MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

HIGGINS BRIDGE
STATE ROUTE 25 OVER DOUGLAS BROOK
GORHAM, MAINE

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Cumberland County
WIN 23899.00

Soils Report 2018-53
Bridge No. 2370

Project No. 23899.00
December 14, 2018

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Appendix B – Laboratory Test Results
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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Higgins Bridge, which carries State Route 25 over Douglas Brook in Gorham, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation design recommendations, and geotechnical parameters for design of the new bridge structure.

The existing structure consists of two six-foot diameter corrugated HDPE pipes that were installed in April 2018. The previous structure, built in 1988, failed and required emergency replacement. It was a buried steel plate pipe arch, 76 feet in length, 13 feet 11 inches wide, with an 8-foot 7-inch rise. On March 30, 2018 a maintenance crew discovered that the inlet end of the pipe was unzipping along a water level bolt line for roughly 20 feet towards the roadway centerline and the condition of the structure was deemed unacceptable. The temporary structure is overlain by approximately 5 feet of fill.

The proposed replacement structure will be a 21-foot span by 8-foot rise precast concrete box culvert. The concrete box culvert will have 1-foot tall precast headwalls and toe walls extending one foot below calculated scour depth. 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 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 new box culvert will be located on approximately the same horizontal alignment as the existing plate arch. An onsite temporary detour will maintain alternating one-way traffic with signals.

2.0 GEOLOGIC SETTING

Higgins Bridge carries State Route 25 over Douglas Brook approximately 1.1 miles southeast of Dingley Spring Road, as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Standish Quadrangle (1999), Open File 99-101, indicates the surficial soil unit at the bridge site consists of glaciomarine deposits of the Presumpscot Formation. These deposits generally consist of clay and silt that washed out of the Lake Wisconsinan glacier and accumulated on the ocean floor when the relative sea level was higher than at present. This soil unit typically overlies an irregular surface of glacial till and may include areas of till exposed at the ground surface. Glacial till is a heterogeneous mixture of sand, silt, clay and stones.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the project site as the Vassalboro Formation consisting of calcareous sandstone, interbedded sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings. Boring BB-GDB-101 was drilled behind the southeast corner of the existing twin pipes and boring BB-GDB-102 was drilled behind the northwest corner. The test boring locations are shown on Sheet 2 – Boring Location Plan.

Test borings BB-GDB-101 and BB-GDB-102 were drilled between June 5 and 9, 2018 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and bedrock, soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

The borings were performed using solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT dill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" prior to the test borings in April 2017. All N values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 for both borings. The hammer efficiency factor (0.928) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

In-situ vane shear tests were made at regular intervals in the soft soils to measure the shear strength of the soils.

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

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of seven standard grain size analyses with natural water contents, seven grain size analyses with hydrometers and natural water contents and seven Atterberg limits tests. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered generally consisted of Fill, Marine Clay, and Glacial Till underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 –Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

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

- Brown, dry, sand, some gravel, little silt;
- Brown, damp, sandy silt.

SPT N-values in the layer ranged from 9 to 48 blows per foot (bpf), indicating the layer is stiff to dense in consistency. Grain size analyses conducted on samples of the Fill indicate the soil is classified as A-1-b and A-4 under the AASHTO Soil Classification System and SM and ML under the Unified Soil Classification System (USCS). The natural water contents of the samples tested ranged from approximately 6 to 19 percent.

5.2 Marine Clay Deposit

A deposit of Marine Clay (Presumpscot Formation) was encountered below the Fill unit in the borings. The thickness of the Marine Clay deposit encountered was approximately 14 to 17 feet thick. The Marine Clay deposit consisted of:

- Light brown to brown, wet, clayey silt;
- Light grey to grey, wet, clayey silt;
- Light grey to grey, wet, silty clay
- Light grey, wet, silt, some clay, trace sand.

SPT N-values in the Marine Clay were 2 and 19 bpf indicating the deposit is soft to very stiff in consistency. The remainder of SPT tests were conducted though soils disturbed by in-situ vane shear tests and therefore are invalid.

Grain size analyses with hydrometer resulted in the Marine Clay deposit being classified as A-4, A-6 and A-7-6 under the AASHTO Soil Classification System and 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 ten successful vane shear tests conducted within the silty clay layers measured undisturbed undrained shear strengths ranging from approximately 469 to 769 psf, indicating the Marine Clay is soft to medium stiff in consistency. One vane shear test showed a measured undrained shear strength of approximately 1116 psf, indicating that sublayer is stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 45 to 156 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the silty clay has a sensitivity ranging from 6 to 17 and is classified as sensitive to slightly quick.

Atterberg limits tests were conducted on seven 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-GDB-101, 2D	Clayey SILT	28.4	33	20	13	0.7
BB-GDB-101, 3D	Clayey SILT	40.1	31	21	10	1.9
BB-GDB-101, 4D	Silty CLAY	43.9	35	21	14	1.6
BB-GDB-102, 2D/B	Clayey SILT	23.5	35	20	15	0.2
BB-GDB-102, 3D	Silty CLAY	46.0	40	22	18	0.1
BB-GDB-102, 4D	Clayey SILT	34.8	26	18	8	3.5
BB-GDB-102, 5D	Clayey SILT	42.8	35	21	14	1.0

Table 1 – Summary of Atterberg Limits Test Results

The plasticity indices of the samples indicate that the soils have low to medium plasticity (Burmister, 1949). The natural water contents of the tested samples ranged from approximately 24 to 46 percent and liquid limits ranged from 26 to 40. The liquidity indices range from 0.1 to 3.5. Interpretation of these results indicates that the soils with liquidity indices of 1 or less are preconsolidated, while those with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as "quick."

5.3 Glacial Till

Glacial Till was encountered beneath the Marine Clay in the borings. The thickness of the deposit encountered was approximately 14 to 17 feet. The Glacial Till generally consisted of:

- Brown to grey to brown-grey, wet, sand, trace to some gravel, trace to little silt;

SPT N-values in the deposit ranged from 14 to 63 bpf indicating the deposit is medium dense to very dense. Grain size analyses conducted on samples of the Glacial Till resulted in the sample being classified as A-1-b, under the AASHTO Soil Classification System and SP-SM, SM, SW-SM under the USCS. The moisture contents of the tested samples ranged from approximately 11 to 18 percent.

5.4 Bedrock

Bedrock was encountered and cored in boring BB-GDB-101. Table 2 summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (R1, R2) (%)
BB-GDB-101	395+43.7	14.5 L	40.5	187.9	57, 29

Table 2 – Summary of Approximate Bedrock Depth, Approximate Bedrock Elevation, and RQD

The bedrock at the site is identified as light grey, Metasandstone/Siltstone/Shale, hard, moderately weathered, joints moderately dipping, close to moderately close, open, with little sand infilling. Detailed bedrock descriptions and the RQD core run are provided on the boring logs on Sheet 4 – Boring Logs and in Appendix A – Boring Logs.

5.5 Groundwater

Groundwater was measured at approximately 7 feet bgs in the boring BB-GDB-102. Water was introduced into the boreholes during drilling operations. Therefore, water levels may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in river water elevation, seasonally, with precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The Preliminary Design Report[1] (PDR) evaluated a 1.2 bankfull width, 21-foot wide by 8-foot tall precast concrete box culvert and a 21-foot span steel plate pipe arch.

A steel pipe arch was ruled out because the previous steel pipe failed prematurely. Furthermore, a 21-foot span steel pipe arch is typically too large for this structure type. The precast concrete box, by comparison, is readily available by local fabricators and the required span is reasonably fabricated.

The selected alternative is, therefore, a 1.2 bankfull width, precast concrete box culvert.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design

The proposed replacement structure will consist of a 21-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 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).

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[1] Preliminary Design Report, Higgins Bridge #2370 over Douglas Brook, Gorham, Maine, July 26, 2018.

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 forces resulting from creep, temperature, and shrinkage deformations of the concrete box culvert. Passive pressure resulting from the embedment of the box culvert and walls with engineered streambed, or any other, material shall not contribute to resisting forces.

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

7.1.3 Precast Concrete Toe Walls

Toe walls shall extend below the bottom slab 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 placed on the native soil subgrade. The bearing elevation of the crushed stone mat will be approximately Elevation 213 at the inlet and Elevation 212.5 at the outlet. The sensitive Marine Clay at this elevation is expected to be soft to medium stiff.

For a precast concrete box culvert with a base width of 23 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 1.5 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 $0.3f'c$. See Appendix C – Calculations for supporting calculations.

7.1.5 Modulus of Subgrade Reaction

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

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction, k_s, equal to 31 pounds per cubic inch (pci). See Appendix C – Calculations for supporting calculations.

7.2 Settlement

An approximately 12 to 14-foot thick deposit of soft to medium stiff Marine Clay was encountered at the proposed bottom slab elevation of the box culvert. The soft deposit will undergo immediate and consolidation settlement in response to a net increase of vertical overburden pressure. Based on an estimated service limit state pressure of 1,500 psf for a 23-foot wide precast concrete box, and a net applied pressure increase of 500 psf, we estimate settlements on the order of 1.5-inch during the first year after construction and an additional 1-inch of long term settlement (i.e. secondary) over 50 years. Should the net pressure increase approach 1500 psf due to the proposed box culvert, 1-year consolidation settlement could potentially be on the order of 3-inches with an additional 1-inch of long term, secondary settlement.

See Appendix C – Calculations for supporting documentation.

7.3 Subgrade Excavation and Subgrade Preparation

The Marine Clay layer encountered at the bearing elevation will easily become disturbed by construction activities. The excavation will require care to maintain bottom stability and bearing capacity. The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to allow the bearing pad construction in the dry;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade;
- Construct the box on 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 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, Gorham has a design freezing index (DFI) of approximately 1300 F-degree days. The anticipated fine grained fill soils were assigned an average water content of 25%. These components correlate to a frost depth of 3.7 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Brunswick, Maine has an air DFI from the Modberg database of approximately 1276 F-degree days. Brunswick was selected because it lies near the same isoline as Gorham and Gorham is not available in the Modberg database. A water content of 25% was assumed. These components correlate to a frost depth of approximately 3.2 feet.

Based on the CRREL Modberg analyses, it is recommended that foundations bearing on fine grained soils be designed with an embedment of approximately 3.2 feet for frost protection. See Appendix C – Calculations for supporting calculations.

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

7.5 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and wingwalls to retain slopes and prevent 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. 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 following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to limit disturbance and rutting of the Marine Clay and to allow the bearing pad construction in the dry. Cofferdams may be required to divert flow away from the new culvert location during construction.
- The contractor shall not operate heavy equipment over the excavated subgrade to minimize subgrade disturbance.
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the clay subgrade.
- The stabilization/reinforcement geotextile shall be hand-deployed on the prepared clay subgrade prior to installing the geogrid-reinforced stone mat.

The 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 temporary detour roadway for maintenance of traffic. 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 sensitive, soft soils. These soils are susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance or rutting occur, the Contractor shall remove and replace the materials with compacted granular borrow or crushed stone.

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

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Higgins Bridge in Gorham, 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.

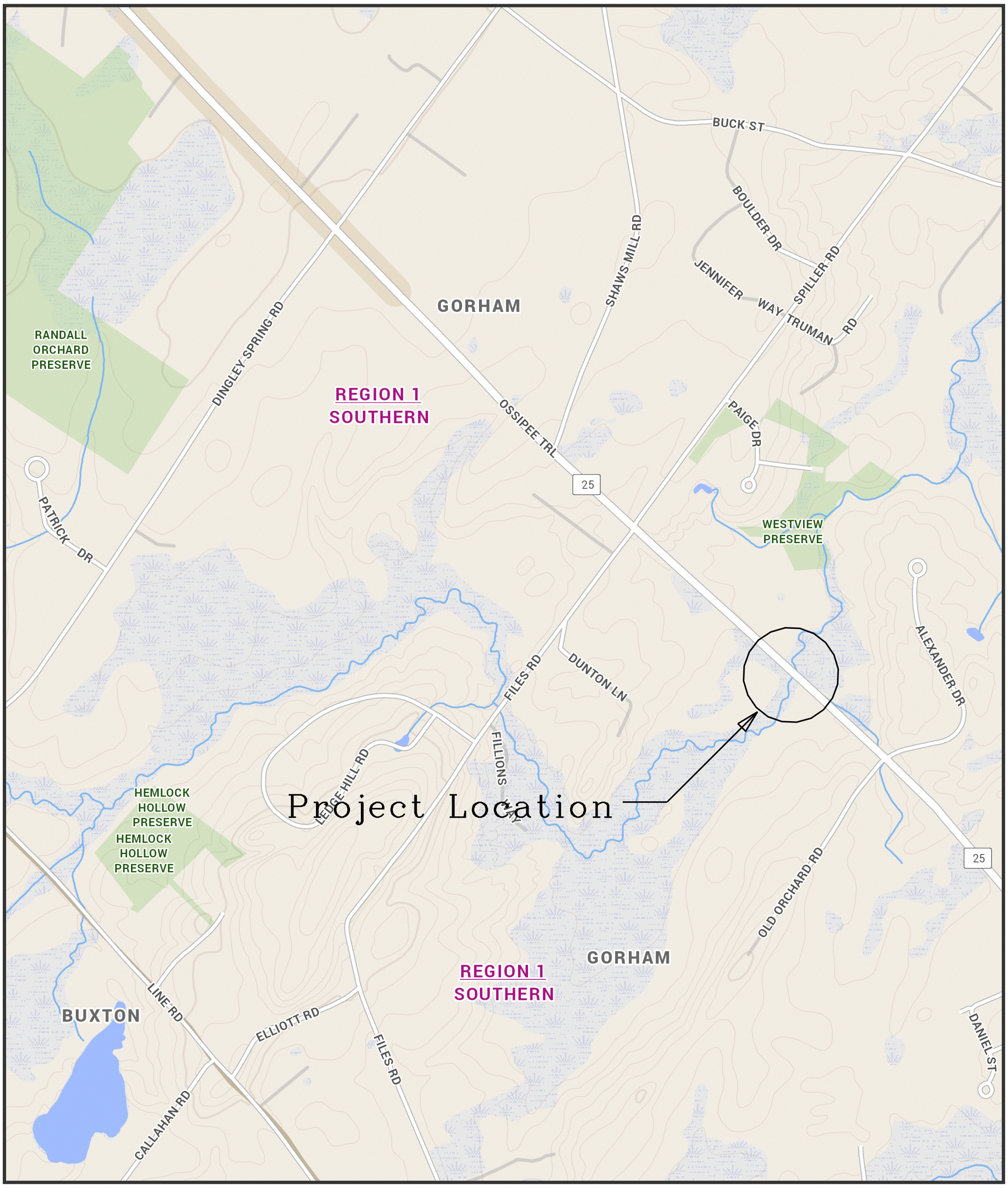
Sheets

Date:10/12/2018

Division: GEOTECH

Username: Terry.White

GORHAM, MAINE



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0.25 Miles

1 inch = 0.28 miles

Date: 10/12/2018
Time: 7:30:57 AM

SHEET NUMBER	HIGGINS BRIDGE DOUGLAS BROOK	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	GORHAM CUMBERLAND COUNTY	23899.00
OF 4	LOCATION MAP	WIN BRIDGE NO. 2370 023899.00 BRIDGE PLANS

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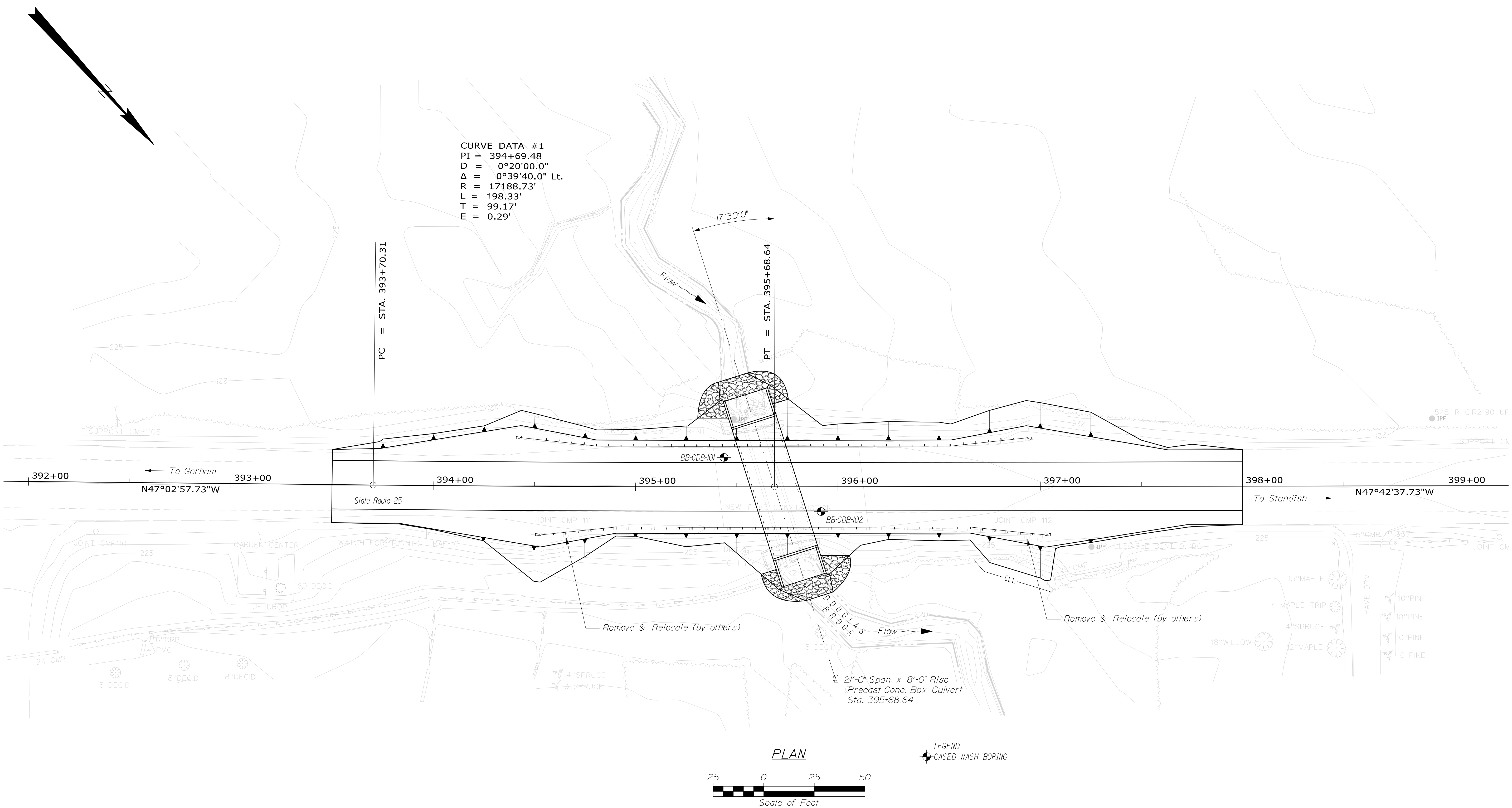


STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

23899.00

WIN 023899.00

BRIDGE NO. 2370

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	Devon Eaton	
DESIGN-DETAILED	D. Eaton	
CHECKED-REVIEWED	D. Damren	
DESIGN2-DETAILED2	A. VANBUSKIRK	
DESIGN3-DETAILED3	T. WHITE	JUN 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

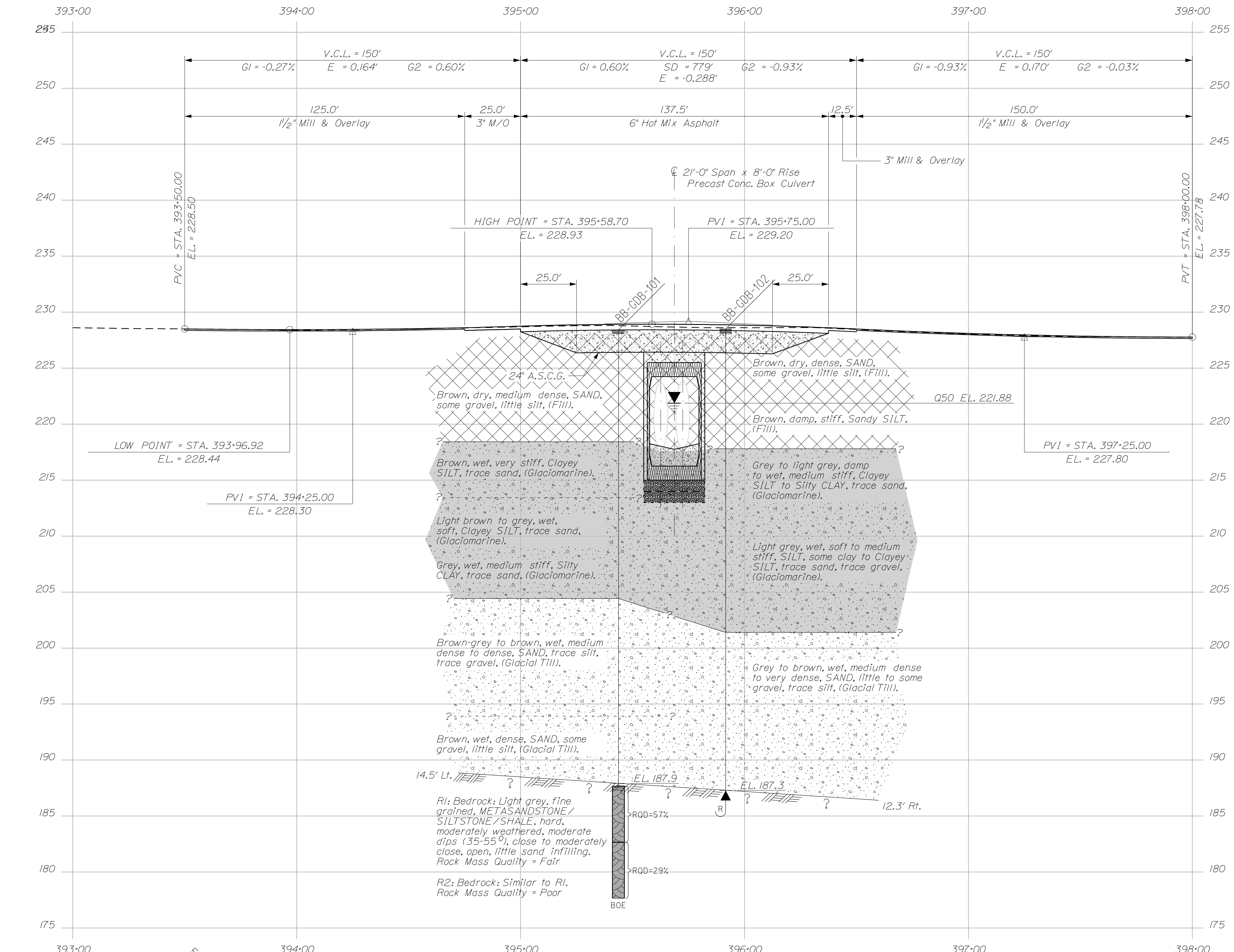
DATE

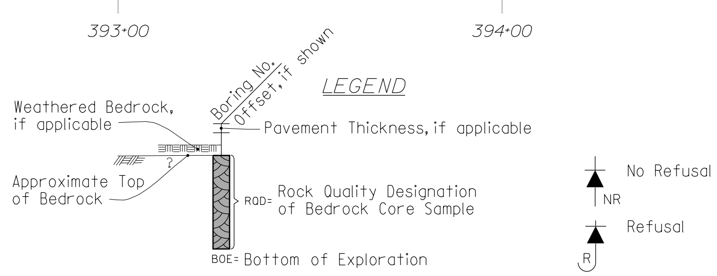
GORHAM

HIGGINS BRIDGE
DOUGLAS BROOK
CUMBERLAND COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4





PROFILE

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

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.

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

23899.00

WIN 023899.00

BRIDGE PLANS

BRIDGE NO. 2370

	BY	DATE
PROJ. MANAGER	Devon Eaton	
DESIGN-DETAILED	D. Eaton	D. Damren
CHECKED-REVIEWED	A.VANBUSKIRK	T. WHITE
DESIGN2-DETAILED2		JUN 2018
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

HIGGINS BRIDGE
DOUGLAS BROOK
GORHAM CUMBERLAND COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\MSTA\008_BORING LOGS1.dgn Division: GEOTECH Username: Andrew.VanBuskirk Date:12/17/2018

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-101

WIN: 23899.00

Driller:	MaineDOT	Elevation (ft.)	228.4	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/5/2018; 08:30-13:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	395+43.7, 14.5 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: 0.928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	228.1	4" HMA. 0.3	
5	1D	24/15	5.00 - 7.00	10/11/8/4	19	29	34		Brown, dry, medium dense, SAND, some gravel, little silt, (Fill).	G#296361 A-1-b, SM WC=10.1%
							23			
							9			
							23			
							51			
10	2D	24/20	10.00 - 12.00	4/6/6/5	12	19	37	218.4	10.0 Brown, wet, very stiff, Clayey SILT, trace sand, (Glaciomarine).	G#296362 A-6, CL WC=28.4% LL=33 PL=20 PI=13
							55			
							52			
							56			
							58			
15	3D	24/24	15.00 - 17.00	WOH/WOH/1/2	1	2	33	213.4	15.0 Light brown to grey, wet, soft, Clayey SILT, trace sand, (Glaciomarine). Roller Coned ahead to 17.0 ft bgs.	G#296363 A-6, CL WC=40.1% LL=31 PL=21 PI=10
							45			
	V1		17.63 - 18.00	Su=536/67 psf			34		55x110 mm raw torque readings: V1: 12.0/1.5 ft lbs	
	V2		18.63 - 19.00	Su=692/89 psf			26		V2: 15.5/2.0 ft lbs	
							25			
20	4D V3	24/24	20.00 - 22.00 20.63 - 21.00	WOH/WOH/WOH/HP Su=647/45 psf	---		HP		Grey, wet, medium stiff, Silty CLAY, trace sand, (Glaciomarine). HP=Hydraulic Push, at 20 = 700 lb force. 55x110 mm raw torque readings: V3: 14.5/1.0 ft lbs V4: 15.5/1.0 ft lbs	G#296365 A-6, CL WC=43.9% LL=35 PL=21 PI=14
	V4		21.63 - 22.00	Su=692/45 psf						
							a13		a13 blows for 0.5 ft.	
							59	204.4	24.0	
25	5D	24/9	25.00 - 27.00	3/6/7/9	13	20	35		Brown-grey, wet, medium dense, SAND, trace silt, trace gravel, (Glacial Till).	G#296366 A-1-b, SP-SM WC=18.0%
							46			
							64			
							71			
							89			
30	6D	24/3	30.00 - 32.00	17/17/10/9	27	42	31		Brown, wet, dense, SAND, trace silt, (Glacial Till).	#296367 Testing Canceled
							34			
							61			
							82			
							79	193.9	34.5	
35	7D	24/8	35.00 - 37.00	8/12/17/21	29	45	33		Brown, wet, dense, SAND, some gravel, little silt, (Glacial Till).	G#296368 A-1-b, SM WC=10.8%
							49			
							69			
							57			
							127		b102 blows for 0.5 ft.	
40	8D R1	8.4/1 60/59	40.00 - 40.70 40.70 - 45.70	45/27(2.4") RQD = 57%	---		b102 NQ-2	187.9 187.7	40.5 Top of Bedrock at Elev. 187.9 ft. Spoon Refusal at 40.5 ft bgs. Roller Coned ahead to 40.7 ft bgs. 40.7 R1: Bedrock: Dark grey, fine grained, METASANDSTONE/ SILTSTONE/SHALE, hard, moderately weathered, bedding dips at moderate to steep angles (50 to 70 degrees); breaks occur along fine laminations where sillimanite crystals create weak zones; close to moderately close, open, little sand infilling. Rock Mass Quality = Fair R1: Core Times (min:sec) 40.7-41.7 ft (1:10) 41.7-42.7 ft (2:11) 42.7-43.7 ft (1:50) 43.7-44.7 ft (1:50) 44.7-45.7 ft (2:03) 98% Recovery	
45	R2	60/60	45.70 - 50.70	RQD = 29%				182.7	45.7 R2: Bedrock: Similar to R1, though less noticeable laminae in the bottom 20" and a significant amount of pyrite (about 5%); break in the lower zone occur along or close to the bedding; bedding in the upper zone is distorted, with a near vertical joint; a majority of breaks are along the bedding and smooth. Rock Mass Quality = Poor R2: Core Times (min:sec) 45.7-46.7 ft (1:32) 46.7-47.7 ft (1:31) 47.7-48.7 ft (1:26) 48.7-49.7 ft (1:04) 49.7-50.7 ft (1:25) 100% Recovery	
50								177.7	50.7 Bottom of Exploration at 50.7 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-GDB-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-102

WIN: 23899.00

Driller:	MaineDOT	Elevation (ft.)	228.5	Auger ID/OD:	5" Solid Stem
Operator:	Wilder/Daggett/Niles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/6,9/2018	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	395+91.6, 12.3 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	6.7 ft bgs.
Hammer Efficiency Factor: 0.928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	228.1	4.5" HMA. 0.4	
5	1D	24/18	5.00 - 7.00	13/16/15/13	31	48			Brown, dry, dense, SAND, some gravel, little silt, (Fill).	G#296369 A-1-b, SM WC=5.7%
10	2D/AB	24/22	10.00 - 12.00	2/3/3/4	6	9	3	217.8	2D/A (10.0-10.7 ft) Brown, damp, stiff, Sandy SILT, (Fill). 10.7 2D/B (10.7-12.0 ft) Grey, damp, stiff, Clayey SILT, trace sand, (Glaciomarine). Roller Coned ahead to 12.0 ft bgs. Failed 55x110 mm vane attempt.	G#296370 A-4, ML WC=18.7% G#296371 A-6, CL WC=23.5% LL=35 PL=20 PI=15
							7			
	MV		12.00 - 12.00	Would not Push			10			
							11			
							13			
15	3D V1	24/24	15.00 - 17.00 15.63 - 16.00	WO1P/24" Su=1116/156 psf	---		HP		Light grey, wet, medium stiff, Silty CLAY, trace sand, (Glaciomarine). 55x110 mm raw torque readings: V1: 25.0/3.5 ft lbs V2: 15.0/2.5 ft lbs	G#296372 A-7-6, CL WC=46.0% LL=40 PL=22 PI=18
	V2		16.63 - 17.00	Su=670/112 psf						
20	4D V3	24/24	20.00 - 22.00 20.63 - 21.00	WOR/WOR/WOR/WOR Su=491/45 psf	---				Light grey, wet, soft to medium stiff, SILT, some clay, trace sand, (Glaciomarine). 55x110 mm raw torque readings: V3: 11.0/1.0 ft lbs V4: 11.5/1.0 ft lbs	G#296373 A-4, CL WC=34.8% LL=26 PL=18 PI=8
	V4		21.63 - 22.00	Su=513/45 psf						
25	5D V5	24/24	25.00 - 27.00 25.63 - 26.00	WO2P/24" Su=469/45 psf	---				Light grey, wet, soft to medium stiff, Clayey SILT, trace sand, trace gravel, (Glaciomarine). 55x110 mm raw torque readings: V5: 10.5/1.0 ft lbs V6: 17.0/1.0 ft lbs	G#296374 A-6, CL WC=42.8% LL=35 PL=21 PI=14
	V6		26.63 - 27.00	Su=759/45 psf						
							a32	201.0	27.5 a32 blows for 0.5 ft.	
							179			
							164			
30	6D	24/6	30.00 - 32.00	WOR/5/4/5	9	14	26		Grey to brown, wet, medium dense, SAND, little gravel, trace silt, (Glacial Till).	G#296375 A-1-b, SP-SM WC=17.2%
							38			
							59			
							70			
							59			
35	7D	24/11	35.00 - 37.00	18/16/25/29	41	63	95		Brown, wet, very dense, SAND, some gravel, trace silt, (Glacial Till).	G#296501 A-1-b, SW-SM WC=11.7%
							144			
							190			
							157			
							b67		b67 blows for 0.2 ft. Casing Refusal at 39.2 ft bgs. Roller Coned ahead to 41.2 ft bgs.	
40	8D	24/1	40.00 - 42.00	50(1")	---			187.3	41.2 Bottom of Exploration at 41.2 feet below ground surface. REFUSAL, apparent Bedrock.	
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-GDB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

23899.00

BRIDGE PLANS

WIN 023899.00

BRIDGE NO. 2370

PROJ. MANAGER	Devon Eaton		
	BY	DATE	SIGNATURE
DESIGN-DETAILED	D. Eaton	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	A.VANBUSKIRK	T.WHITE	DEC 2018
DESIGN3-DETAILED3			P.E. NUMBER
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			DATE
REVISIONS 4			
FIELD CHANGES			

GORHAM
HIGGINS BRIDGE
DOUGLAS BROOK
CUMBERLAND COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-101
WIN: 23899.00

Driller:	MaineDOT	**Elevation (ft.)**	228.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/5/2018; 08:30-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	395+43.7, 14.5 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	228.1		4" HMA. 0.3	
5	1D	24/15	5.00 - 7.00	10/11/8/4	19	29	34			Brown, dry, medium dense, SAND, some gravel, little silt, (Fill).	G#296361 A-1-b, SM WC=10.1%
							23				
							9				
							23				
							51				
10	2D	24/20	10.00 - 12.00	4/6/6/5	12	19	37	218.4		10.0 Brown, wet, very stiff, Clayey SILT, trace sand, (Glaciomarine).	G#296362 A-6, CL WC=28.4% LL=33 PL=20 PI=13
							55				
							52				
							56				
							58				
15	3D	24/24	15.00 - 17.00	WOH/WOH/1/2	1	2	33	213.4		15.0 Light brown to grey, wet, soft, Clayey SILT, trace sand, (Glaciomarine). Roller Coned ahead to 17.0 ft bgs.	G#296363 A-6, CL WC=40.1% LL=31 PL=21 PI=10
							45				
	V1		17.63 - 18.00	Su=536/67 psf			34			55x110 mm raw torque readings: V1: 12.0/1.5 ft lbs V2: 15.5/2.0 ft lbs	
	V2		18.63 - 19.00	Su=692/89 psf			26				
							25				
20	4D V3	24/24	20.00 - 22.00 20.63 - 21.00	WOH/WOH/WOH/HP Su=647/45 psf	---		HP			Grey, wet, medium stiff, Silty CLAY, trace sand, (Glaciomarine). HP=Hydraulic Push, at 20 = 700 lb force. 55x110 mm raw torque readings: V3: 14.5/1.0 ft lbs V4: 15.5/1.0 ft lbs	G#296365 A-6, CL WC=43.9% LL=35 PL=21 PI=14
	V4		21.63 - 22.00	Su=692/45 psf							
							a13			a13 blows for 0.5 ft.	
							59	204.4		24.0	
25											

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-101
WIN: 23899.00

Driller:	MaineDOT	**Elevation (ft.)**	228.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/5/2018; 08:30-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	395+43.7, 14.5 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/9	25.00 - 27.00	3/6/7/9	13	20	35			Brown-grey, wet, medium dense, SAND, trace silt, trace gravel, (Glacial Till).	G#296366 A-1-b, SP-SM WC=18.0%
							46				
							64				
							71				
							89				
30	6D	24/3	30.00 - 32.00	17/17/10/9	27	42	31			Brown, wet, dense, SAND, trace silt, (Glacial Till).	#296367 Testing Canceled
							34				
							61				
							82				
							79	193.9		34.5	
35	7D	24/8	35.00 - 37.00	8/12/17/21	29	45	33			Brown, wet, dense, SAND, some gravel, little silt, (Glacial Till).	G#296368 A-1-b, SM WC=10.8%
							49				
							69				
							57				
							127			b102 blows for 0.5 ft.	
40	8D	8.4/1	40.00 - 40.70	45/27(2.4")	---		b102	187.9		40.5 Top of Bedrock at Elev. 187.9 ft.	
	R1	60/59	40.70 - 45.70	RQD = 57%			NQ-2	187.7		Spoon Refusal at 40.5 ft bgs, Roller Coned ahead to 40.7 ft bgs. 40.7	
										R1: Bedrock: Dark grey, fine grained, METASANDSTONE/ SILTSTONE/SHALE, hard, moderately weathered, bedding dips at moderate to steep angles (50 to 70 degrees); breaks occur along fine laminations where sillimanite crystals create weak zones; close to moderately close, open, little sand infilling.	
										Rock Mass Quality = Fair	
										R1: Core Times (min:sec)	
45	R2	60/60	45.70 - 50.70	RQD = 29%						40.7-41.7 ft (1:10) 41.7-42.7 ft (2:11) 42.7-43.7 ft (1:50)	
										43.7-44.7 ft (1:50) 44.7-45.7 ft (2:03)	
										98% Recovery	
										R2: Bedrock: Similar to R1, though less noticeable laminae in the bottom 20" and a significant amount of pyrite (about 5%); break in the	
50										lower zone occur along or close to the bedding; bedding in the upper zone is distorted, with a near vertical joint; a majority of breaks are	

Remarks:

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-101

WIN: 23899.00

Driller:	MaineDOT	**Elevation (ft.)**	228.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/5/2018; 08:30-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	395+43.7, 14.5 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								177.7		along the bedding and smooth. Rock Mass Quality = Poor R2: Core Times (min:sec) 45.7-46.7 ft (1:32) 46.7-47.7 ft (1:31) 47.7-48.7 ft (1:26) 48.7-49.7 ft (1:04) 49.7-50.7 ft (1:25) 100% Recovery 50.7 **Bottom of Exploration at 50.7 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook
Location: Gorham, Maine

Boring No.: BB-GDB-102
WIN: 23899.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/6,9/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	395+91.6, 12.3 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	6.7 ft bgs.
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	228.1		4½" HMA. 0.4	
5	1D	24/18	5.00 - 7.00	13/16/15/13	31	48				Brown, dry, dense, SAND, some gravel, little silt, (Fill).	G#296369 A-1-b, SM WC=5.7%
10	2D/AB	24/22	10.00 - 12.00	2/3/3/4	6	9	3	217.8		2D/A (10.0-10.7 ft) Brown, damp, stiff, Sandy SILT, (Fill). 10.7	G#296370 A-4, ML WC=18.7%
							7			2D/B (10.7-12.0 ft) Grey, damp, stiff, Clayey SILT, trace sand, (Glaciomarine). Roller Coned ahead to 12.0 ft bgs. Failed 55x110 mm vane attempt.	G#296371 A-6, CL WC=23.5% LL=35 PL=20 PI=15
	MV		12.00 - 12.00	Would not Push			10				
							11				
							13				
15	3D	24/24	15.00 - 17.00	WO1P/24"	---		HP			Light grey, wet, medium stiff, Silty CLAY, trace sand, (Glaciomarine). 55x110 mm raw torque readings: V1: 25.0/3.5 ft lbs V2: 15.0/2.5 ft lbs	G#296372 A-7-6, CL WC=46.0% LL=40 PL=22 PI=18
	V1		15.63 - 16.00	Su=1116/156 psf							
	V2		16.63 - 17.00	Su=670/112 psf							
20	4D	24/24	20.00 - 22.00	WOR/WOR/WOR/ WOR	---					Light grey, wet, soft to medium stiff, SILT, some clay, trace sand, (Glaciomarine). 55x110 mm raw torque readings: V3: 11.0/1.0 ft lbs V4: 11.5/1.0 ft lbs	G#296373 A-4, CL WC=34.8% LL=26 PL=18 PI=8
	V3		20.63 - 21.00	Su=491/45 psf							
	V4		21.63 - 22.00	Su=513/45 psf							
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Higgins Bridge #2370 carries State Route 25 over Douglas Brook Location: Gorham, Maine	Boring No.: BB-GDB-102 WIN: 23899.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/6,9/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	395+91.6, 12.3 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	6.7 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D ~~V5~~ V6	24/24	25.00 - 27.00 ~~25.63 - 26.00~~ 26.63 - 27.00	WO2P/24" ~~Su=469/45 psf~~ Su=759/45 psf	---					Light grey, wet, soft to medium stiff, Clayey SILT, trace sand, trace gravel, (Glaciomarine). 55x110 mm raw torque readings: V5: 10.5/1.0 ft lbs V6: 17.0/1.0 ft lbs	G#296374 A-6, CL WC=42.8% LL=35 PL=21 PI=14
							a32	201.0		27.5 a32 blows for 0.5 ft.	
							179				
							164				
30	6D	24/6	30.00 - 32.00	WOR/5/4/5	9	14	26			Grey to brown, wet, medium dense, SAND, little gravel, trace silt, (Glacial Till).	G#296375 A-1-b, SP-SM WC=17.2%
							38				
							59				
							70				
							59				
35	7D	24/11	35.00 - 37.00	18/16/25/29	41	63	95			Brown, wet, very dense, SAND, some gravel, trace silt, (Glacial Till).	G#296501 A-1-b, SW-SM WC=11.7%
							144				
							190				
							157				
							b67			b67 blows for 0.2 ft. Casing Refusal at 39.2 ft bgs. Roller Coned ahead to 41.2 ft bgs.	
40	8D	24/1	40.00 - 42.00	50(1")	---						
								187.3		41.2 **Bottom of Exploration at 41.2 feet below ground surface.** REFUSAL, apparent Bedrock.	
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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Gorham Work Number: 23899.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-GDB-101, 1D	395+43.7	14.5 Lt.	5.0-7.0	296361	1	10.1			SM	A-1-b	II
BB-GDB-101, 2D	395+43.7	14.5 Lt.	10.0-12.0	296362	1	28.4	33	13	CL	A-6	III
BB-GDB-101, 3D	395+43.7	14.5 Lt.	15.0-17.0	296363	1	40.1	31	10	CL	A-6	IV
BB-GDB-101, 4D	395+43.7	14.5 Lt.	20.0-22.0	296365	1	43.9	35	14	CL	A-6	III
BB-GDB-101, 5D	395+43.7	14.5 Lt.	25.0-27.0	296366	2	18.0			SP-SM	A-1-b	0
BB-GDB-101, 6D	395+43.7	14.5 Lt.	30.0-32.0	296367	----	Testing Canceled					
BB-GDB-101, 7D	395+43.7	14.5 Lt.	35.0--37.0	296368	2	10.8			SM	A-1-b	II
BB-GDB-102, 1D	395+91.6	12.3 Rt.	5.0-7.0	296369	3	5.7			SM	A-1-b	II
BB-GDB-102, 2D/A	395+91.6	12.3 Rt.	10.0-10.7	296370	3	18.7			ML	A-4	IV
BB-GDB-102, 2D/B	395+91.6	12.3 Rt.	10.7-12.0	296371	3	23.5	35	15	CL	A-6	III
BB-GDB-102, 3D	395+91.6	12.3 Rt.	15.0-17.0	296372	3	46.0	40	18	CL	A-7-6	III
BB-GDB-102, 4D	395+91.6	12.3 Rt.	20.0-22.0	296373	4	34.8	26	8	CL	A-4	IV
BB-GDB-102, 5D	395+91.6	12.3 Rt.	25.0-27.0	296374	4	42.8	35	14	CL	A-6	III
BB-GDB-102, 6D	395+91.6	12.3 Rt.	30.0-32.0	296375	4	17.2			SP-SM	A-1-b	0
BB-GDB-102, 7D	395+91.6	12.3 Rt.	35.0-37.0	296501	4	11.7			SW-SM	A-1-b	0

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.

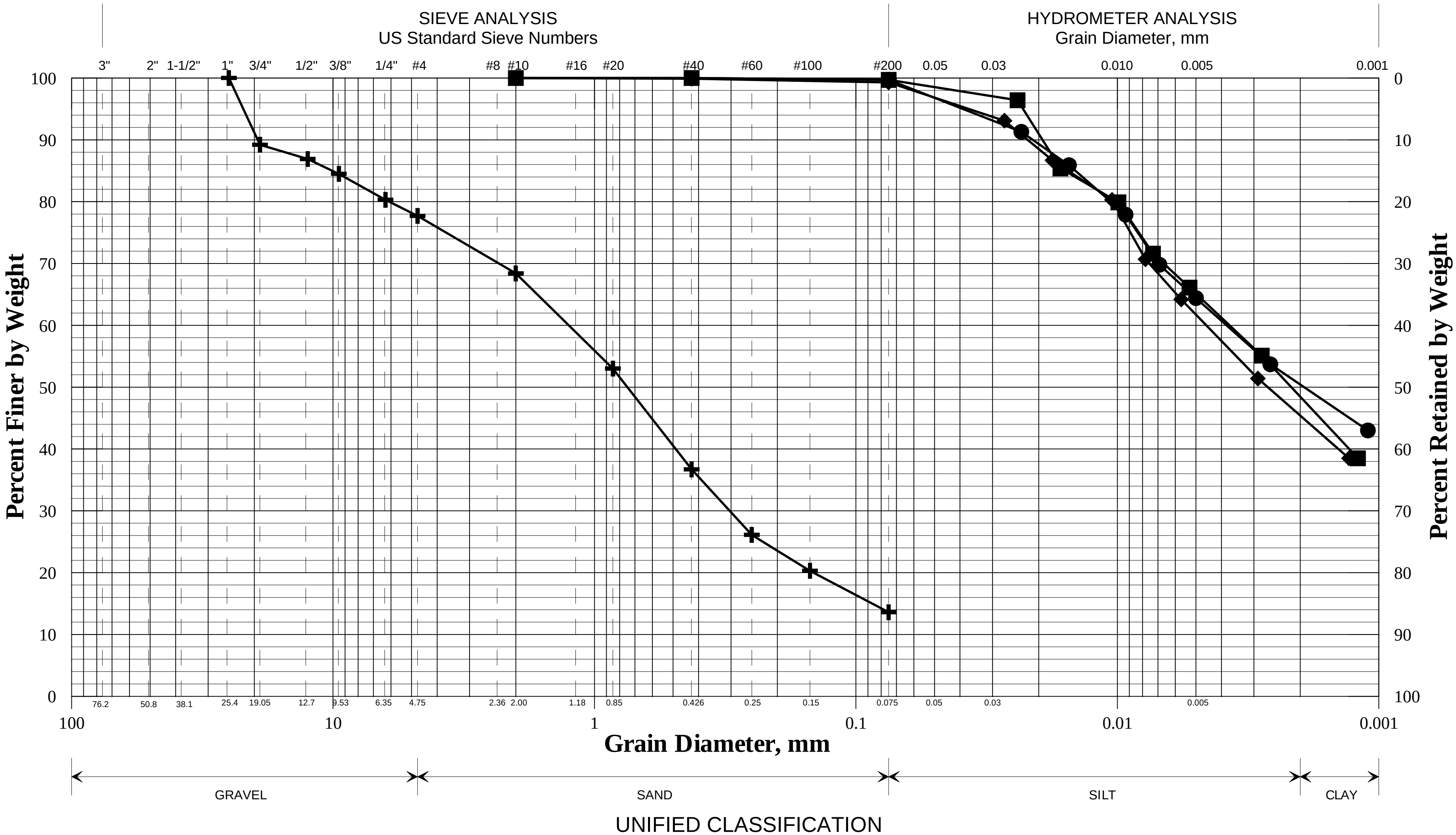
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

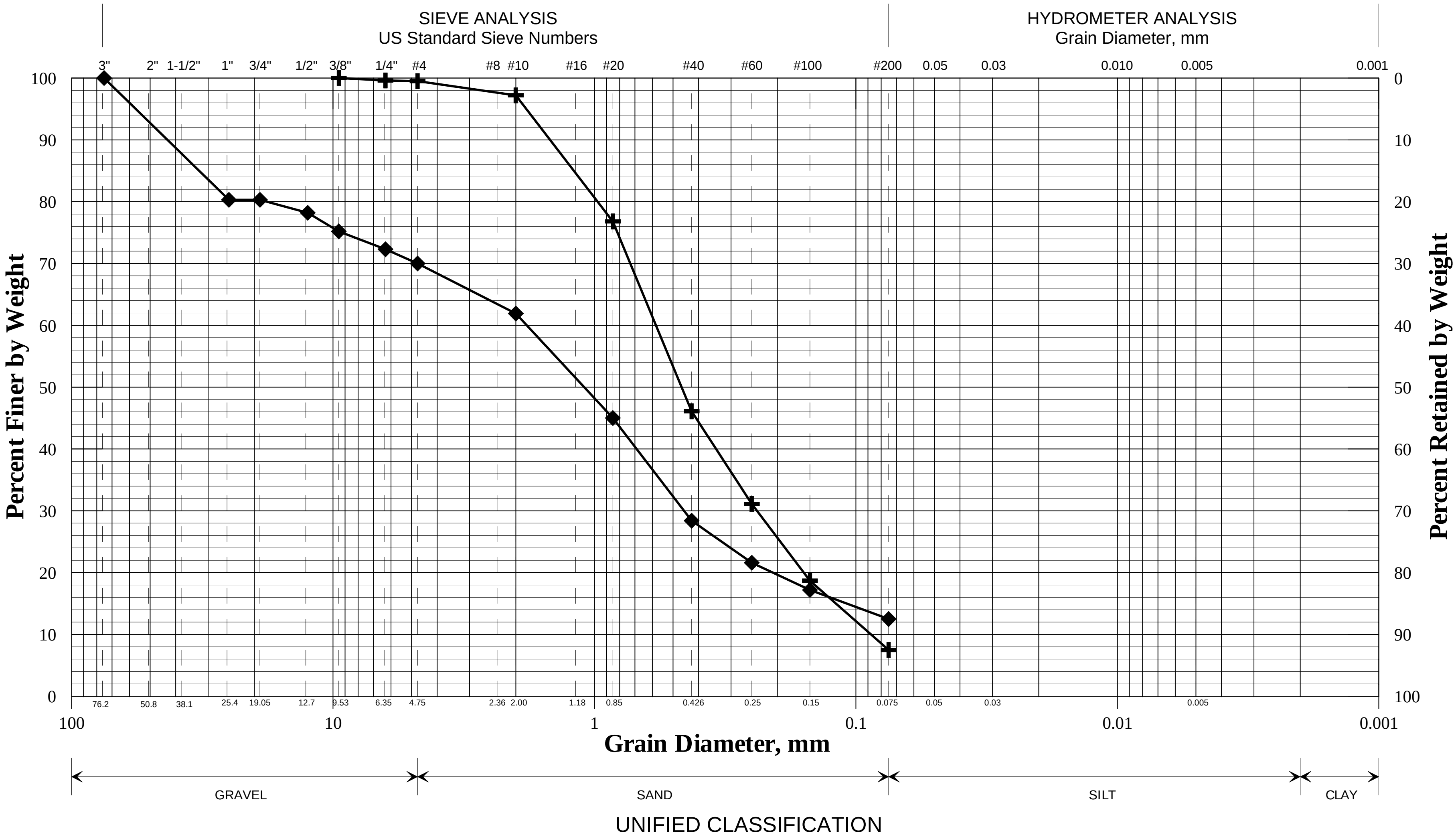


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GDB-101/1D	395+43.7	14.5 LT	5.0-7.0	SAND, some gravel, little silt.	10.1			
◆	BB-GDB-101/2D	395+43.7	14.5 LT	10.0-12.0	Clayey SILT, trace sand.	28.4	33	20	13
■	BB-GDB-101/3D	395+43.7	14.5 LT	15.0-17.0	Clayey SILT, trace sand.	40.1	31	21	10
●	BB-GDB-101/4D	395+43.7	14.5 LT	20.0-22.0	Silty CLAY, trace sand.	43.9	35	21	14
▲									
✖									

WIN
023899.00
Town
Gorham
Reported by/Date
WHITE, TERRY A 6/25/2018

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

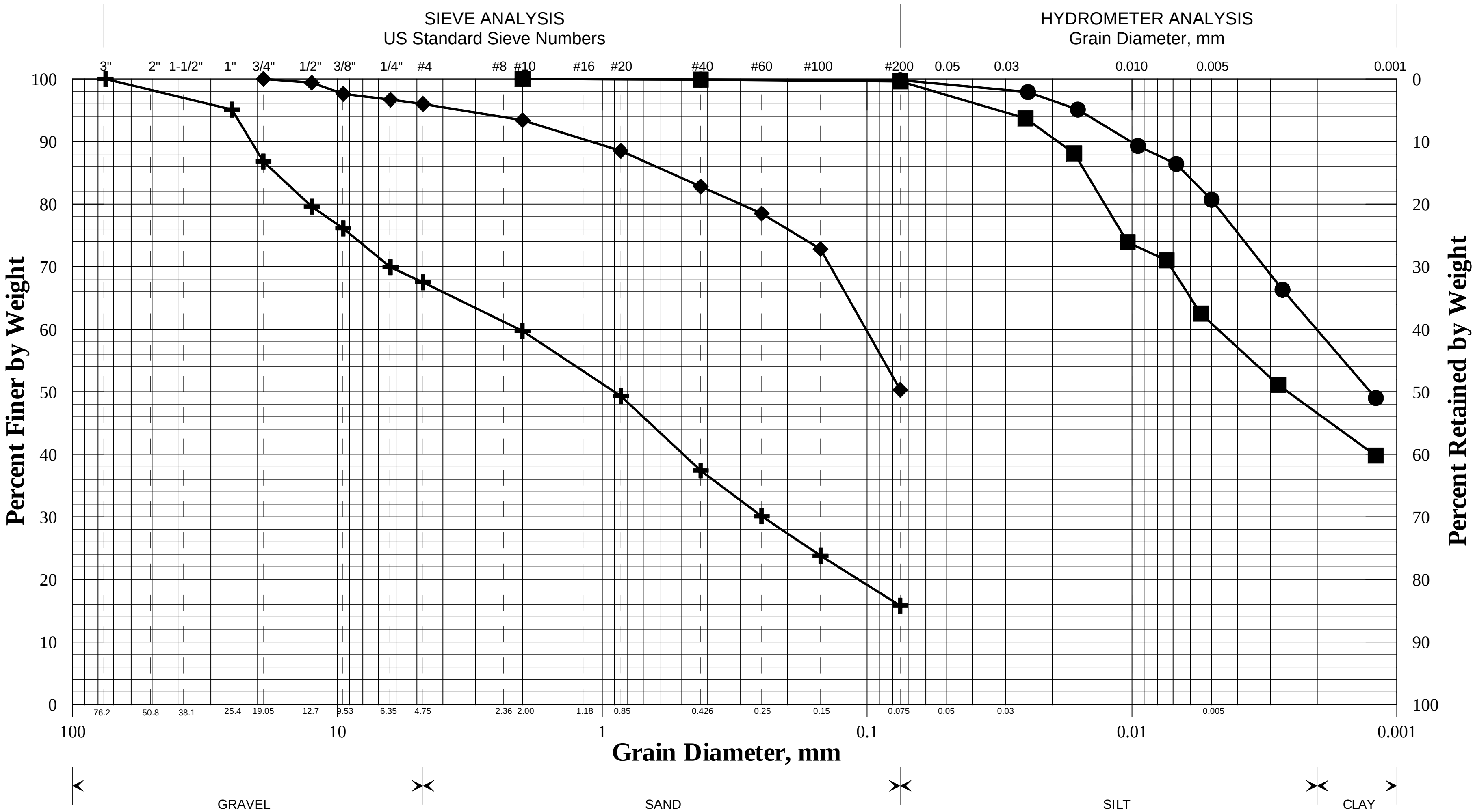


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GDB-101/5D	395+43.7	14.5 LT	25.0-27.0	SAND, trace silt, trace gravel.	18.0			
◆	BB-GDB-101/7D	395+43.7	14.5 LT	35.0-37.0	SAND, some gravel, little silt.	10.8			
■									
●									
▲									
✖									

WIN	
023899.00	
Town	
Gorham	
Reported by/Date	
WHITE, TERRY A	6/25/2018

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

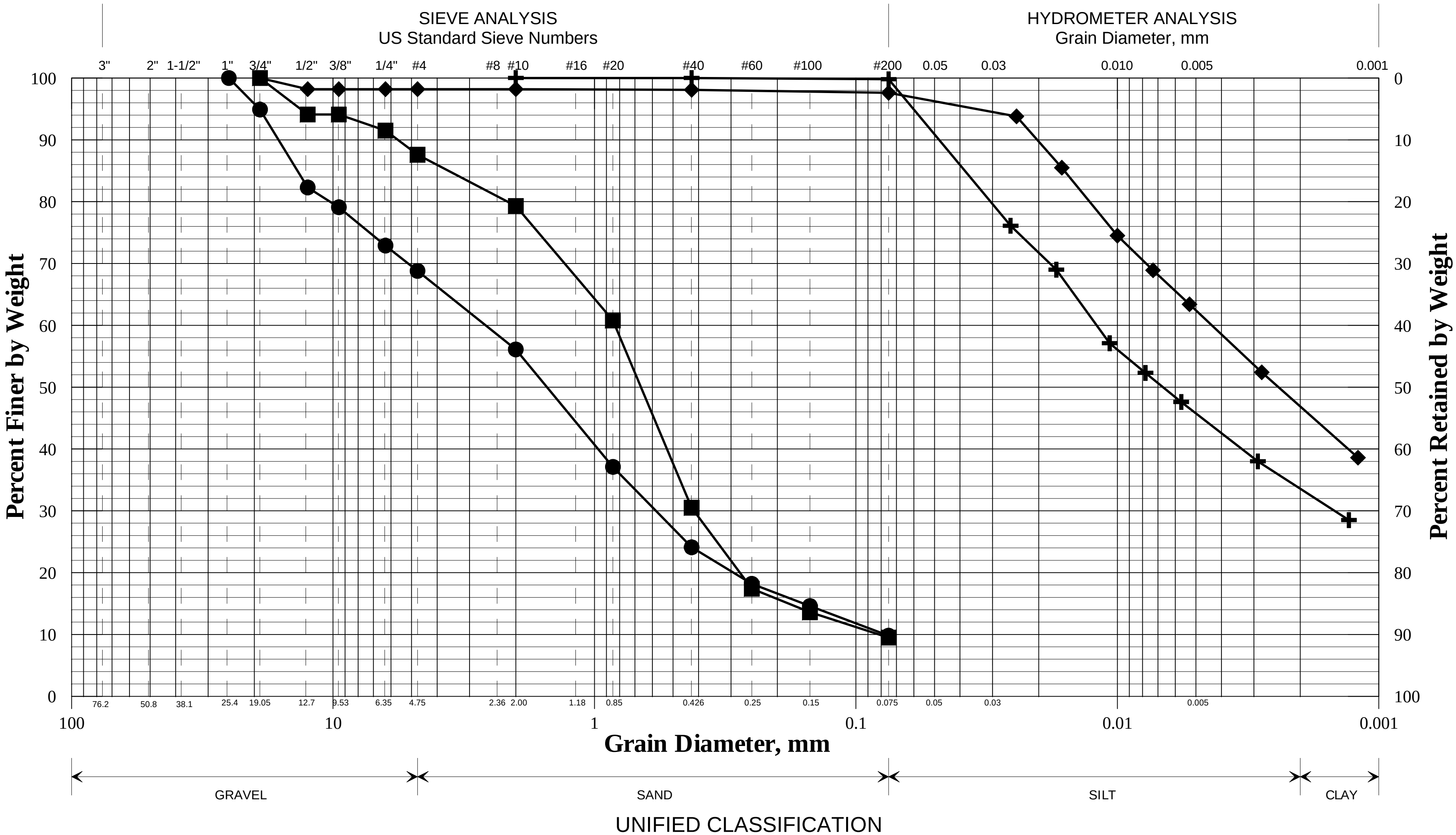
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GDB-102/1D	395+91.6	12.3 RT	5.0-7.0	SAND, some gravel, little silt.	5.7			
◆	BB-GDB-102/2DA	395+91.6	12.3 RT	10.0-10.7	Sandy SILT, trace gravel.	18.7			
■	BB-GDB-102/2DB	395+91.6	12.3 RT	10.7-12.0	Clayey SILT, trace sand.	23.5	35	20	15
●	BB-GDB-102/3D	395+91.6	12.3 RT	15.0-17.0	Silty CLAY, trace sand.	46.0	40	22	18
▲									
✖									

WIN
023899.00
Town
Gorham
Reported by/Date
WHITE, TERRY A 6/25/2018

SHEET 3

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GDB-102/4D	395+91.6	12.3 RT	20.0-22.0	SILT, some clay, trace sand.	34.8	26	18	8
◆	BB-GDB-102/5D	395+91.6	12.3 RT	25.0-27.0	Clayey SILT, trace sand, trace gravel.	42.8	35	21	14
■	BB-GDB-102/6D	395+91.6	12.3 RT	30.0-32.0	SAND, little gravel, trace silt.	17.2			
●	BB-GDB-102/7D	395+91.6	12.3 RT	35.0-37.0	SAND, some gravel, trace silt.	11.7			
▲									
✖									

WIN
023899.00
Town
Gorham
Reported by/Date
WHITE, TERRY A 6/25/2018

SHEET 4

MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296362	BB-GDB-101/2D	GEOTECHNICAL (DISTURBED)	6/8/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+43.7** Offset, ft: **14.5 LT** Dbfg, ft: **10.0-12.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.3
[0.0270 mm]	93.1
[0.0177 mm]	86.7
[0.0105 mm]	80.3
[0.0078 mm]	70.7
[0.0057 mm]	64.2
[0.0029 mm]	51.4
[0.0013 mm]	38.5

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	33
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	13
Specific Gravity, Corrected to 20°C (T 100)	2.58
Loss on Ignition, % (T 267)	
Water Content (T 265), %	28.4

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

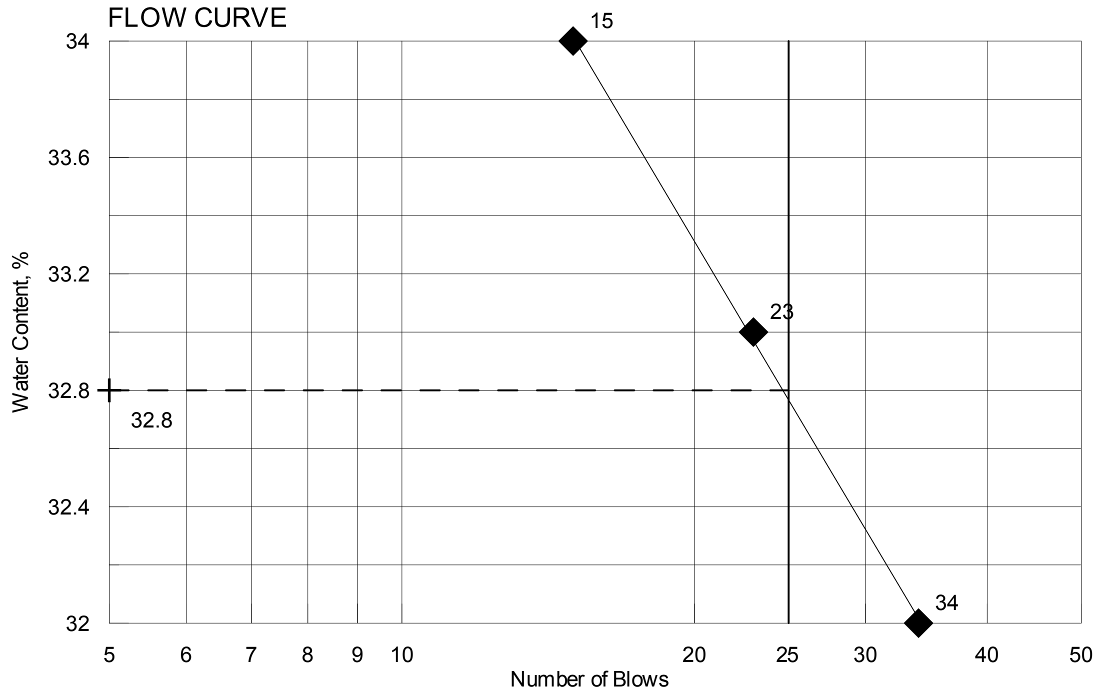
Comments:

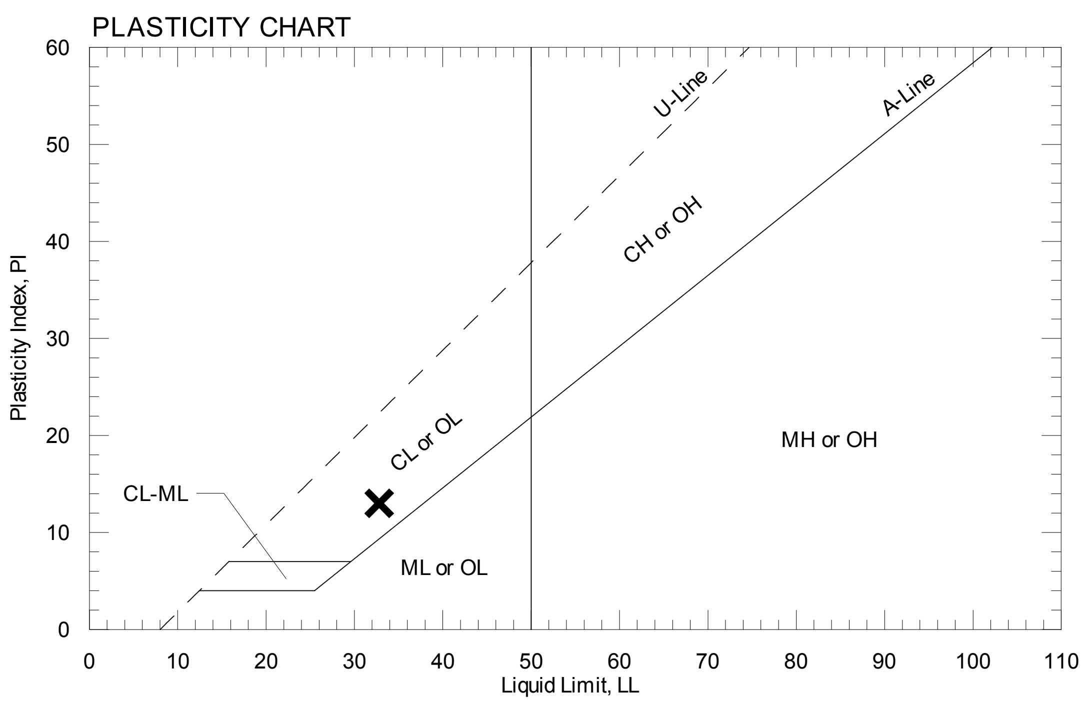
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Gorham	Reference No.	296362
WIN	023899.00	Water Content, %	28.4
Sampled	6/8/2018	Liquid Limit @ 25 blows (T 89), %	33
Boring No./Sample No.	BB-GDB-101/2D	Plastic Limit (T 90), %	20
Station	395+43.7	Plasticity Index (T 90), %	13
Depth	10.0-12.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296363	BB-GDB-101/3D	GEOTECHNICAL (DISTURBED)	6/8/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+43.7** Offset, ft: **14.5 LT** Dbfg, ft: **15.0-17.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0241 mm]	96.4
[0.0165 mm]	85.4
[0.0099 mm]	79.9
[0.0073 mm]	71.6
[0.0053 mm]	66.1
[0.0028 mm]	55.1
[0.0012 mm]	38.5

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	31
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	10
Specific Gravity, Corrected to 20°C (T 100)	2.59
Loss on Ignition, % (T 267)	
Water Content (T 265), %	40.1

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

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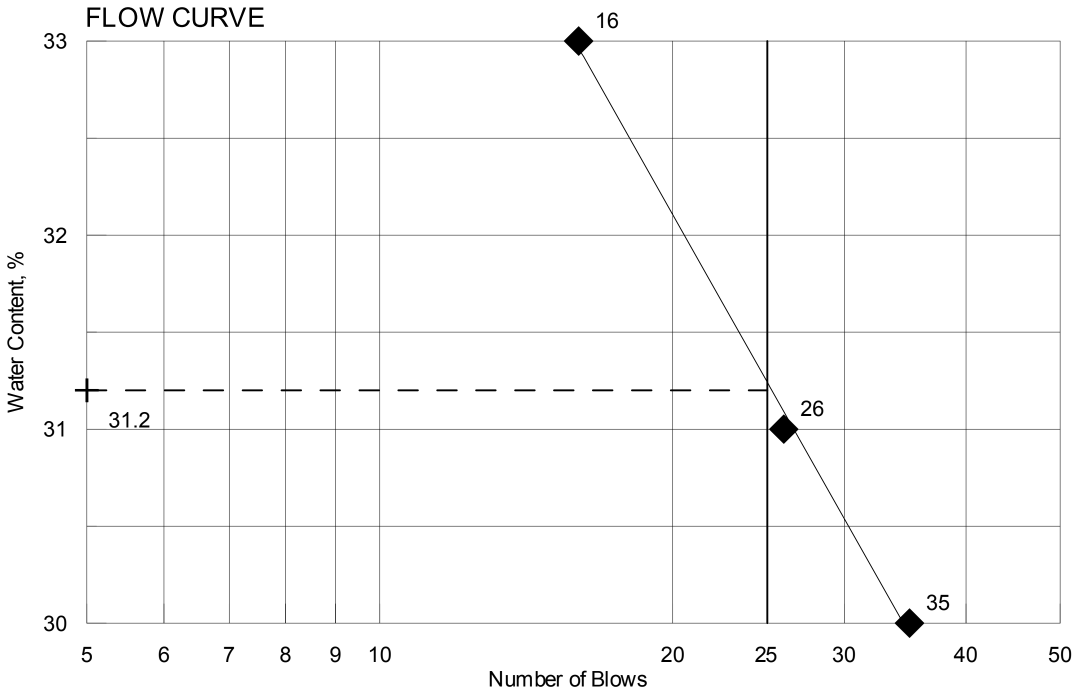
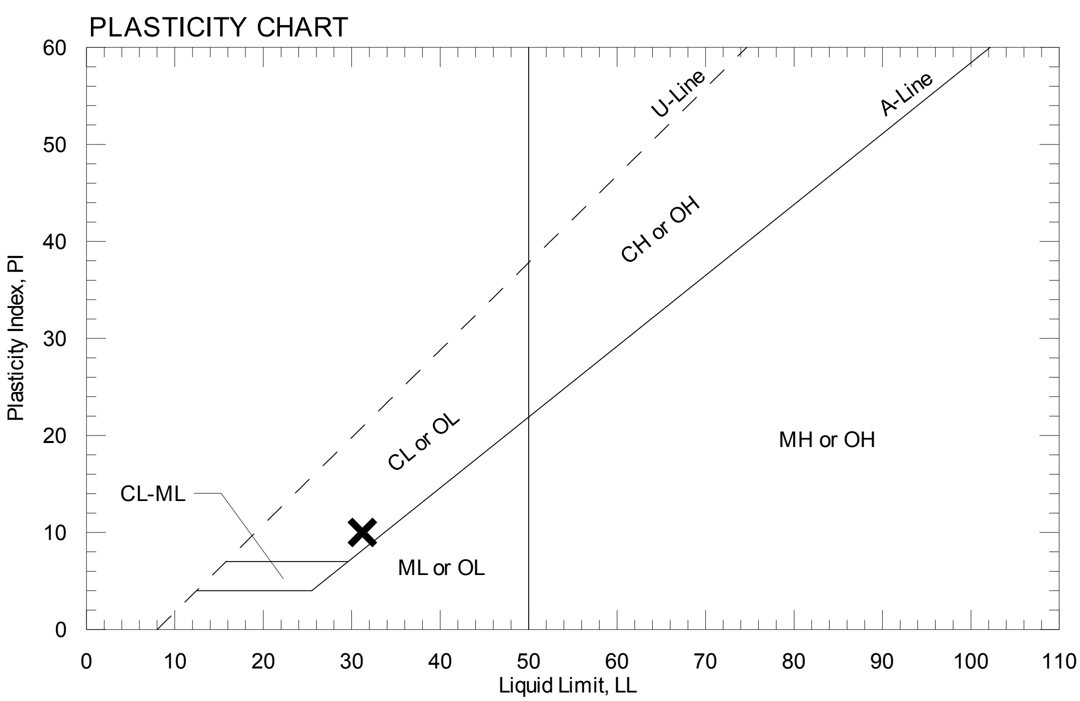
TOWN	Gorham	Reference No.	296363
WIN	023899.00	Water Content, %	40.1
Sampled	6/8/2018	Liquid Limit @ 25 blows (T 89), %	31
Boring No./Sample No.	BB-GDB-101/3D	Plastic Limit (T 90), %	21
Station	395+43.7	Plasticity Index (T 90), %	10
Depth	15.0-17.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296365	BB-GDB-101/4D	GEOTECHNICAL (DISTURBED)	6/8/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+43.7** Offset, ft: **14.5 LT** Dbfg, ft: **20.0-22.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	100.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0233 mm]	91.3
[0.0153 mm]	85.9
[0.0093 mm]	77.9
[0.0069 mm]	69.8
[0.0050 mm]	64.4
[0.0026 mm]	53.7
[0.0011 mm]	43.0

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	35
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	2.78
Loss on Ignition, % (T 267)	
Water Content (T 265), %	43.9

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

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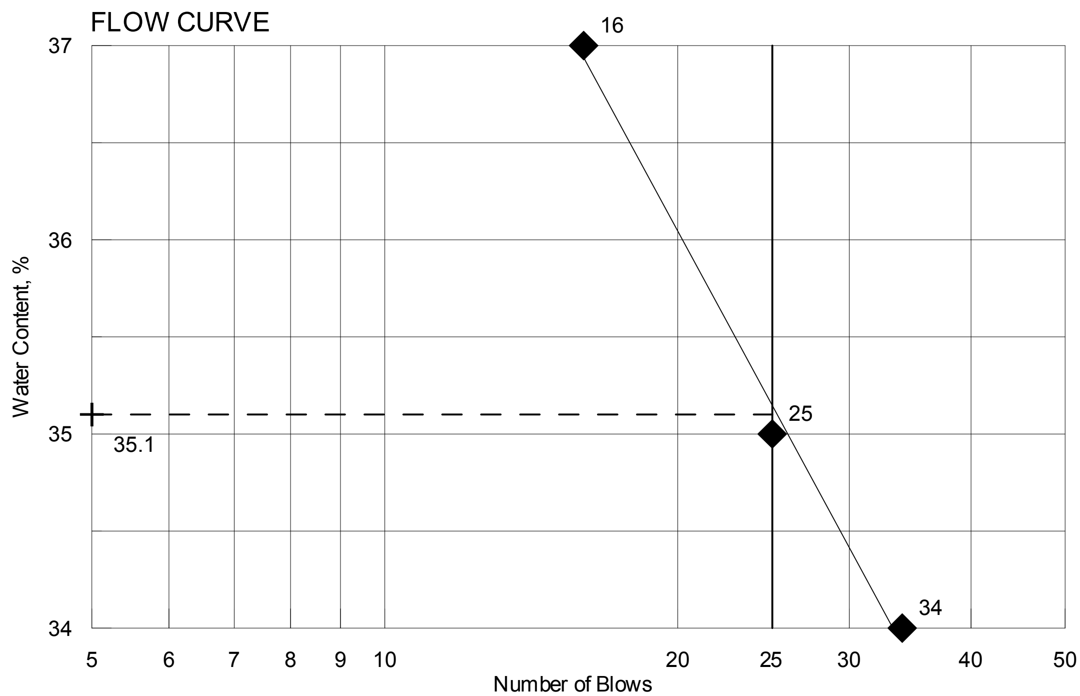
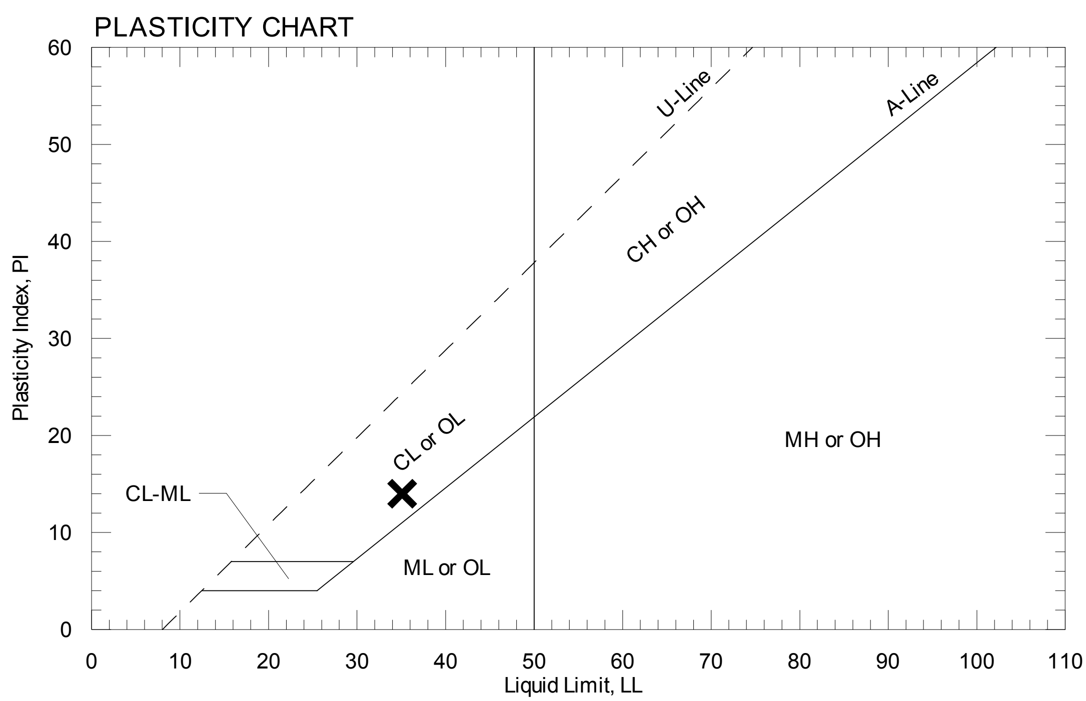
TOWN	Gorham	Reference No.	296365
WIN	023899.00	Water Content, %	43.9
Sampled	6/8/2018	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-GDB-101/4D	Plastic Limit (T 90), %	21
Station	395+43.7	Plasticity Index (T 90), %	14
Depth	20.0-22.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296371	BB-GDB-102/2DB	GEOTECHNICAL (DISTURBED)	6/6/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+91.6** Offset, ft: **12.3 RT** Dbfg, ft: **10.7-12.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.6
[0.0252 mm]	93.7
[0.0165 mm]	88.1
[0.0104 mm]	73.9
[0.0074 mm]	71.0
[0.0055 mm]	62.5
[0.0028 mm]	51.1
[0.0012 mm]	39.8

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	35
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	15
Specific Gravity, Corrected to 20°C (T 100)	2.58
Loss on Ignition, % (T 267)	
Water Content (T 265), %	23.5

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

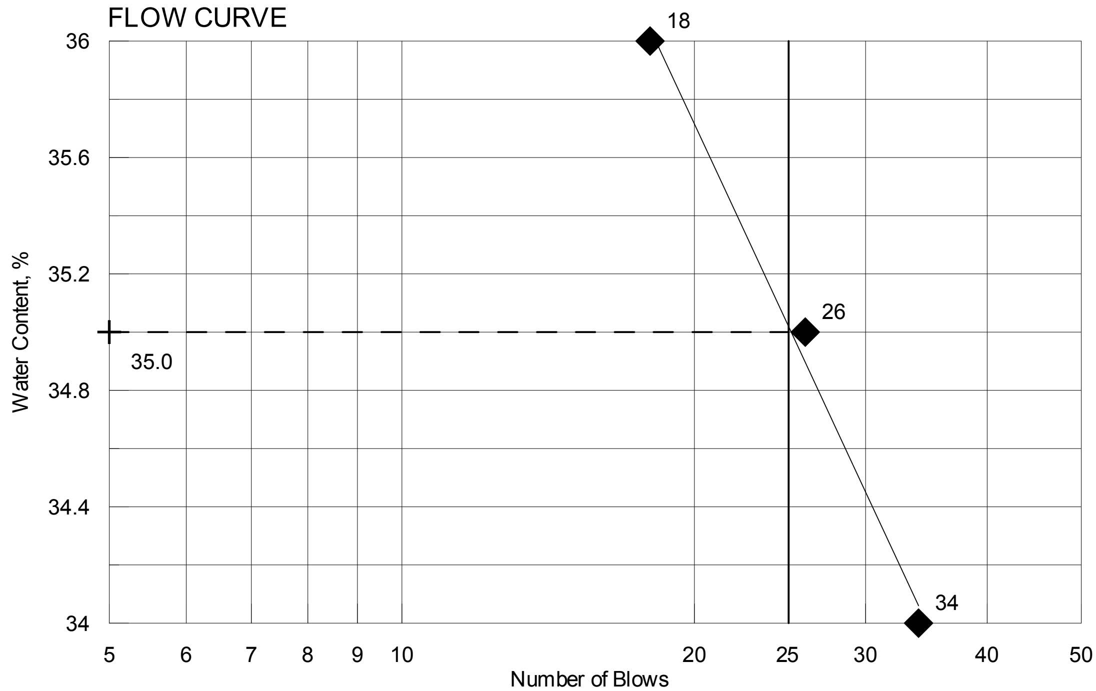
Comments:

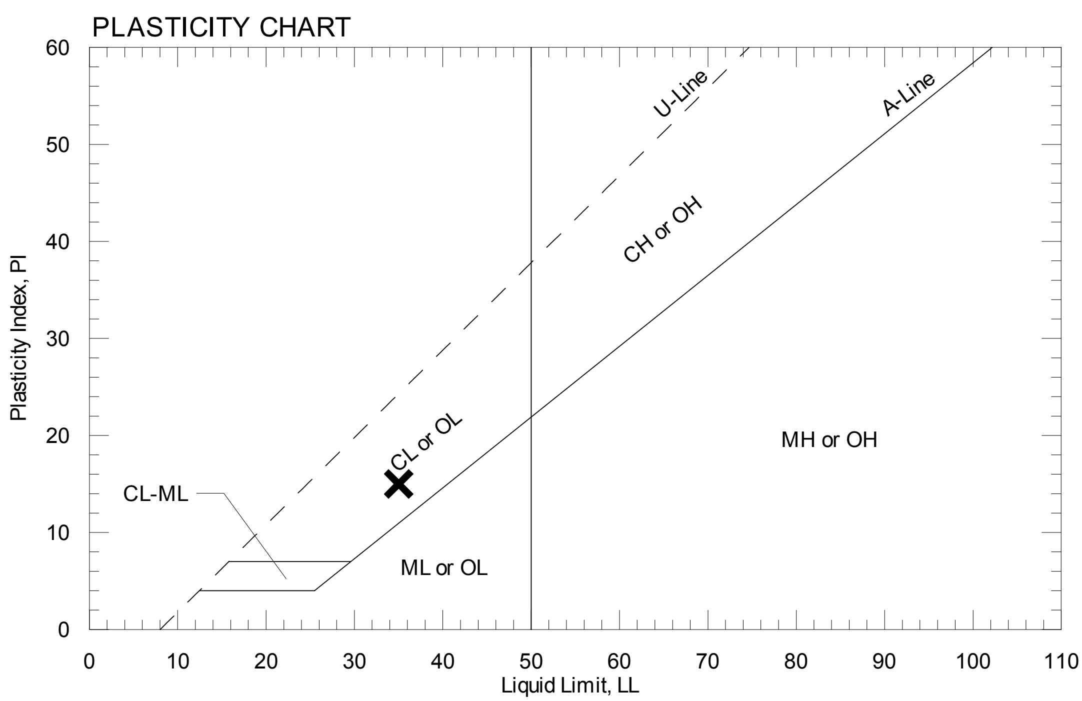
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Gorham	Reference No.	296371
WIN	023899.00	Water Content, %	23.5
Sampled	6/6/2018	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-GDB-102/2DB	Plastic Limit (T 90), %	20
Station	395+91.6	Plasticity Index (T 90), %	15
Depth	10.7-12.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296372	BB-GDB-102/3D	GEOTECHNICAL (DISTURBED)	6/6/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+91.6** Offset, ft: **12.3 RT** Dbfg, ft: **15.0-17.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.8
[0.0247 mm]	97.9
[0.0160 mm]	95.1
[0.0095 mm]	89.3
[0.0068 mm]	86.4
[0.0050 mm]	80.7
[0.0027 mm]	66.3
[0.0012 mm]	49.0

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	40
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	18
Specific Gravity, Corrected to 20°C (T 100)	2.62
Loss on Ignition, % (T 267)	
Water Content (T 265), %	46.0

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

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Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

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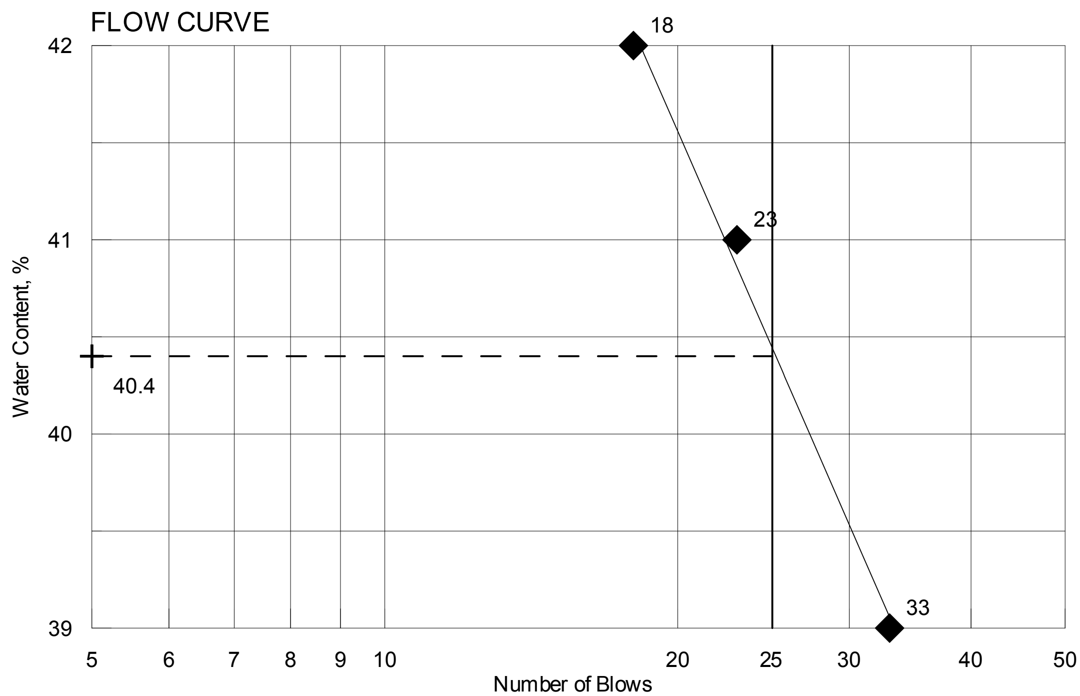
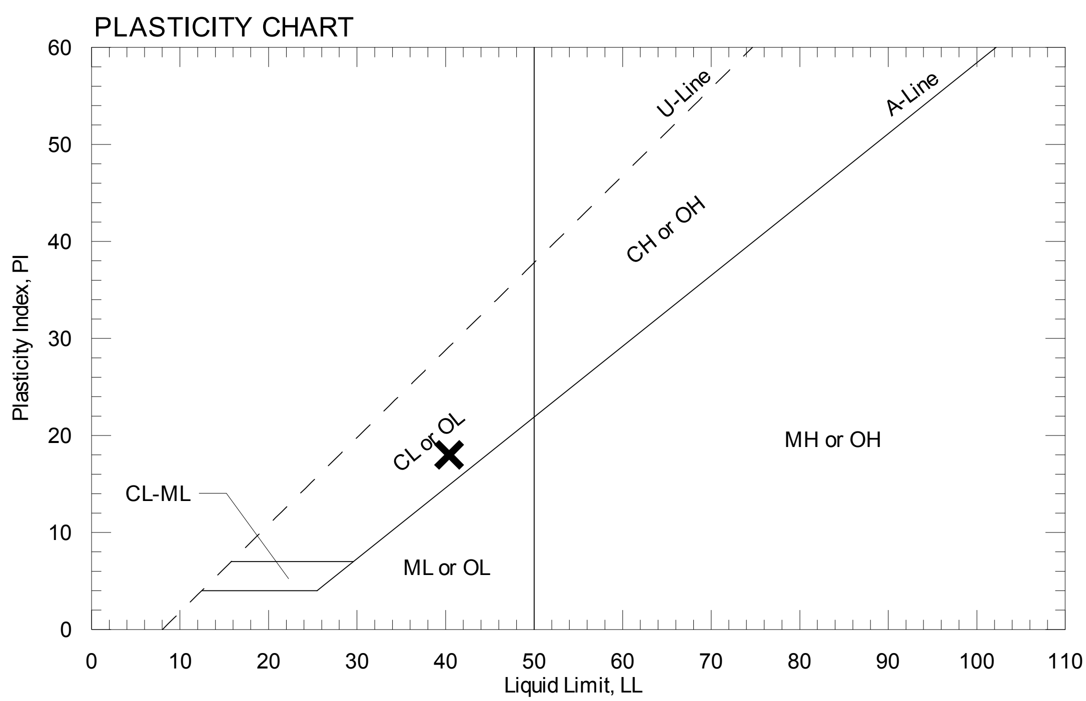
TOWN	Gorham	Reference No.	296372
WIN	023899.00	Water Content, %	46
Sampled	6/6/2018	Liquid Limit @ 25 blows (T 89), %	40
Boring No./Sample No.	BB-GDB-102/3D	Plastic Limit (T 90), %	22
Station	395+91.6	Plasticity Index (T 90), %	18
Depth	15.0-17.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296373	**BB-GDB-102/4D**	**GEOTECHNICAL (DISTURBED)**	**6/6/2018**	**6/19/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **395+91.6** Offset, ft: **12.3 RT** Dbfg, ft: **20.0-22.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	**100.0**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**100.0**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**99.8**
[0.0256 mm]	**76.1**
[0.0171 mm]	**69.0**
[0.0107 mm]	**57.1**
[0.0078 mm]	**52.3**
[0.0057 mm]	**47.6**
[0.0029 mm]	**38.0**
[0.0013 mm]	**28.5**

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**26**
Plastic Limit (T 90), %	**18**
Plasticity Index (T 90), %	**8**
Specific Gravity, Corrected to 20°C (T 100)	**2.59**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**34.8**

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

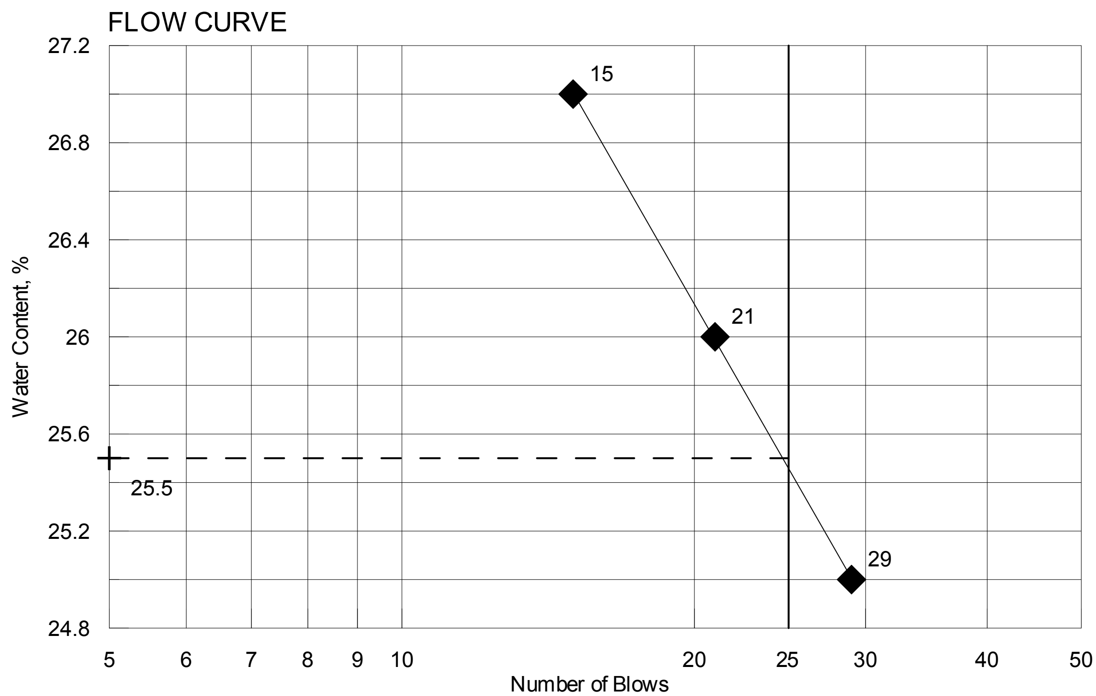
Comments:

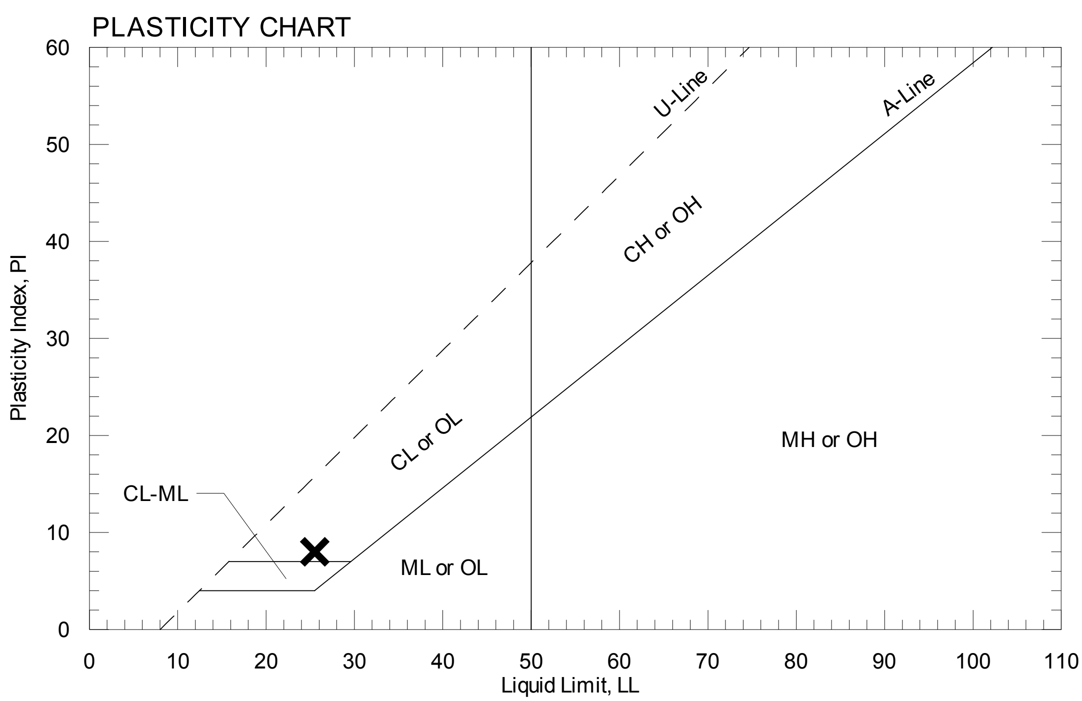
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Gorham	Reference No.	296373
WIN	023899.00	Water Content, %	34.8
Sampled	6/6/2018	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-GDB-102/4D	Plastic Limit (T 90), %	18
Station	395+91.6	Plasticity Index (T 90), %	8
Depth	20.0-22.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
296374	BB-GDB-102/5D	GEOTECHNICAL (DISTURBED)	6/6/2018	6/19/2018

Sample Type: **GEOTECHNICAL** Location: Station: **395+91.6** Offset, ft: **12.3 RT** Dbfg, ft: **25.0-27.0**

WIN/Town **023899.00 - GORHAM** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	98.2
⅜ in. [9.5 mm]	98.2
¼ in. [6.3 mm]	98.2
No. 4 [4.75 mm]	98.2
No. 10 [2.00 mm]	98.2
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	98.1
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	97.6
[0.0243 mm]	93.8
[0.0163 mm]	85.5
[0.0100 mm]	74.5
[0.0073 mm]	68.9
[0.0053 mm]	63.4
[0.0028 mm]	52.4
[0.0012 mm]	38.6

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	35
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	2.64
Loss on Ignition, % (T 267)	
Water Content (T 265), %	42.8

Consolidation (T 216)

Trimmings, Water Content, %

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

Vane Shear Test on Shelby Tubes (Maine DOT)

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

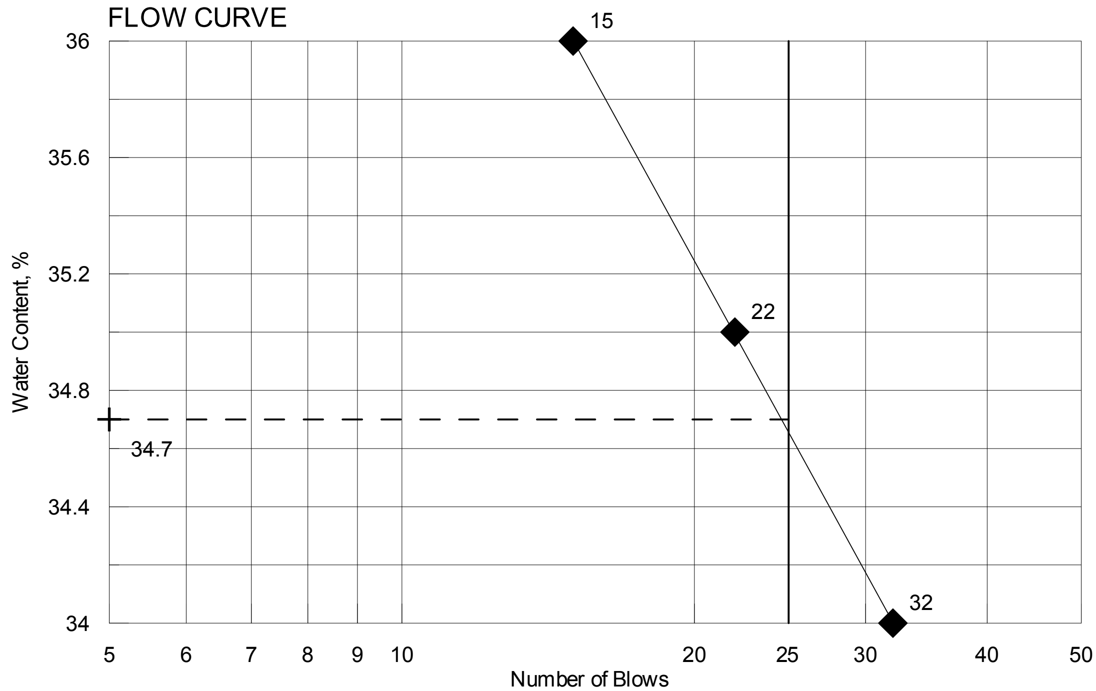
Comments:

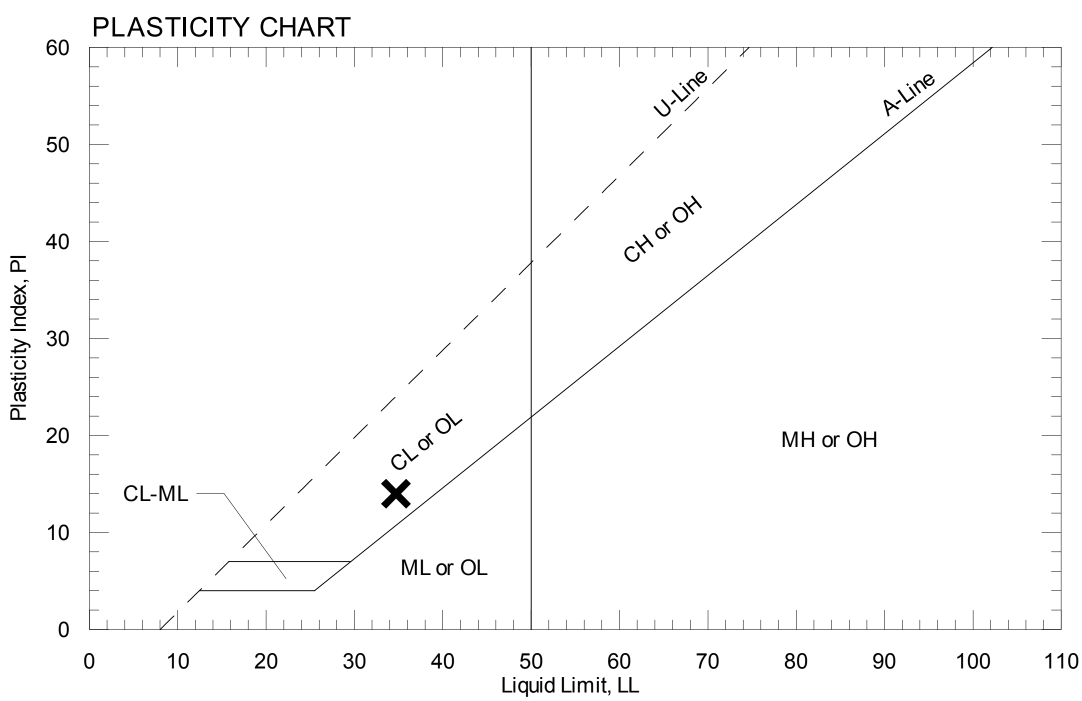
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **6/22/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Gorham	Reference No.	296374
WIN	023899.00	Water Content, %	42.8
Sampled	6/6/2018	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-GDB-102/5D	Plastic Limit (T 90), %	21
Station	395+91.6	Plasticity Index (T 90), %	14
Depth	25.0-27.0	Tested By	BBURR





Appendix C

Calculations

Earth Pressure

Earth Pressure

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

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

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

Cohesion $c := 0 \cdot psf$

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

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

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

$$K_o = 0.47$$

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

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

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

For cantilver walls with horizontal backslope:

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

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

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

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

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

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

6.1 AT-REST LATERAL PRESSURES

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

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

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

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

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

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

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

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

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

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

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

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

[a] Unloading cycle.

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

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

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

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

6.2.1 Active Pressure

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

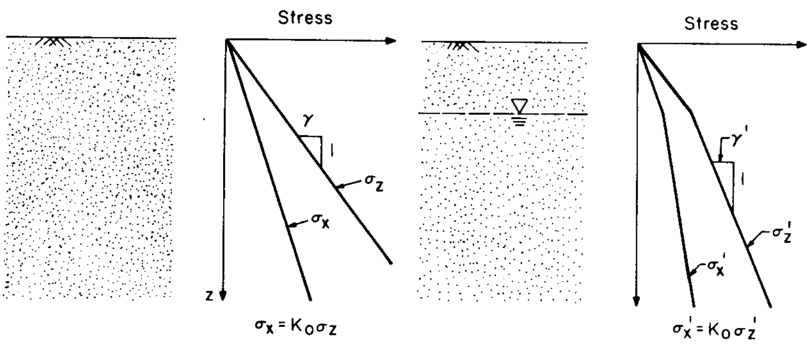


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Figure 3-2 Calculating β with Broken Backfill Surface

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

3.6.5.2 Rankine Theory

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

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

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

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

where:

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

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

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

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

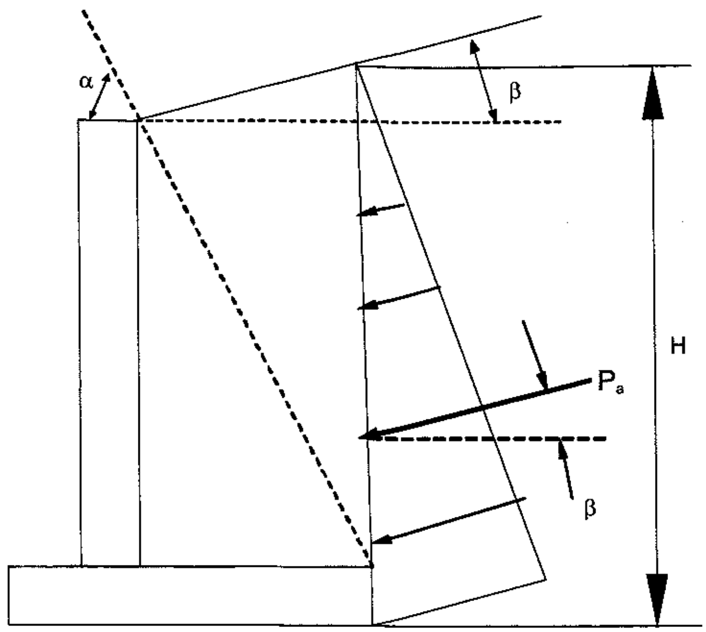


Figure 3-3 Rankine Theory

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

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

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

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

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

where:

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

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

Bearing Resistance

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

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

Assumptions:
1. The box culvert's embedment into the streambed is conservatively assumed as 2 feet. Include 2 foot crushed stone mat as Foundation Footing, therefore Foundation Depth is 2.0 feet
2. Crushed stone considered part of base slab
3. The proposed bearing elevation slopes from El. 215.0 at inlet to 214.5 at outlet.
4. Bearing elevation of crushed stone mat is El. 213.0 at inlet and 212.5 at outlet.
5. Proposed finish roadway grade elevation is approximately 228.9 feet.
6. Proposed precast concrete box has a span of 21 feet and a base of 23 feet wide. Use dimensions of geogrid reinforced crushed stone mat as the Foundation Width, 23 + 3 = 26 feet
7. The bottom of the box culvert will be submerged for the structure's design life.

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

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

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Inorganic clay, sandy or silty clay (CL)	Loose Medium Dense to Dense	1-2 2-6	1 4

The Marine Clay (CL) deposit is generally soft to medium stiff in consistency. 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

$$B := 26ft$$

$$D_f := 4.0 \cdot ft$$

$$D_w := 0 \cdot ft$$

$$\gamma_w := 62.4 \cdot pcf$$

Foundation soils:

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

$w_{sat} := 46.0\%$ — Highest moisture contents of saturated samples near bearing elevation: BB-GDB-102 3D.

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

$\gamma 1_{sat} = 121.2 \cdot pcf$

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

Cohesion $c_1 := 536 \cdot psf$ — BB-GDB-101 V1

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

Shape Factors for strip footing

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

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

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

Nominal Bearing Resistance per Terzaghi equation

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $\quad q = 0.235 \cdot ksf$ — 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

$q_n = 3 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.50$

$q_r := q_n \cdot \varphi_b$

$\boxed{q_r = 1.5 \cdot ksf}$

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

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

MaineDOT Bridge Design Guide p. 3-3
Soil Type 4

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

$N_c := 5.14$

$N_q := 1$

$N_\gamma := 0$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume:

$L := 69ft$

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

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

$s_\gamma = 0.849$ $\qquad$ $s_q = 1$

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.

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

Load Inclination factors

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

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

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

$$\frac{D_f}{B} = 0.154$$

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)

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

$$q_n = 3 \cdot ksf$$

Factored Bearing Resistance

$$\phi_b := 0.50$$

$q_r := q_n \cdot \phi_b$ $\boxed{q_r = 1.5 \cdot ksf}$

Recommend a factored bearing resistance of 1.5 ksf for precast box culvert constructed on 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	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

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

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

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

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

in which:

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

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

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

where:

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

C10.6.3.1.2a

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 though 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh, et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

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.

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

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

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.

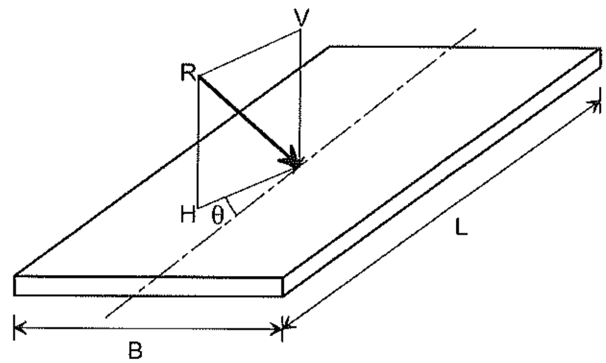


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

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.

Modulus of Subgrade Reaction

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

Given:
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 neglected based on sensitivity analysis.
3. The proposed bearing elevation of base slab slopes from Elev. 215.0 at inlet to 214.5 at outlet.
4. Proposed finished grade matches existing.
5. Proposed precast concrete box is 23 feet wide and 69 feet long (excluding slab connecting slope tapered outlet and inlet sections).
6. Model subsurface conditions based on upper Marine Clay deposit, Su=536 to 670 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

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

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

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} = 29 pci

Adjust Published values for dimensions of base slab

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

Assume a subgrade modulus of 40 pci for 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 = 23 ft, Length L = 69 ft) and dimensions of crushed stone bedding layer (Width B = 23, Length L = 69). Check for sensitivity in the change of Width B.

Square to rectangle base adjustment:

$$k_{s1} := 40\text{pci} \qquad B := 23\text{ft} \qquad L := 69\text{ft}$$

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

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

$$k = 31.1 \cdot \text{pci}$$

Recommend a subgrade modulus of 31 pci

for either a horizontal or lateral modulus of subgrade reaction is

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

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

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

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

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

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

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

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

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

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

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

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

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

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

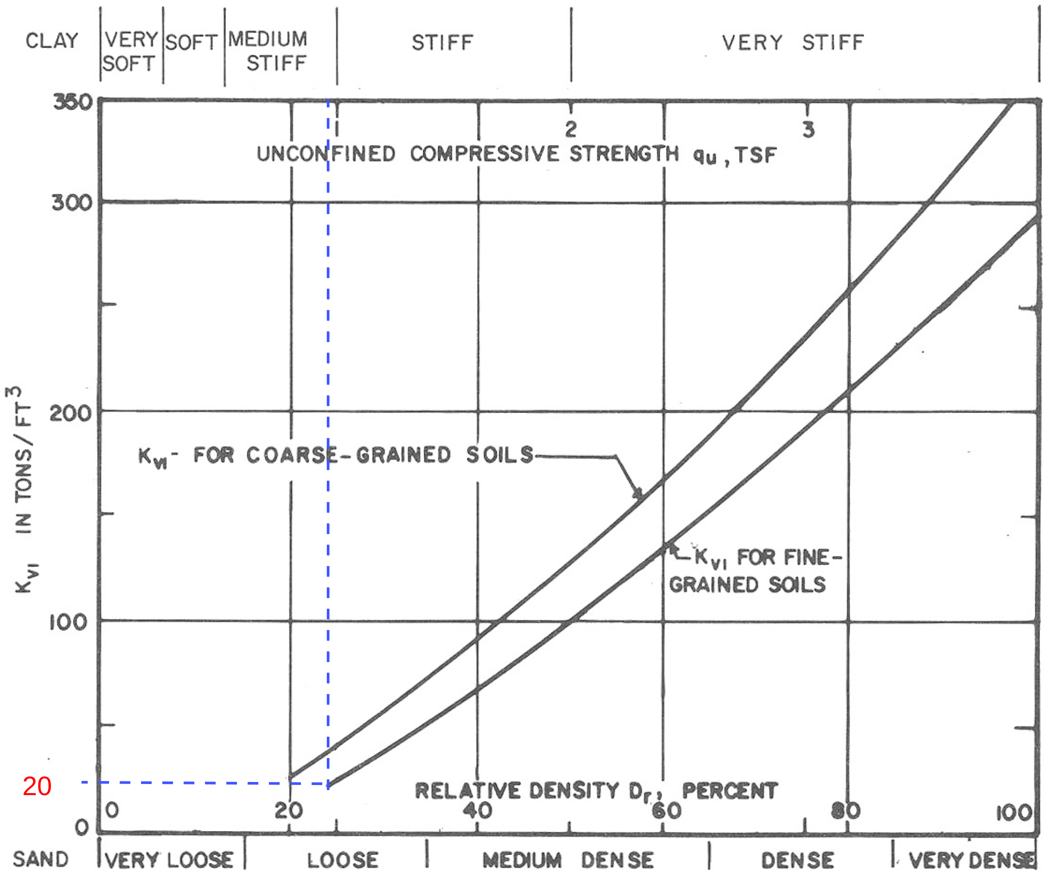
You should note that k_s does not depend on the contact pressure of the base q_o.

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)]

Clay Subgrade - lowest value per chart is 20 tcf = 23 pci



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING

q = FOOTING UNIT LOAD IN tsf

B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE

K_{VI} = MODULUS OF VERTICAL SUBGRADE REACTION

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

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

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

FOR $B \geq 40$ FT:

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

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

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

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

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

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

 Das, Priciples of Foundation Engineering 7th Ed.

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

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

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

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

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

or

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

where

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

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

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

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

Terzaghi, Geotechnique, Vol. 5 No. 4

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

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

EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION

General procedure

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

Vertical subgrade reaction

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

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

Table 1.

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

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

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

Foundations on Sandy Soils

For foundations on sandy soils,

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

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

Foundations on Clays

For foundations on clays,

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

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

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

Method 1:

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

where

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

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

Settlement

Settle3D Analysis Information
Gorham Higgins Bridge

Project Settings

Document Name	23899 q=500; upper limit.s3z
Project Title	Gorham Higgins Bridge
Analysis	Immediate, 1-year consolidation and secondary compression
Author	Van Buskirk/Krusinski
Company	MaineDOT
Date Created	10/19/2018, 1:21:00 PM
Comments	
22' span - L=68 ft, B=24 ft	
Net q = 500 psf	
1 stiff 1 soft Clayey Silt Subunits	
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	

Stage Settings

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

Results

Time taken to compute: 0.932613 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.23114
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	1.23114
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	3.03226
Total Stress [ksf]	0	5.4198
Total Strain	0	0.00870549
Pore Water Pressure [ksf]	0	2.38754
Excess Pore Water Pressure [ksf]	0	0.5
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.00875	3.67209
Over-consolidation Ratio	1	2.5
Void Ratio	0	1.1
Permeability [ft/y]	0	0.0166464
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.653023

Stage: Consolidation = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.37317
Consolidation Settlement [in]	-0.00184338	0.142027
Immediate Settlement [in]	0	1.23114
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	3.11369
Total Stress [ksf]	0	5.4198
Total Strain	-0.00105799	0.0111255
Pore Water Pressure [ksf]	0	2.53669
Excess Pore Water Pressure [ksf]	0	0.377649
Degree of Consolidation [%]	0	73.4371
Pre-consolidation Stress [ksf]	0.00875	3.67209
Over-consolidation Ratio	1	2.73467
Void Ratio	0	1.10064
Permeability [ft/y]	0	0.0166464
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	60.472
Undrained Shear Strength	0	0.656952

Stage: Long-term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.22099
Consolidation Settlement [in]	0	0.34491
Immediate Settlement [in]	0	1.23114
Secondary Settlement [in]	0	0.648249
Loading Stress [ksf]	0	0.5
Effective Stress [ksf]	0	3.26076
Total Stress [ksf]	0	5.4198
Total Strain	0	0.0171708
Pore Water Pressure [ksf]	0	2.15904
Excess Pore Water Pressure [ksf]	0	2.17564e-009
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.00875	3.67209
Over-consolidation Ratio	1	2.5
Void Ratio	0	1.1
Permeability [ft/y]	0	0.0166464
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.67681

Loads

1. Rectangular Load

Length	24 ft
Width	68 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1632 ft^2
Load	0.5 ksf
Depth	14 ft
Installation Stage	Immediate = 0 y

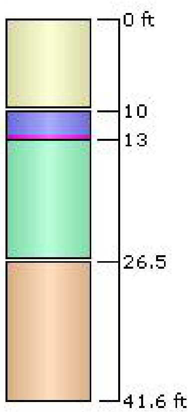
Coordinates

X [ft]	Y [ft]
-5.566	-19.309
18.434	-19.309
18.434	48.691
-5.566	48.691

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Fill	10	0	No
2	Stiff Clayey Silt	3	10	No
3	Upper Soft Clayey Silt	0	13	No
4	Soft Clayey Silt	13.5	13	No
5	Glacial Outwash	15.1	26.5	No



Soil Properties

Property	Fill	Glacial Outwash	Stiff Clayey Silt	Upper Soft Clayey Silt	Soft Clayey Silt
Color					
Unit Weight [kips/ft^3]	0.125	0.115	0.108	0.083	0.083
Saturated Unit Weight [kips/ft^3]	0.115	0.128	0.135	0.121	0.121
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled
Es [ksf]	450	8000	146.198	146.198	57.435
Esur [ksf]	1800	32000	480	208.9	208.9
Primary Consolidation	Disabled	Disabled	Enabled	Enabled	Enabled
Material Type			Non-Linear	Non-Linear	Non-Linear
Cc			0.164	0.4	0.4
Cr			0.0164	0.04	0.04
e0			1.1	0.9	0.9
OCR	1	1	2.5	2	1.8
Cv [ft^2/y]			73	36.5	36.5
B-bar			1	1	1
Secondary Consolidation	Disabled	Disabled	Standard	Standard	Standard
Cae/Ca			0.01	0.01	0.011
Car/Care			0.01	0.01	0.01
Undrained Su A [kips/ft2]	0	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1	1

Groundwater

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

Piezometric Line Entities

ID	Depth (ft)
1	7 ft

Query Points

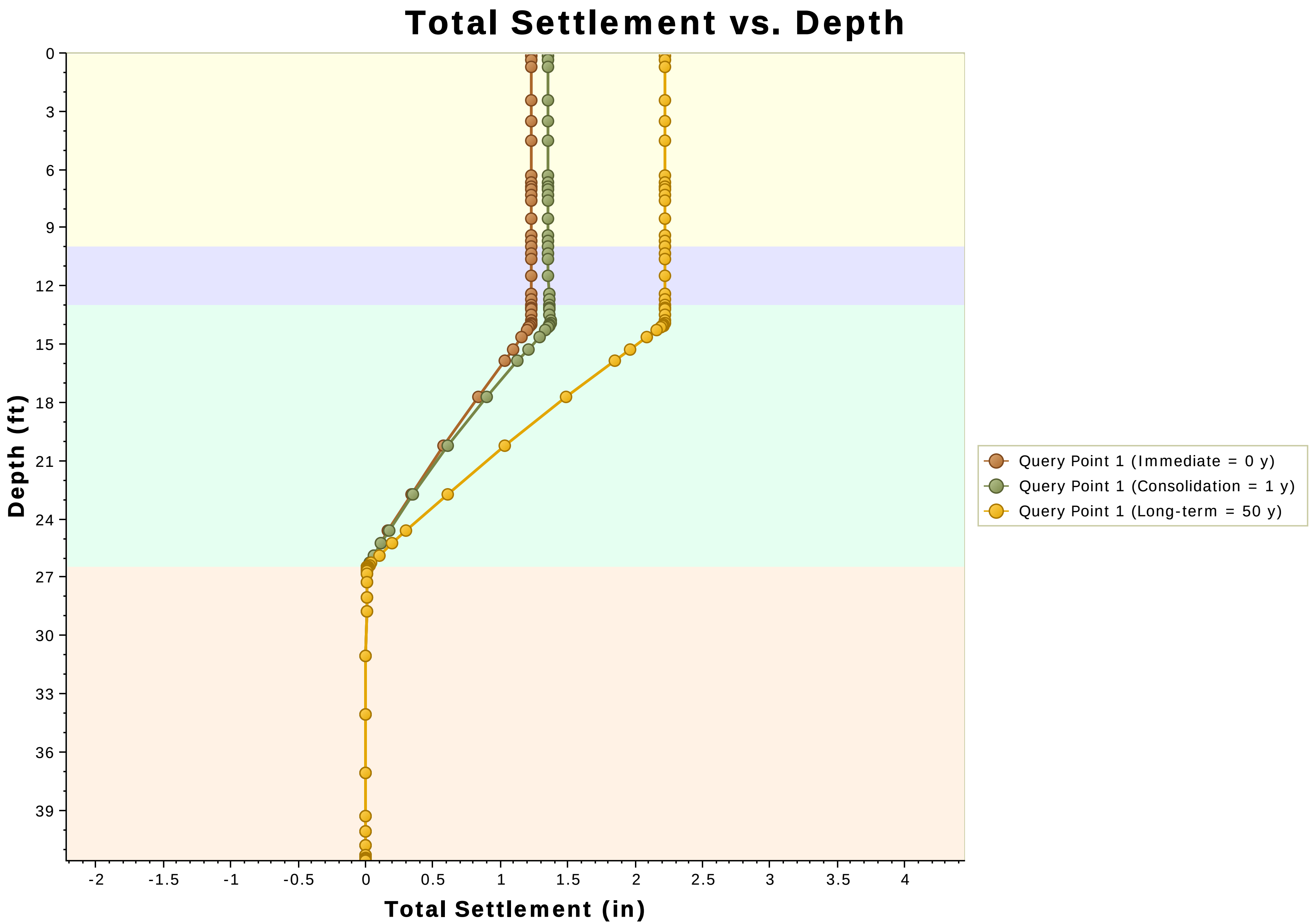
Point #	(X,Y) Location	Number of Divisions
1	6.434, 14.691	Auto: 65

Field Point Grid

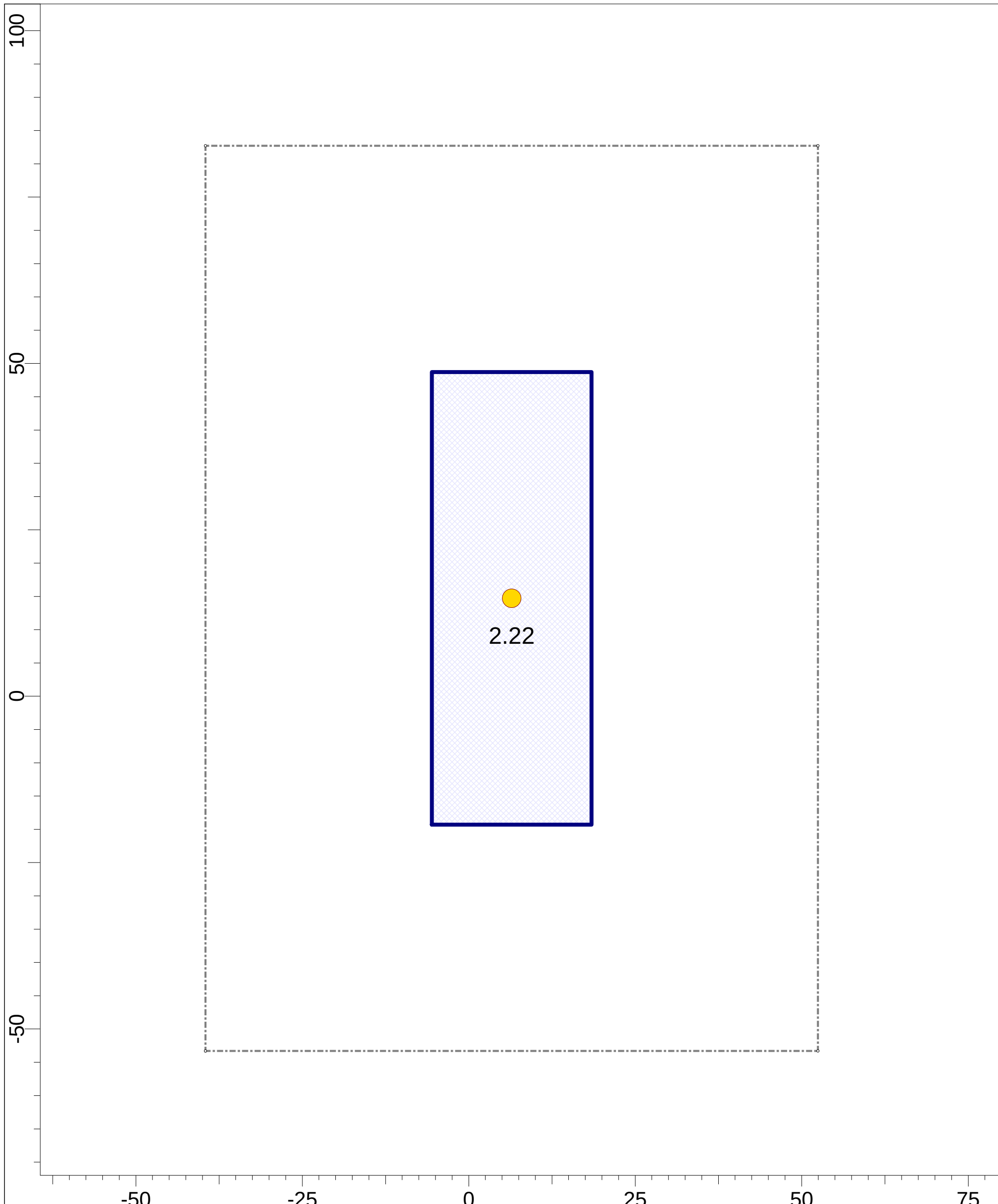
Number of points	294
Expansion Factor	2

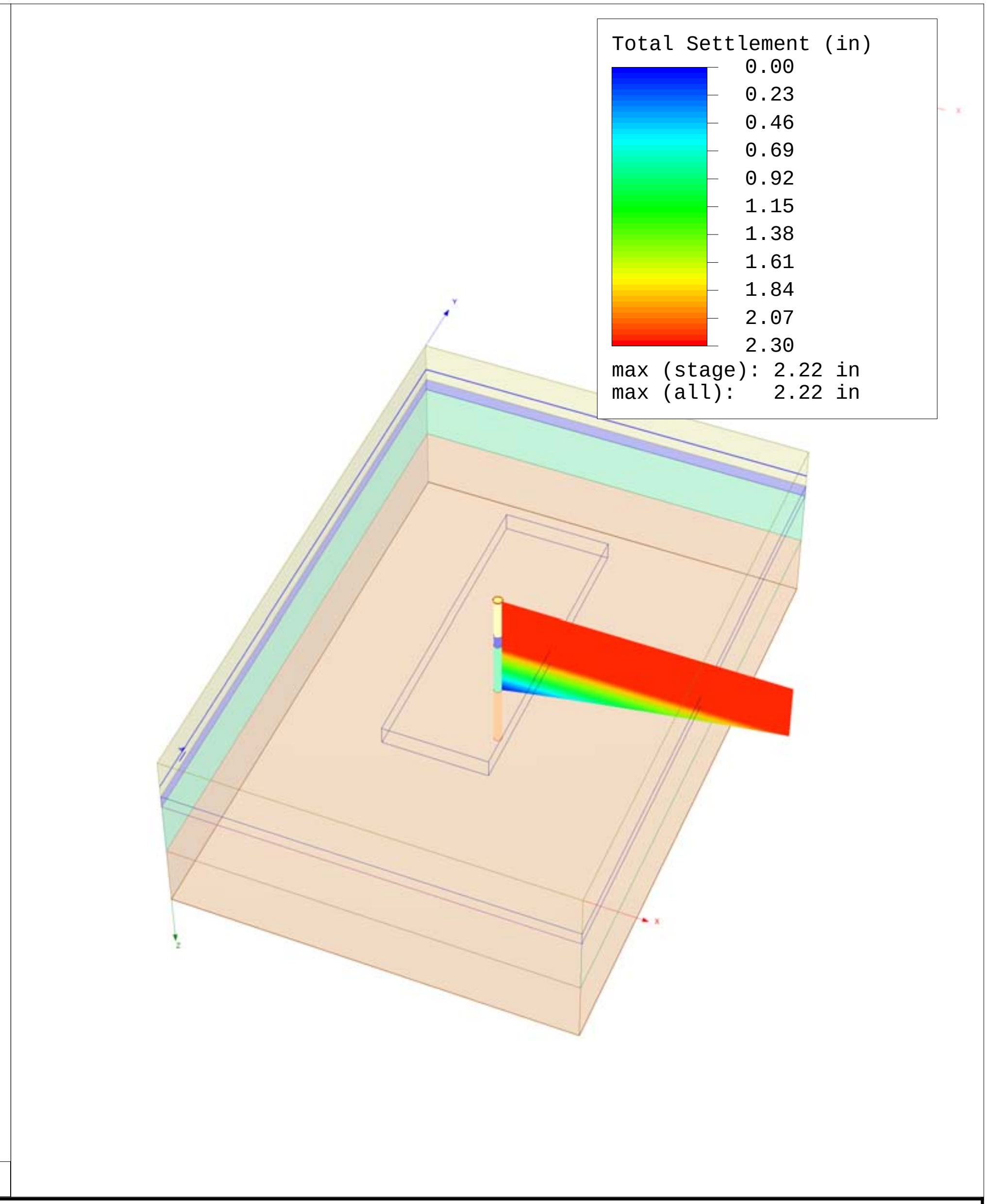
Grid Coordinates

X [ft]	Y [ft]
52.434	82.691
52.434	-53.309
-39.566	-53.309
-39.566	82.691



Reference Stage: None





rocscience	*Project*	Gorham Higgins Bridge	
	Analysis Description	Immediate, 1-year consolidation and secondary compression	
	Drawn By Van Buskirk/Krusinski	*Company* MaineDOT	
SETTLE3D 3.020	*Date* 10/19/2018, 1:21:00 PM	*File Name* 23899 q=500; upper limit.s3z	

Settle3D Analysis Information
Gorham Higgins Bridge

Project Settings

Document Name	23899 q=1500; with Immediate.s3z
Project Title	Gorham Higgins Bridge
Analysis	Immediate, 1-year consolidation and secondary compression
Author	Van Buskirk/Krusinski
Company	MaineDOT
Date Created	10/19/2018, 1:21:00 PM
Comments	
22' span - L=68 ft, B=24 ft	
Net q = 500 psf	
3 Clayey Silt Subunits	
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	years
Permeability Units	feet/year
Use average properties to calculate layered stresses	

Stage Settings

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

Results

Time taken to compute: 0.0404705 seconds

Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.92815
Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	2.92815
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	3.03226
Total Stress [ksf]	0	5.8768
Total Strain	0	0.0257434
Pore Water Pressure [ksf]	0	2.84454
Excess Pore Water Pressure [ksf]	0	1.5
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.00875	3.67304
Over-consolidation Ratio	1	2.5
Void Ratio	0	1.1
Permeability [ft/y]	0	0.15645
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.653173

Stage: Consolidation = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.99915
Consolidation Settlement [in]	-0.177417	0.0751094
Immediate Settlement [in]	0	2.92815
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	2.33713
Total Stress [ksf]	0	5.8768
Total Strain	-0.0224787	0.0266181
Pore Water Pressure [ksf]	0	3.53967
Excess Pore Water Pressure [ksf]	0	1.38063
Degree of Consolidation [%]	0	6.92991
Pre-consolidation Stress [ksf]	0.00875	3.67304
Over-consolidation Ratio	1	13.9662
Void Ratio	0	1.10813
Permeability [ft/y]	0	0.15645
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	21.3046
Undrained Shear Strength	0	0.641088

Stage: Long-term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	4.19088
Consolidation Settlement [in]	0	1.0704
Immediate Settlement [in]	0	2.92815
Secondary Settlement [in]	0	0.192329
Loading Stress [ksf]	0	1.5
Effective Stress [ksf]	0	3.70944
Total Stress [ksf]	0	5.8768
Total Strain	-0.00464626	0.0399425
Pore Water Pressure [ksf]	0	2.16737
Excess Pore Water Pressure [ksf]	0	0.00832683
Degree of Consolidation [%]	0	99.4973
Pre-consolidation Stress [ksf]	0.00875	3.70882
Over-consolidation Ratio	1	2.51005
Void Ratio	0	1.10003
Permeability [ft/y]	0	0.15645
Coefficient of Consolidation [ft^2/y]	0	73
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	99.5358
Undrained Shear Strength	0	0.716047

Loads

1. Rectangular Load

Length	24 ft
Width	68 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1632 ft^2
Load	1.5 ksf
Depth	14 ft
Installation Stage	Immediate = 0 y

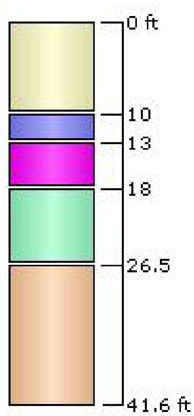
Coordinates

X [ft]	Y [ft]
-5.566	-19.309
18.434	-19.309
18.434	48.691
-5.566	48.691

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Fill	10	0	No
2	Stiff Clayey Silt	3	10	No
3	Upper Soft Clayey Silt	5	13	No
4	Soft Clayey Silt	8.5	18	No
5	Glacial Outwash	15.1	26.5	No



Soil Properties

Property	Fill	Glacial Outwash	Stiff Clayey Silt	Upper Soft Clayey Silt	Soft Clayey Silt
Color					
Unit Weight [kips/ft^3]	0.125	0.115	0.108	0.083	0.083
Saturated Unit Weight [kips/ft^3]	0.115	0.128	0.135	0.121	0.121
Immediate Settlement	Enabled	Enabled	Enabled	Enabled	Enabled
Es [ksf]	450	8000	146.198	146.198	57.435
Esur [ksf]	1800	32000	480	208.9	208.9
Primary Consolidation	Disabled	Disabled	Enabled	Enabled	Enabled
Material Type			Non-Linear	Non-Linear	Non-Linear
Cc			0.164	0.4	0.4
Cr			0.0164	0.04	0.04
e0			1.1	0.9	0.9
OCR	1	1	2.5	2	1.8
Cv [ft^2/y]			73	36.5	36.5
B-bar			1	1	1
Secondary Consolidation	Disabled	Disabled	Standard	Standard	Standard
Cae/Ca			0.01	0.01	0.011
Car/Care			0.01	0.01	0.01
Undrained Su A [kips/ft2]	0	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1	1

Groundwater

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

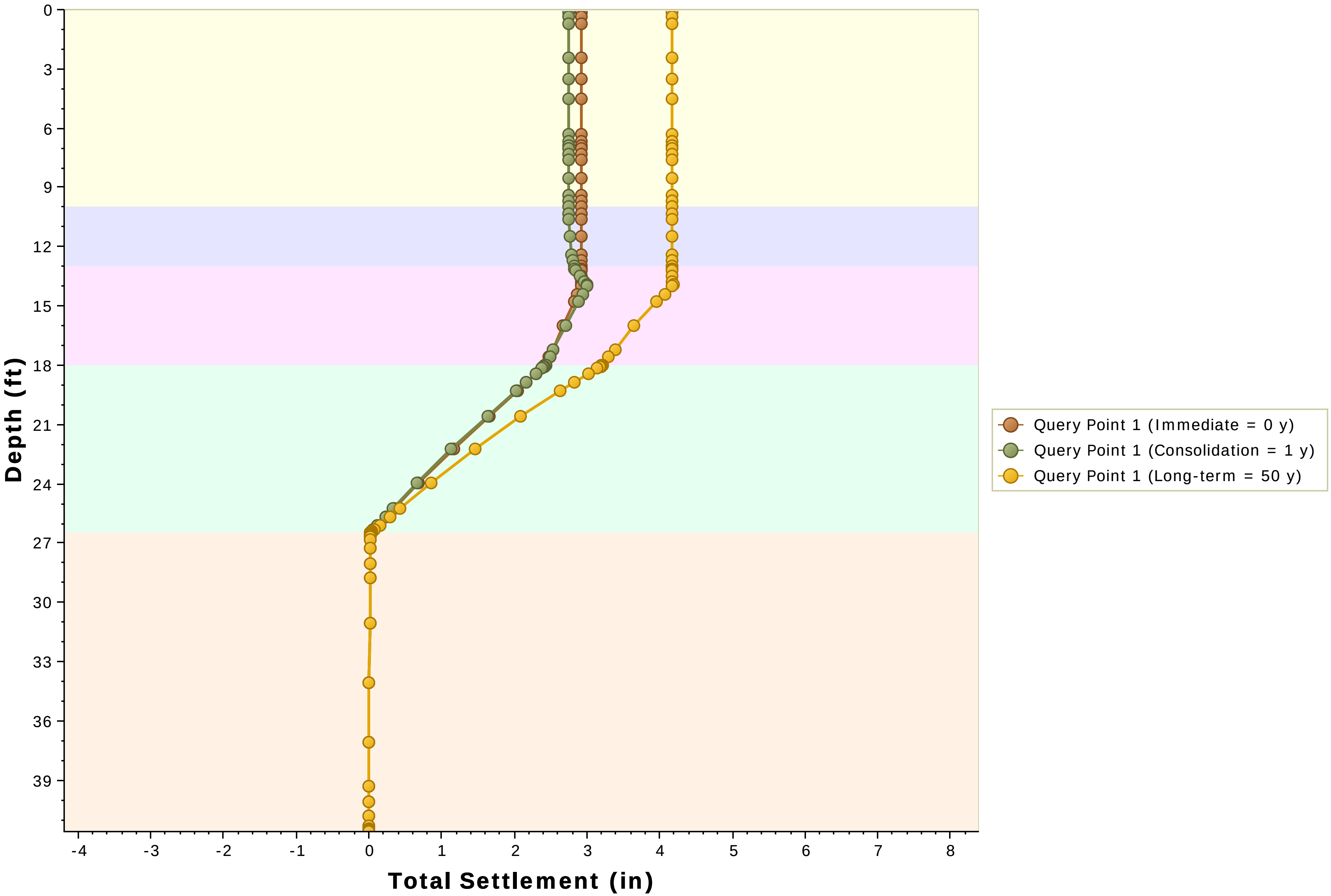
Piezometric Line Entities

ID	Depth (ft)
1	7 ft

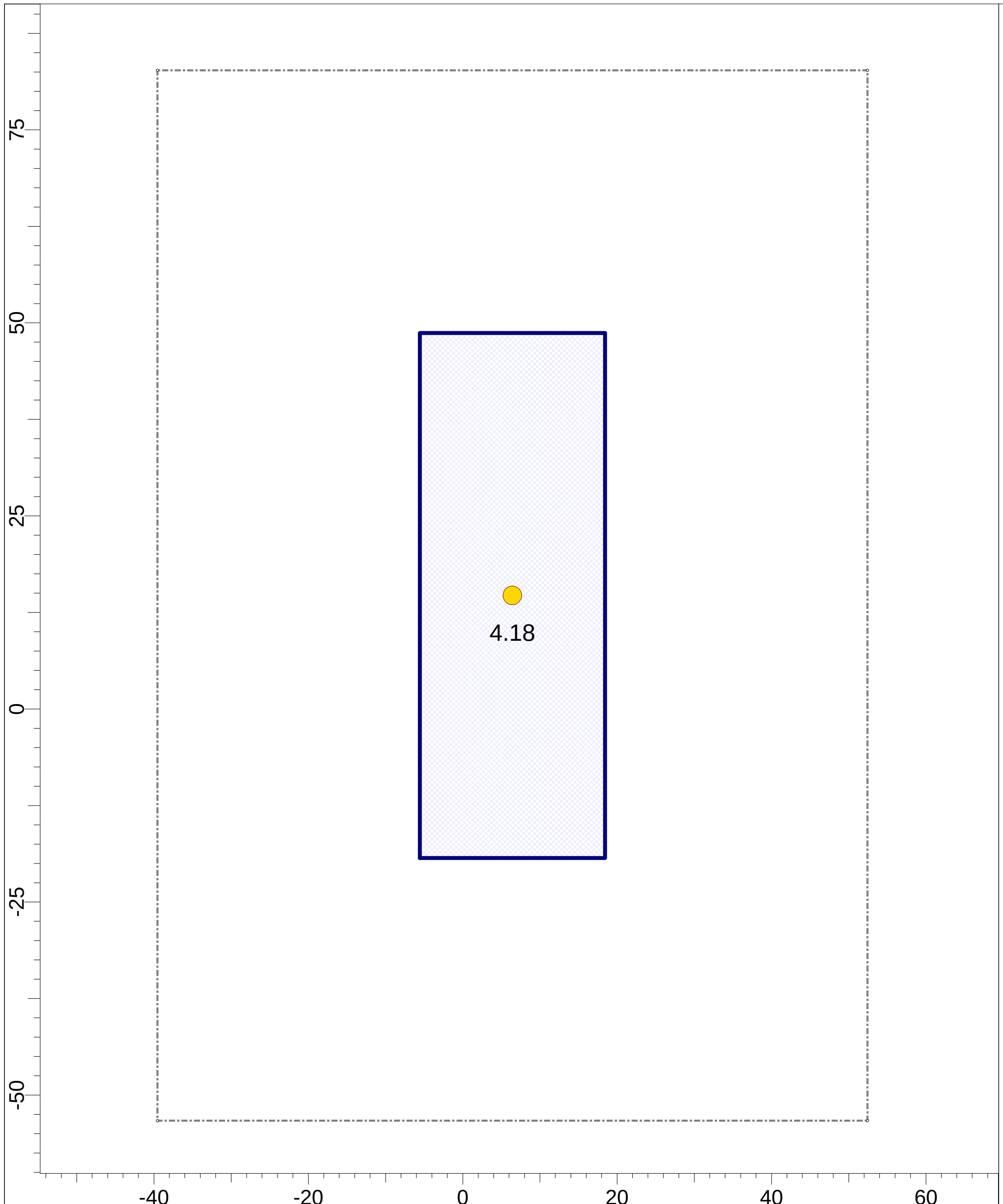
Query Points

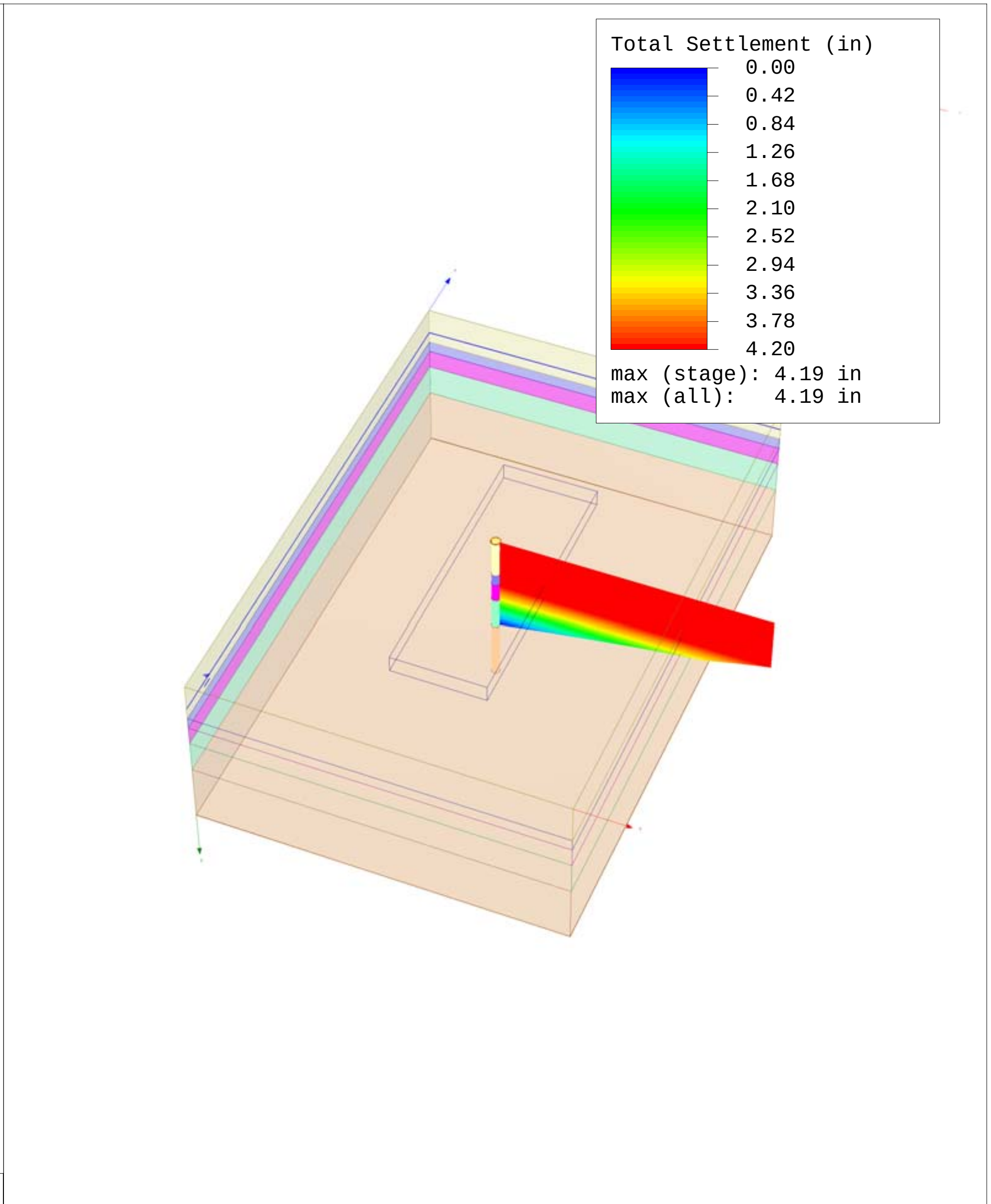
Point #	(X,Y) Location	Number of Divisions
1	6.434, 14.691	Auto: 71

Total Settlement vs. Depth



Reference Stage: None





rocscience	*Project* Gorham Higgins Bridge		
	Analysis Description Immediate, 1-year consolidation and secondary compression		
	Drawn By Van Buskirk/Krusinski	*Company* MaineDOT	
SETTLE3D 3.015	*Date* 10/19/2018, 1:21:00 PM	*File Name* 23899 q=1500; with Immediate.s3z	

Frost

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

From Design Freezing Index Map: Gorham, **Maine**
Case 1 - fine grained granular fill soils W=25%

DFI = 1300

Depth of Frost Penetration

$d_1 := 46.6\text{in}$ for $w = 20\%$

$d_2 := 42.2\text{in}$ for $w = 30\%$

Calculate average to find frost depth for $w = 25\%$

$$\frac{d_1 + d_2}{2} = 3.7\,\text{ft}$$

Method 2 - ModBerg Software

Examine foundations placed on fine grained fill soils

Brunswick lies near the 1300 F-days isoline

```
            --- ModBerg Results ---

  Project Location: Brunswick, Maine

  Air Design Freezing Index       = 1276 F-days
  N-Factor                        = 0.70
  Surface Design Freezing Index   =  893 F-days
  Mean Annual Temperature         = 45.9 deg F
  Design Length of Freezing Season = 118 days
  ----------------------------------------------------------
  Layer
  #:Type       t   w%   d    Cf  Cu   Kf  Ku    L
  ----------------------------------------------------------
  1-Fine      38.7 25.0 108.0  32  45   1.5  1.1  3,888
  ----------------------------------------------------------

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

  ***********************************************************
   Total Depth of Frost Penetration = 3.22 ft = 38.7 in.
  ***********************************************************
```

Recommendation: 3.2 feet for design of foundations constructed on fine grained soils

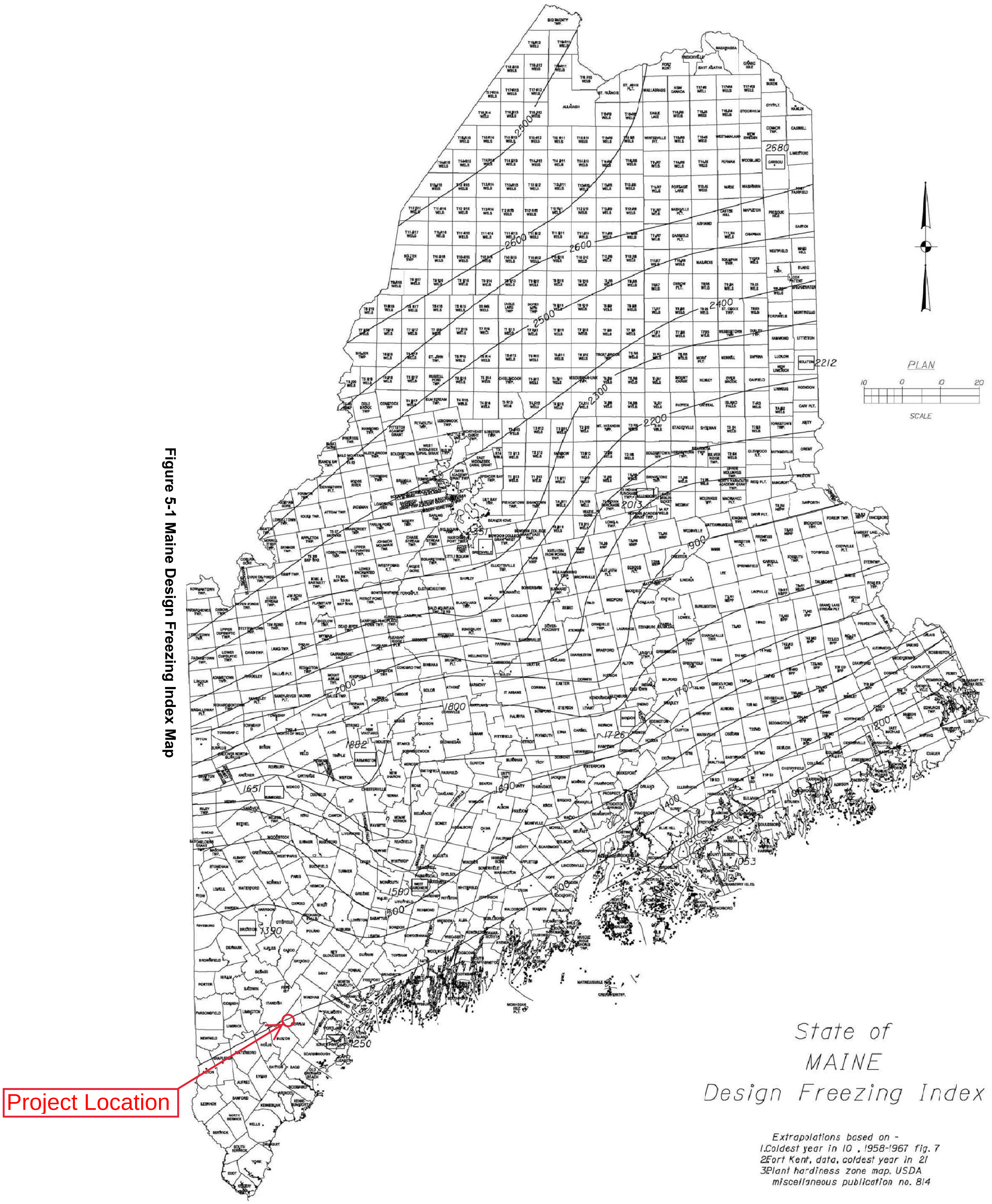


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

W = 25%
Frost Penetration = 44.4 in

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