soils:

$\gamma 1_d := 83 \cdot pcf$ — Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 Soft Clay

$w_{sat} := .26$ — Moisture content of saturated samples BB-NS-101;7D, 8D

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

$\gamma 1_{sat} = 104.58 \cdot pcf$

$\phi := 0 \cdot deg$ — Lambe and Whitman, Soil Mechanics, 1969 Figure 11.14 N vs. phi

Cohesion $c_1 := 335 \cdot psf$

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

Shape Factors for strip footing

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

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

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

Nominal Bearing Resistance per Terzaghi equation

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $q = 0.169 \cdot ksf$ — Das Principles of Foundation Engineering 7th Ed. p. 142: Eq. 3.16 Water table modification

$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$ — Bowles Foundation Analysis and Design 5th Ed. p. 220: Table 4-1 Bearing-capacity Equations

$q_n = 1.9 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.50$

$q_r := q_n \cdot \varphi_b$

$\boxed{q_r = 0.95 \cdot ksf}$

Nominal Bearing Resistance for Strength Limit States

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

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

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

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

$N_c := 5.14$

$N_q := 1$

$N_\gamma := 0$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume:

$L := 80ft$

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

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

$s_\gamma = 0.9$ $s_q = 1$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

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

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

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

Load Inclination factors

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

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

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

LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.2$$

Therefore : $d_q := 1.0$

Terms

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

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

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

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

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

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

$$q_n = 2 \cdot ksf$$

Factored Bearing Resistance

$$\phi_b := 0.50$$

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

$$q_r = 1 \cdot ksf$$

Recommend a factored bearing resistance of 1 ksf for precast box culvert constructed on Alluvial Soils overlying Marine Clay.

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

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

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

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

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft^3	Dry unit weight, γ_d 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

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

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

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

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

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

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

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

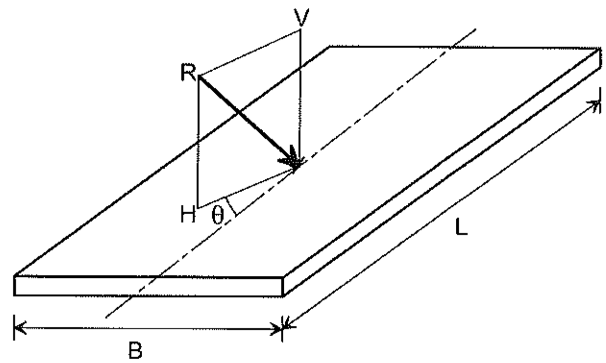


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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

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

AASHTO LRFD Bridge Design Specification, 7th ed. 2014

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

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

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

C10.5.5.2.2

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

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

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

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

Given:
1. Boring logs, SPT N-values, Su from vane shear tests, lab data

Assumptions:
1. The box culvert embedded 2 feet into the streambed.
2. The two foot layer of crushed stone bedding is not considered.
3. The proposed bearing elevation of base slab slopes from Elev. 313 at inlet to 311 at outlet.
4. Proposed finished grade matches existing
5. Proposed precast concrete box is 17 feet wide and 80 feet long (excluding slab connecting slope tapered outlet and inlet sections).
6. 3 feet of Recent Alluvium (Silty Sand) separates bottom of crushed stone mat from top of Marine Clay. Model subsurface conditions based on upper Marine Caly deposit, Su=335 to 580 psf
7. The bottom of the box culvert will be submerged for the structure's design life.

Published values of subgrade modulus

Published values of subgrade modulus in submerged, loose, sand and soft clay:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:
Range of modulus of subgrade reaction
Subgrade of Clayey soil, qa < 4000 psf or 4 ksf: k_s = 44 pci
Subgrade of Loose sand (Alluvium) ks = 40 pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:
Range of modulus of subgrade reaction
Loose submerged coarse-grained soils: K_{v1}, 23 pci
Clay subgrade (min. value of curve) K_{v1}, 23 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:
Typical subgrade reaction values for 0.3 m x 0.3 m plate
Stiff clay, 37-92 pci (no value for soft clay): $k_{0.3}$ (k_1) = 30 pci
Loose, saturated sand subgrade, 37-55 pci, say $k_{0.3}$ (k_1) = 40 pci

Terzaghi Geotechnique, Vol. 5, No. 4, Table 1:
Values of vertical subgrade reaction for 1 ft x 1 ft plate on sand
No values for clay, use value for loose, submerged sand (Alluvium): k_{s1} = 25 pci

Adjust Published values for dimensions of base slab

Published range for loose, saturated sand (Recent Alluvium) is 23 - 40 pci.

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

Assume a subgrade modulus of 40 pci for either situation, subgrade is sand or clay.

Value of k_{s1} = 40 pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base. (Width B - 17 ft, Length L = 80 ft)

Square to rectangle base adjustment:

$k_{s1} := 40\text{pci} \qquad B := 17\text{ft} \qquad L := 80\text{ft}$

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

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

$k = 29.5 \cdot \text{pci}$

Recommend a subgrade modulus of 30 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_sZ term by one of two simple methods:

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

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

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

to estimate a value of k_s to determine the correct order of magnitude btained using one of the approximations given here. Obviously if a three times larger than the table range indicates, the computations possible gross error. Note, however, if you use a reduced value of r 12 mm) instead of 0.0254 m you may well exceed the table range. putational error (or a poor assumption) is found then use judgment table values are intended as guides. The reader should not use, say, ven as a "good" estimate.
d in Fig. 9-9c (and used in your diskette program FADBEMLP as stimated at some small value of, say, 6 to 25 mm, or from inspection e 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

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

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

40 pci

44 pci

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

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

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

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

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

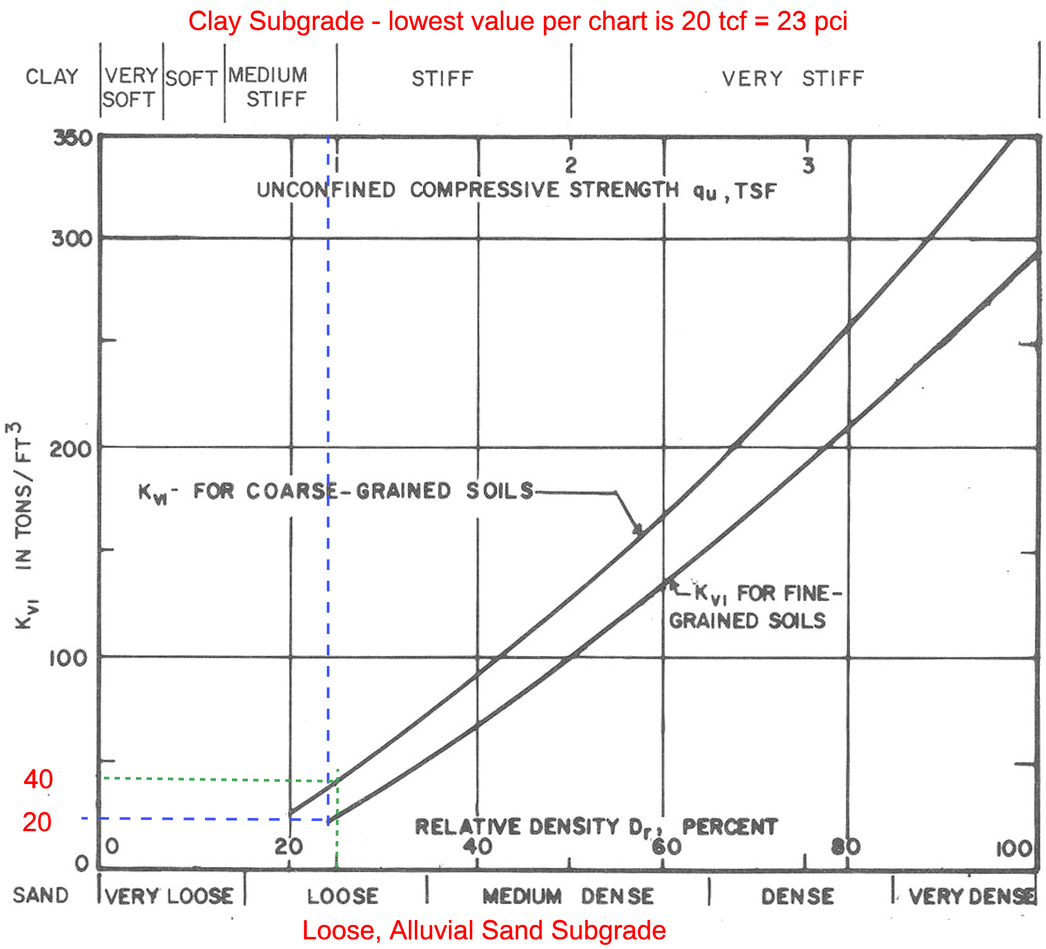
using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.
At $D/B = 0.61/1.22 = 0.5$, we obtain $I_F = 0.80$ from Fig. 5-7 (or when using FFACTOR for the I_s factors). Substitution into Eq. (9-7) with $B' = 1.22/2 = 0.61$, and $m = 4$ yields

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

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_{vl} = MODULUS OF VERTICAL SUBGRADE REACTION

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

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

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS D ≦ B

FOR B ≦ 20 FT:

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

FOR B ≧ 40 FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION D ≧ 5B

FOR B ≦ 20 FT:

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

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

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

Loose, Alluvial Sand Subgrade (saturated) k=40 tcf/2 x 1.157 = 23 pci

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

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 value 130 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
Submerged sand, proposed values	25	80	300

Loose alluvium

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

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

25 ton/ft^3 x 1.157 = 29 pci

EVAL

problem can be cor
headings. Experien
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the value:

For $l = \infty$, $\bar{k}_{s1}$ =
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 × 0.3 m, and values of k can be calculated. The value of k can be related to large foundations measuring $B \times B$ in the following ways:

Foundations on Sandy Soils

For foundations on sandy soils,

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

where $k_{0.3}$ and k = coefficients of subgrade reaction of foundations measuring 0.3 m × 0.3 m and $B\,(\mathrm{m}) \times B\,(\mathrm{m})$, 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:

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

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 $B \times B$

 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

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

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

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

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

From Design Freezing Index Map: New Sharon Maine
DFI = 1750 degree-days.
Case 1 - coarse grained granular fill soils W=20% (based on one lab test of Recent Alluvium).

For DFI = 1700 $d1 := 72.4$

For DFI = 1800 $d2 := 74.5$

$$d := in \cdot \left(\frac{d2 - d1}{10} \cdot 5 + d1 \right)$$

Depth of Frost Penetration $d = 73 \cdot in$ $d = 6.1 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Madison lies along the same Maine Design Freezing Index contour - use Madison data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Madison, Maine

Air Design Freezing Index = 1847 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1478 F-days
Mean Annual Temperature = 42.4 deg F
Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	83.2	20.0	120.0	32	44	3.2	1.7	3,456

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

Total Depth of Frost Penetration = 6.93 ft = 83.2 in.

Recommendation: 6.1 feet for design of foundations constructed on coarse grained soils