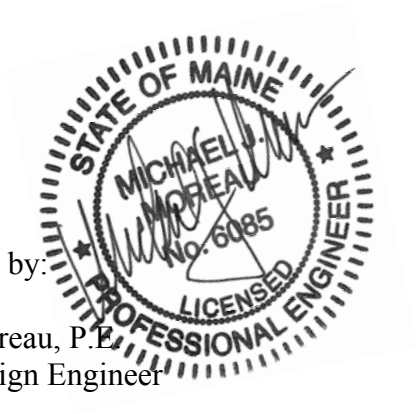
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

PRELIMINARY GEOTECHNICAL DESIGN REPORT

For the Replacement of:

UNION STREET BRIDGE
OVER INTERSTATE 95
BANGOR, MAINE



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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for replacement of the Union Street Bridge over Interstate 95 in Bangor, Maine. The replacement structure will be a two-span semi-integral bridge with cantilever-type abutments on spread footings cast on compacted fill or native glacial till. The abutments will include U-shape return wingwalls and cast-in-place protected slopes in front of the breastwalls. The replacement bridge design must conform to the requirements of the Bridge Design Guide (BDG) and the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010, (herein referred to as LRFD). The design and construction recommendations below are discussed in greater detail in Section 7.0 Evaluation and Recommendations.

Cantilever Abutments and Wingwalls – The abutments and wingwalls will be designed to resist all lateral earth loads, vehicular loads, superstructure loads, collision forces and any loads transferred through the superstructure. Abutments and wingwalls will be designed for all relevant strength, service and extreme limit states in accordance with LRFD.

The design of project abutments founded on spread footings at the strength and extreme limit states shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on soil. A maximum frictional coefficient of 0.55 at the soil-concrete interface should be assumed. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-quarter (1/4) of the footing dimensions, in either direction.

Service limit load conditions may control foundation design of the Union Street Bridge. A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and overall stability. The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. The foundations shall be constructed a minimum of 6.5 feet below exterior finished grade.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31 calculated using Rankine Theory for cantilever wingwalls. The designer may assume Soil Type 4 [Bridge Design Guide (BDG) Section 3.6.1] for backfill soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Additional lateral earth pressure due to construction or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of surcharge loads is permitted.

All abutment and wingwall designs shall include a drainage system behind them to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4, Drainage, of the BDG. The footing subgrade excavation should be protected from disturbance by construction traffic or water-softening. We recommend that the contractor place and compact a 12-inch thick layer of granular borrow over the excavated subgrade to

prepare the site for foundation construction and protect the native subgrade soil from disturbance.

The abutment and wingwall footing elevations must be determined in the final design phase. Although preliminary foundation elevations are presented in the Union Street Bridge Preliminary Design Report (PDR), site conditions, construction staging, utilities or other final design considerations may require establishment of footing elevations different than proposed in the preliminary design concept.

If wingwalls are designed with step-up foundations constructed over pre-existing fill soil, the fill soil shall be sampled and subjected to Proctor tests. The wingwall footing subgrade soil shall then be compacted to 95% of the maximum dry density in accordance with AASHTO T-180.

Pier Foundation – Strength and extreme limit state design of the pier foundation shall consider bearing resistance, eccentricity (overturning), failure by sliding and structural failure. Extreme event load combinations are those relating to vehicle collision.

Service limit load conditions may control foundation design of the Union Street Bridge. Service limit state design checks shall be used to assess pier footing settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. The foundation shall be constructed a minimum of 6.5 feet below exterior finished grade.

For sliding analyses at the strength limit state, a sliding resistance factor of φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on soil. A maximum frictional coefficient of 0.55 at the soil-concrete interface should be assumed. For pier footings on soil, the eccentricity of loading at the strength limit state based on factored loads shall not exceed one-quarter (1/4) of the footing dimensions, in either direction.

The pier footing elevation must be determined in the final design phase. Although a preliminary foundation elevation is presented in the Union Street Bridge PDR, site conditions, construction staging, utilities or other final design considerations may require establishment of a footing elevation different than proposed in the preliminary design concept.

Vehicle Collision Extreme Event Design - Any abutment, pier or wingwall constructed within 30 feet of the edge of the roadway must be protected from vehicular collision with barriers as specified in LRFD Article 3.6.5.1. If barriers are not used, the abutment, pier or wingwall shall be designed for an equivalent static force of 400 kips assumed to act in any direction in a horizontal plane at a distance of 4.0 feet above the ground surface. The Extreme Event II limit state design check related to collision by vehicles includes bearing resistance, eccentricity, sliding and structural failure. A resistance factor $\varphi = 1.0$ is used for the Extreme Event II limit state. Current design concepts include a concrete revetment in

front of the abutments but the current pier design concept must consider extreme event conditions.

Factored Bearing Resistance – Based on presumptive bearing resistance values and local experience, a factored bearing resistance of 4 ksf should be used when analyzing the service limit state and for footing sizing to control settlement, as allowed in LRFD C10.6.2.6.1. The strength and extreme limit state factored bearing resistances for spread footings on compacted granular fill or native glacial till shall depend on the footing size as presented in the graph in Section 7.4 of this report. In no instance shall the service limit state bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3f'c$. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

Settlement – We estimate that settlement as a result of fill replacement and minor embankment fill extensions over natural soils will be negligible. We have estimated that the total settlement of a prepared subgrade consisting of compacted fill or native glacial till will be on the order of 1 to 2 inches for conventional spreads footings. We estimate that differential settlement will be on the order of 1 inch or less. In all cases above, this settlement is acceptable and will occur during construction. We anticipate that post-construction settlement will be negligible.

Frost Protection – Foundations placed on granular soils shall be founded a minimum of 6.5 feet below finish exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on soil and not those founded on bedrock.

Seismic Design Considerations – Seismic analysis is not required for multiple-span bridges in Seismic Zone 1. Nevertheless, superstructure connections and bridge seat dimensions shall be designed in accordance with LRFD requirements. The following are the Union Street Bridge seismic design parameters:

- Peak Ground Acceleration coefficient (PGA) = 0.067g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.233g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.105g
- Site Class D (stiff soil with $15 < N_{avg} < 50$ blows per foot)
- Seismic Zone 1, based on an $S_{D1} < 0.15$g

Construction Considerations –

<u>Excavation</u>

- Remove the old abutments and pier in their entirety. This will require staged construction methods and may require earth support systems.
- Construction of new abutment, pier and retaining wall structures will require soil excavation. Earth support systems may be required.

<u>Subgrade Preparation</u>

- The high fines and water contents of the native glacial till make this soil susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and/or rutting occur, the contractor should remove and replace the disturbed soil materials and replace it with compacted granular borrow.

- If any of the abutment or wing wall footings will be designed to bear on existing fill soils, over-excavate the footing location 2 feet and replace the excavated soil with granular borrow. Prior to placing the granular borrow, subject the fill subgrade soil to Proctor testing and then compact the fill subgrade to 95% of the maximum dry density as determined by AASHTO T-180. The contractor should subsequently place granular borrow up to the proposed new subgrade level and compact it to 95% of the AASHTO T-180 maximum dry density prior to constructing the footings.

Dewatering

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Cofferdams, temporary ditches, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert surface water or groundwater if significant seepage is encountered during excavation.

Reuse of Excavated Soil and Bedrock

- Do not use excavated existing subbase aggregate for pavement structure construction or to re-base shoulders or for abutment and wall backfill soil. Excavated subbase sand and gravel may be used as fill below subgrade elevation in fill embankment areas.
- Do not use excavated existing fill or native soils for fill anywhere beneath the pavement structure, dressing slopes, abutments or walls. Use these soils to dress slopes only below the bottom elevation of the shoulder subbase gravel.
- Silty native soils or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. It may be necessary to spread out and dry portions of these soils that are excessively moist.

Embankment Fill Areas

- Bench existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes.

Erosion Control

- Use MaineDOT Best Management Practices February 2008 to minimize erosion of fine-grained soils found on the project site.

1.0 INTRODUCTION

The Maine Department of Transportation (MaineDOT) plans to replace the Union Street Bridge which carries Route 222 over Interstate 95 in the Town of Bangor, Penobscot County, Maine. We show the project location on Sheet 1, Site Location Map, appended to this report. We conducted subsurface investigations at the bridge site to develop geotechnical recommendations for the structure replacement. This report summarizes our findings, discusses our evaluation of the subsurface conditions and presents our geotechnical recommendations for design and construction of the bridge foundations.

The existing simply supported two-span steel I-beam bridge was originally built in 1960. The abutments were founded on spread footings formed and cast over native glacial till soil. The bridge was rehabilitated with a new concrete wearing surface in 1980 and the steel girders are currently in good condition. MaineDOT is considering bridge replacement due to deck cracking, under-deck concrete spalling, pier and abutment cracks with some delamination and efflorescence and corrosion of exposed rebar. Many of the bridges along the I-95 corridor in Bangor were constructed around the same time and a number of these bridges have shown signs of Alkali-Silica-Reactivity (ASR). The bridge had a sufficiency rating of 61.9 in 2009.

Preliminary design studies by Becker Structural Engineers, Inc. of Portland, Maine, have identified cantilever-type abutments on spread footings to be the most practicable foundation type for this site. The spread footings will be founded directly on compacted fill or native glacial till soil. The proposed bridge will consist of a 184-foot, two-span steel girder superstructure with a total width of 74 feet. The bridge will have a curb-to-curb width of 59 feet which will accommodate four 11-foot travel lanes, one 11-foot turning lane and two 2-foot wide shoulders. The proposed bridge also includes two 6-foot wide sidewalks. The current bridge replacement plans include profile changes of approximately 2 feet higher than original grades at the center of the bridge and grading back down to original grade roughly 200 feet east and west of the bridge.

2.0 GEOLOGIC SETTING

The Union Street Bridge foundations and approaches are constructed onto the backslopes and base of a large open cut in the native glaciated sediments. The highway cut was excavated for the construction of twin northbound and southbound lanes for Interstate 95.

The Maine Geologic Survey (MGS) "Surficial Geology of Bangor Quadrangle, Maine, Open-file No. 77-24" (1977) indicates that surficial soils in the vicinity of the Union Street Bridge are predominantly glacial till deposits with nearby glacial-marine soil unit and bedrock outcrop unit contacts. The glacial till is typically a heterogeneous mixture of sand, silt, clay, and stones. Glacial-marine deposits generally consist of silt, clay, and sand, commonly a clayey silt, but sand is abundant at the surface in some places.

According to the Maine Geologic Survey "Bedrock Geologic Map of Maine" (1985), the

bedrock at the Union Street Bridge site consists of Silurian-Ordovician, calcareous sandstone, interbedded sandstone and impure limestone. Locally the bedrock has been identified as phyllite and metasandstone and is part of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions in the vicinity of the existing bridge by drilling eight test borings, BB-BUS-101, BB-BUS-101A, BB-BUS-101B, and BB-BUS-102 through BB-BUS-106. The approximate boring locations are shown on Sheet 2 Boring Location Plan, found at the end of this report. We terminated all of the borings with bedrock cores except BB-BUS 101 and BB-BUS-101A where drilling difficulties caused early termination of the borings. MaineDOT drill rig and crew and the contract drill crew, Northern Test Borings of Gorham, Maine, conducted the borings on May 18 through May 20, 2009. We present the details and sampling methods used, field data obtained, and soil and groundwater conditions encountered in the boring logs in Appendix A provided at the end of this report.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. A geotechnical engineer or a MaineDOT New England Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions in the borings. The MaineDOT survey crew determined the boring location coordinates in the field after the borings were completed. The survey coordinates are based on the NAVD 88 datum.

The borings were drilled using solid stem auger and cased wash boring techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the SPT N_{60}-values by applying an average hammer energy transfer factor of 0.84 and 0.678 to the raw field N-values obtained with the MaineDOT and Northern Test Boring automatic hammers, respectively. Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core.

4.0 LABORATORY TESTING PROGRAM

We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification and soil properties. We performed the soil laboratory testing at the AASHTO accredited MaineDOT Soils Laboratory in Bangor, Maine. Laboratory testing consisted of 26 standard grain size analyses with natural water content, one with hydrometer analysis, and one Atterberg Limits test. We present the results of the laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classification and water content data are also presented on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Regional surficial geology maps show that the bridge site is situated in an area where glacial till deposits predominate with nearby glacial-marine soil unit and bedrock outcrop unit contacts. The bridge itself is situated at the end of short fill extensions built into the I-95 highway cut. The approach embankment soil behind the existing bridge abutments is predominantly granular fill overlying approximately 4.5 to 6.0 feet of glaciomarine clay-silt which is underlain by approximately 24.5 to 49.4 feet of glacial till. The glaciomarine clay-silt unit was absent in BB-BUS-101, BB-BUS-101A BB-BUS-104. Each of the abutment borings also encountered and penetrated concrete approach slabs.

At the pier locations, we observed granular fill over approximately 17.3 to 34.6 feet of glacial till. The glacial till overlies bedrock at all the abutment and pier boring locations. A summary description of the subsurface conditions follows.

5.1 Granular Fill

We encountered granular fill to a depth ranging between approximately 4.5 and 13.3 feet below ground surface (bgs) at the abutment locations. At the pier locations we observed approximately 4.5 to 5.9 feet of granular fill. The granular fill consists of fine to coarse sand, with some gravel to gravelly and trace to little silt, or fine to coarse sandy gravel with trace silt, or fine to coarse sand and gravel with little silt. The SPT N_{60}-values in the granular fill ranged from 6 to 43 blows per foot (bpf) indicating that the unit is loose to dense in consistency.

The granular fill samples selected for laboratory testing had water contents ranging between approximately 2 and 5 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-a and A-1-b by the AASHTO Classification System and GP-GM and SM under the Unified Soil Classification System.

5.2 Glaciomarine Silt and Clay

We encountered glaciomarine clay-silt beneath the approach fills in most of the abutment borings, but we did not observe the clay-silt layer in BB-BUS-101, BB-BUS-101A and BB-BUS-104. This soil unit consists of clay-silt with trace fine sand and gravel. The fine grained soil unit is over-consolidated and slightly plastic. The thickness of the glaciomarine sediments encountered ranged between approximately 4.5 and 6.0 feet. SPT N_{60}-values ranged from 11 to 22 bpf, indicating those deposits are stiff to very stiff in consistency. The clay-silt sample subjected to Atterberg Limits testing had a liquid limit of 31, a plasticity index of 9, and a natural water content of 21 percent. The second clay-silt sample tested had a natural water content of 18 percent. Grain size analyses indicate that the soils are classified as A-4 and CL or ML by the AASHTO and Unified Soil Classification Systems, respectively. Below the glaciomarine clay-silt we encountered the glacial till soil unit.

5.3 Glacial Till

The glacial till found in the borings generally comprised of silt with trace to some fine to coarse sand and trace to some gravel with occasional to numerous cobbles. The glacial till unit was often well bonded or cemented also. The thickness of this soil unit ranged between approximately 24.5 to 49.4 feet at the abutment locations and approximately 17.3 to 34.6 feet at the pier locations. SPT N_{60}-values ranged from 7 to 101 bpf, indicating the till deposit is loose to very dense in consistency. We observed the glacial till unit over bedrock in each of the borings.

The glacial till samples had water contents ranging between approximately 9 and 16 percent. Grain size analyses conducted on selected samples of the till soils indicate that the soils are classified as A-4 by the AASHTO Classification System and ML or SM under the Unified Soil Classification System.

5.4 Bedrock

We encountered bedrock at approximate depths ranging from 29.6 to 58.4 feet bgs at the abutment locations and 21.8 to 40.5 feet bgs at the pier location. Regionally, the bedrock is mapped by MGS as calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation. We visually identified the local bedrock cores at all the boring locations as a grey, fine-grained, metasedimentary phyllite that is soft to moderately hard, fresh to slightly weathered with close open joints. The bedrock contains fractures that are oriented from horizontal to vertical and have minor silt in-filling and iron-staining. The bedrock also contains numerous quartz and calcite seams 2 to 4 mm thick that often follow foliation. We determined that the rock quality designation (RQD) of the bedrock ranged from 7 to 100 percent which correlates to a very poor to excellent rock mass quality.

We also observed one metasandstone interbed in boring BB-BUS-103. The metasandstone layer consisted of brown, fine-grained, highly weathered and soft rock, with very close and open fractures from horizontal to vertical with significant silt in-filling and iron and manganese staining. We observed a friable layer within the metasandstone interbed at 61.6 feet to 63.0 feet bgs. The RQD of the bedrock ranged from 7 to 15 percent indicating a very poor rock mass quality. The table below summarizes the top of bedrock elevations at the boring locations:

Substructure	Boring	Station	Depth to Bedrock (feet bgs)	Elev. of Apparent Bedrock Surface (feet)
Abutment No. 1	BB-BUS-103	4+13.1, 23.7 LT	58.4	95.7
Abutment No. 1	BB-BUS-104	4+11.6, 16.1 RT	42.6	111.6
Abutment No. 2	BB-BUS-101B	6+36.8, 22.7 LT	37.5	117.4
Abutment No. 2	BB-BUS-106	6+26.7, 16.8 RT	29.6	125.2
Pier	BB-BUS-102	5+11.2, 38.0 LT	40.5	94.3
Pier	BB-BUS-105	5+11.4, 36.2 RT	21.8	113.7

Bedrock Depth and Elevation at the Boring Locations

5.5 Groundwater

We observed the groundwater level at approximate depths ranging between 6.0 and 13.0 feet bgs in borings BB-BUS-101, BB-BUS-101B, BB-BUS-102, BB-BUS-105 and BB-BUS-106. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

For a more detailed description of the subsurface conditions, please refer to Appendix A, Boring Logs attached to this report.

6.0 FOUNDATION ALTERNATIVES

Both rehabilitation and replacement options were initially considered. Rehabilitation options were dismissed because rehabilitation would not solve the need for a fifth lane on the bridge, nor would it correct the inadequate bridge under-clearance condition. Replacement options included mechanically stabilized earth (MSE) supported semi-integral stub abutments, integral abutments, and conventional spread footing substructures, and butted box beam or welded steel girder superstructures.

MSE supported abutments were eliminated because of the surface area required to accommodate the minimum 22-foot long reinforcing strips. Integral abutments were dismissed because this site requires long U-shaped wingwalls contrary to the 10-foot maximum length straight extension wingwalls allowed for integral abutments per BDG Section 5.4.2.9. The butted box beams were eliminated because of difficulties accommodating existing utility ducts, inability to follow the planned bridge vertical curve, and expense. Consequently, the most practical and durable alternative is a welded steel girder superstructure supported on full height cantilever abutments on conventional spread footings. Section 7.0, Evaluation and Recommendations, of this report provides geotechnical design recommendations for the full height cantilever abutments, the pier and retaining walls on spread footings founded on compacted fill or native glacial till.

7.0 EVALUATION AND RECOMMENDATIONS

The preliminary bridge alternate selected for the Union Street Bridge replacement is a two-span welded steel girder superstructure supported by full height cast-in-place cantilever abutments and wingwalls on spread footings cast directly on compacted fill or native glacial till. The new pier will also be founded on a spread footing cast directly on compacted fill or native glacial till. The new bridge has a proposed total width of 74 feet and length of 184 feet. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010.

7.1 Abutment and Wingwall Design

Abutments and wingwalls shall be proportioned for all applicable load combinations in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, service and extreme limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure.

A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place abutments and wingwalls founded on spread footings on compacted fill or glacial till. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.55 at the concrete footing to soil interface. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-quarter (1/4) of the footing dimensions, in either direction.

Service limit load conditions may control foundation design of the Union Street Bridge. A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and overall stability. The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. The foundations shall be constructed a minimum of 6.5 feet below exterior finished grade.

Cantilever-type abutments and wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, $K_a = 0.31$, calculated using Rankine Theory for cantilever-type abutments and wingwalls. See Appendix C – Calculations, for supporting documentation. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and wingwalls if an approach slab is not specified. In the case where a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of no less than 2.0 feet, per LRFD Table 3.11.6.4-1. The live load surcharge on

abutments may be estimated as a uniform earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Abutment Height (feet)	h_{eq} (feet)
5.0	4.0
10.0	3.0
$\geq$ 20.0	2.0

All abutment and wingwall designs shall include a drainage system behind them to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4, Drainage, of the BDG. The footing subgrade excavation should be protected from disturbance by construction traffic or water-softening. We recommend that the contractor place and compact a 12-inch thick layer of granular borrow over the excavated subgrade to prepare the site for foundation construction and protect the native subgrade soil from disturbance.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to MaineDOT Specification 709.19, Granular Borrow for Underwater Backfill. This gradation specifies 10 percent or less of material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure and below the approach slab.

If wingwalls are designed with step-up foundations constructed over pre-existing fill soil, the fill soil shall be sampled and subjected to Proctor tests. The wingwall footing subgrade soil shall then be compacted to 95% of the maximum dry density in accordance with AASHTO T-180.

Abutment and wingwall design shall also consider extreme event load combinations relating to vehicle collision, including bearing resistance, eccentricity (overturning), failure by sliding and structural failure as discussed in Section 7.3, Vehicle Collision Extreme Event Design.

The abutment and wingwall footing elevations must be determined in the final design phase. Although preliminary foundation elevations are presented in the Union Street Bridge PDR, site conditions, construction staging, utilities or other final design considerations may require establishment of footing elevations different than proposed in the preliminary design concept.

7.2 Pier Foundation Design

The pier foundation shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, extreme and service limit states. The pier foundation strength limit state design shall consider bearing resistance, eccentricity (overturning), failure by sliding and structural failure. For sliding analyses at the strength limit state, a sliding resistance factor of φ_τ, of 0.80 shall be applied to the nominal sliding resistance of a pier founded on a spread footing on soil. A maximum frictional coefficient of 0.55 at the soil-concrete interface should be assumed. For pier

footings on soil, the eccentricity of loading at the strength limit state based on factored loads shall not exceed one-quarter (1/4) of the footing dimensions, in either direction.

Service limit state load conditions may control foundation design at the Bangor Union Street site. A resistance factor, φ, of 1.0 shall be used to assess pier footing settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. The foundations shall be constructed a minimum of 6.5 feet below exterior finished grade.

The pier design shall also consider the extreme event load combination relating to vehicle collision, including bearing resistance, eccentricity (overturning), failure by sliding and structural failure as discussed in Section 7.3, Vehicle Collision Extreme Event Design.

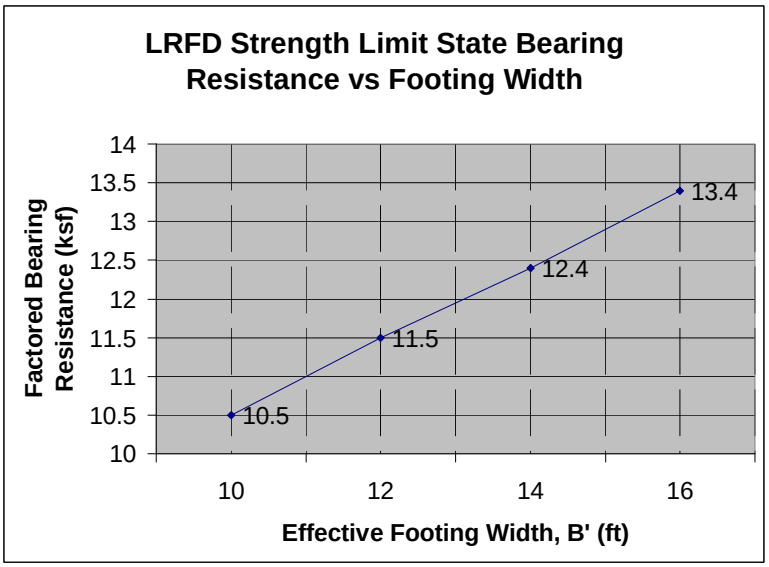
The pier footing elevation must be determined in the final design phase. Although a preliminary foundation elevation is presented in the Union Street Bridge PDR, site conditions, construction staging, utilities or other final design considerations may require establishment of a footing elevation different than proposed in the preliminary design concept.

7.3 Vehicle Collision Extreme Event Design

Any abutment, pier or wingwall constructed within 30 feet of the edge of the roadway must be protected from vehicular collision with barriers as specified in LRFD Article 3.6.5.1. If barriers are not used, the abutment, pier or wingwall shall be designed for an equivalent static force of 400 kips assumed to act in any direction in a horizontal plane at a distance of 4.0 feet above the ground surface. The Extreme Event II limit state design check related to collision by vehicles includes bearing resistance, eccentricity, sliding and structural failure. A resistance factor $\varphi = 1.0$ is used for the Extreme Event II limit state as specified in LRFD Article 11.5.7. The extreme event factored bearing resistances are presented in Section 7.4, Factored Soil Bearing Resistances.

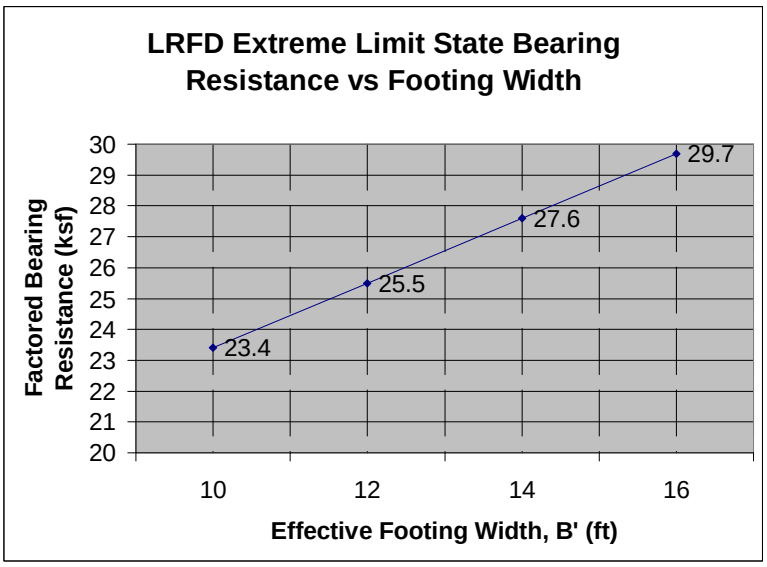
7.4 Factored Soil Bearing Resistance

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The factored bearing resistance for any structure founded on compacted fill or native glacial till shall be investigated at the strength limit state using factored loads and a factored bearing resistance dependent on the proposed footing width. The graph below shows the appropriate bearing resistance for comparable effective footing widths, B'. These bearing resistances assume a bearing resistance factor, φ_b, for spread footings on soil of 0.45, based on bearing resistance evaluation using semi-empirical methods.



A service limit factored bearing resistance of 4 ksf may be used for preliminary footing sizing and to control settlements when analyzing the service limit state load combination. The service limit state loading conditions may control foundation design for the Bangor Union Street Bridge project. See Appendix C, Calculations, for supporting documentation.

The extreme limit state factored bearing resistances are dependent on footing size. The graph below shows the appropriate bearing resistance for comparable effective footing widths, B'. These extreme event bearing resistances assume a factor, φ, of 1.0 for spread footings on soil as required by LRFD 11.5.7.



In no instance shall the factored strength, service or extreme limit state bearing stress exceed the nominal compressive resistance of the footing concrete, which may be taken as $0.3f'_c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.5 Settlement

We estimate that settlement as a result of fill replacement and minor embankment fill extensions over natural soils will be negligible. We have estimated that the total settlement of a prepared subgrade consisting of compacted fill or native glacial till will be on the order of 1 to 2 inches for conventional spreads footings. We estimate that differential settlement will be on the order of 1 inch or less. In all cases above, this settlement is acceptable and will occur during construction. We anticipate that post-construction settlement will be negligible.

7.6 Frost Protection

We have evaluated the potential frost depth at the site for footings placed on soil. Based on State of Maine frost depth maps, BDG Figure 5-1, the site has a design-freezing index of approximately 1730 F-degree days. Considering an assumed water content of 10 percent, this correlates to a frost depth of 7.4 feet at this site. We also considered frost depth projections computed by Modberg software developed by the US Army Cold Regions Research and Engineering Laboratory. The results of the Modberg frost depth model indicate a potential frost depth of 6.5 feet. Consequently, we recommend that any foundations or leveling pads constructed on soil at this site be founded a minimum of 6.5 feet below finished exterior grade. This minimum embedment applies only to foundations constructed on soil and not those founded on bedrock.

7.7 Seismic Design Considerations

The replacement Union Street Bridge in Bangor will be a multiple span bridge over Interstate 95 which is on the National Highway System. The bridge may be considered “essential” or “critical” according to the operational classifications of LRFD Article 3.10.5. LRFD Article 4.7.3.1 specifies that seismic analysis is not required for essential or critical multi-span bridges in Seismic Zone 1. However, superstructure connections, bridge seat dimensions and support lengths at expansion bearings should all be designed per LRFD Articles 3.10.9 and 4.7.4.4.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak Ground Acceleration coefficient (PGA) = 0.067g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.233g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.105g
- Site Class D (stiff soil with $15 < N_{avg} < 50$ blows per foot)
- Seismic Zone 1, based on an $S_{D1} < 0.15$g

7.8 Construction Considerations

7.8.1 Excavation

We anticipate that the existing abutments and pier will be removed in their entirety using staged construction methods. Earth support systems, shoring or braced excavations may be needed.

Construction of the new abutment structures, new pier and new retaining walls will require soil excavation. Earth support systems may be required at the abutment locations. The pier foundation excavation will require an internally braced earth support structure.

Surface water should be diverted from the foundation excavation throughout the period of construction. We recommend removing any groundwater encountered at the base of the foundation excavation by using a sump pump located in a corner of the excavation outside of the foundation footprint.

7.8.2 Subgrade Preparation

The silty native soils at the site are susceptible to water softening, rutting and/or disturbance as a result of exposure to water or construction activity. The contractor must protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and/or rutting occur, we recommend that the contractor remove the disturbed soil materials and replace it with compacted granular borrow.

If any of the abutment or wing wall footings will be designed to bear on existing fill soils, over-excavate the footing location 2 feet and replace the excavated soil with granular borrow. Prior to placing the granular borrow, we recommend that the contractor subject the fill subgrade soil to Proctor testing and then compact the fill subgrade to 95% of the maximum dry density as determined by AASHTO T-180. The contractor should subsequently place granular borrow up to the proposed new subgrade level and compact it to 95% of the AASHTO T-180 maximum dry density prior to constructing the footings.

7.8.3 Dewatering

The silty native glacial till soils at the site may become saturated and water seepage may occur during construction potentially resulting in localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater and surface water infiltration to permit construction in-the-dry. We recommend that the contractor use temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert surface water and groundwater if significant seepage is encountered during construction. We also recommend using French drains daylighted to nearby ditches if significant seepage is encountered in the subgrade along the construction areas.

7.8.4 Reuse of Excavated Soil and Bedrock

The project plans call for excavation of the existing approach areas to achieve planned grades. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and any granular fill excavation may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

We do not recommend using excavated native soils as fill directly beneath the pavement structure. The silty native soils are typically susceptible to strength loss when wet or disturbed. The excavated soils may be allowed as fill in accordance with the Standard Specification 203 as shown on Standard Detail 203 (01). This soil may also be used for dressing slopes, but only below the bottom elevation of the shoulder subbase gravel.

The native silty soils or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that prior to placement and compaction it may be necessary to spread out and dry portions of these soils that are excessively moist.

7.8.5 Embankment Areas Outside of Abutment/Wingwall Backfill Envelope

Embankment approach slopes that are created or extended as part of the bridge construction effort should be designed as earth fill slopes no steeper than 2:1 (H:V). Slopes steeper than 2:1 (H:V) typically require reinforcement or rock fill surfacing.

We recommend that all new embankment fill be thoroughly and systematically compacted to the full limit of the slope. Where new fill slope extensions are constructed over existing slopes, we recommend benching the existing slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, to prevent creation of a preferential slip plane under the new embankment fill.

7.8.6 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices February 2008 guidelines to minimize erosion of the fine-grained soils at the site.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement of the Union Street Bridge carrying Route 222 over Interstate 95 in Bangor, Maine. We have prepared the report in accordance with generally accepted

soil 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. Further, the analyses and recommendations are based in part upon limited soil explorations completed at discrete locations on the project 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.

We recommend that we be provided the opportunity for a general review of the final design drawings and specifications in order that we may verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

REFERENCES

AASHTO, (2010), AASHTO LRFD Bridge Design Specifications, Fifth Edition, 2010, AASHTO, Washington, D.C.

Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York, NY.

Fang, Hsai-Yang (1991), Foundation Engineering Handbook, Second Edition, Van Nostrand Reinhold, New York, NY.

MaineDOT, (2003), Bridge Design Guide, MaineDOT Bridge Program, Augusta, ME.

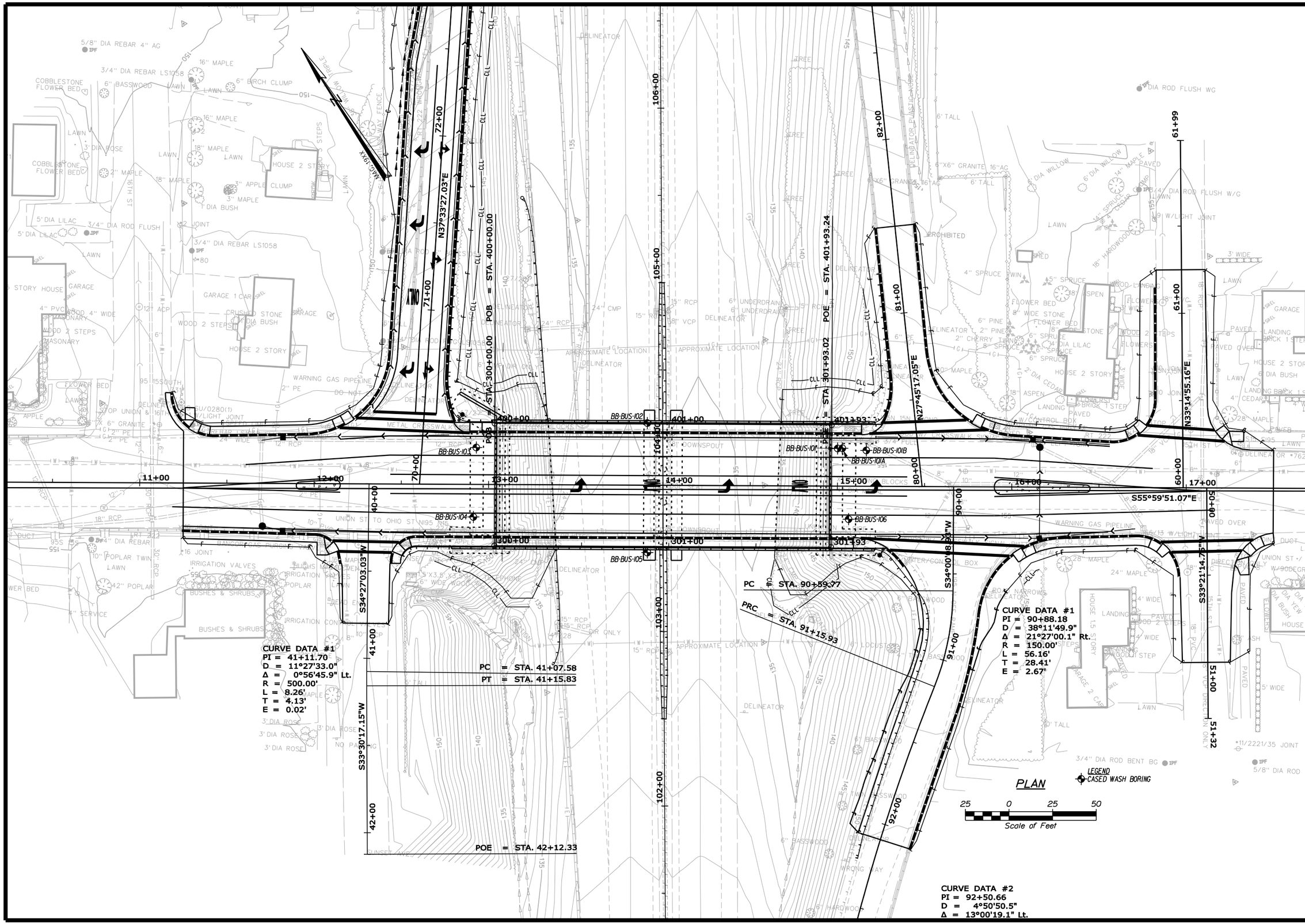
Sheets

Location Map

Sheet 1

Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.



CURVE DATA #1
PI = 41+11.70
D = 11°27'33.0"
Δ = 0°56'45.9" Lt.
R = 500.00'
L = 8.26'
T = 4.13'
E = 0.02'

CURVE DATA #1
PI = 90+88.18
D = 38°11'49.9"
Δ = 21°27'00.1" Rt.
R = 150.00'
L = 56.16'
T = 28.41'
E = 2.67'

CURVE DATA #2
PI = 92+50.66
D = 4°50'50.5"
Δ = 13°00'19.1" Lt.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

IM-1668(100)E

PIN 16681.00

BRIDGE NO. 5797

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M.MOREAU	T.WHITE	OCT 2009
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

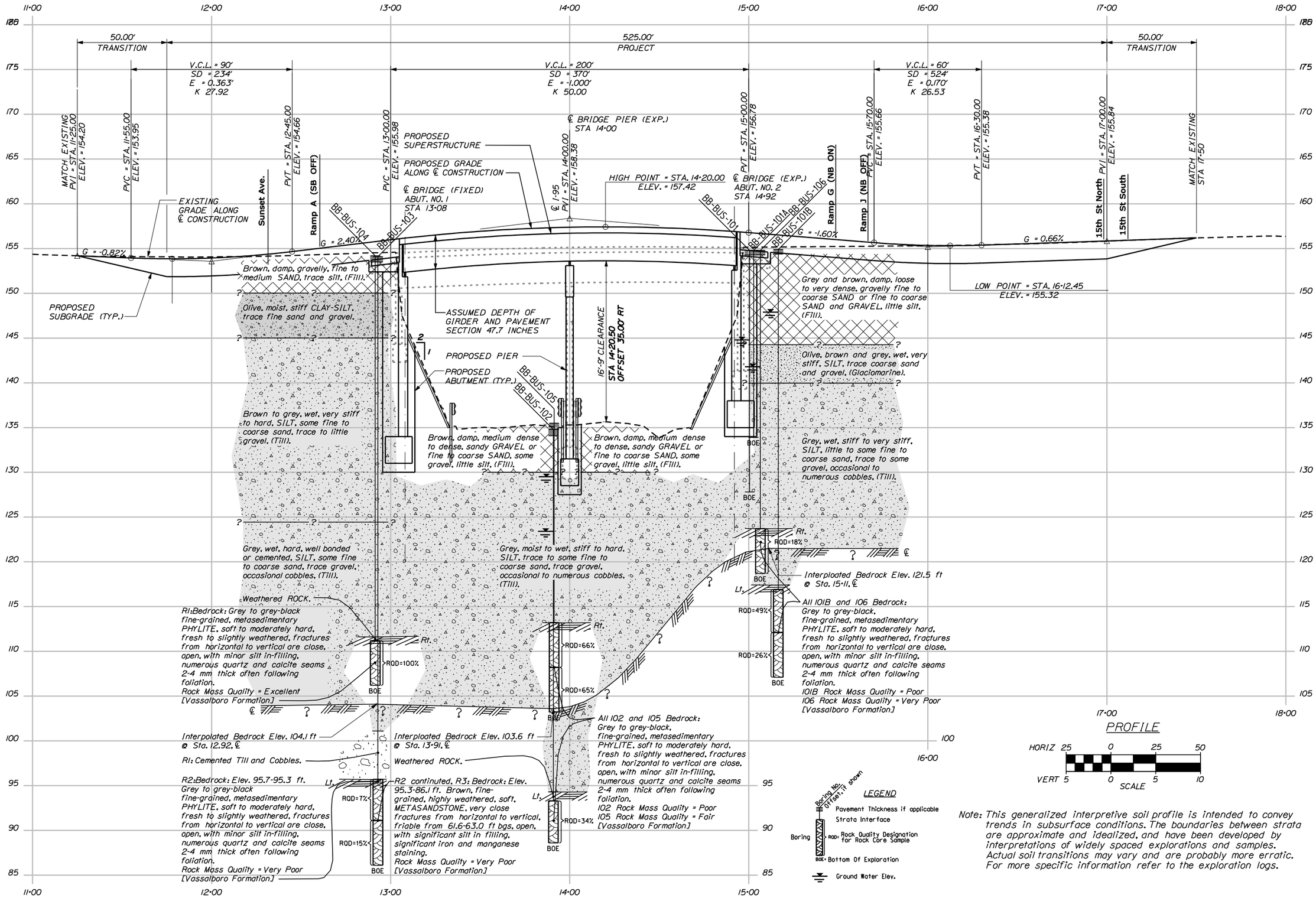
DATE

UNION STREET BRIDGE
OVERPASS I-95
BANGOR PENOBSCOT COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 5

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:5/16/2011



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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

IM-1668(100)E

PIN 16681.00

BRIDGE NO. 5797

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M.MOREAU	MAY 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

UNION STREET BRIDGE
OVERPASS I-95
BANGOR PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Filename: ...\00\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:5/17/2011

Filename: ...\msta\008_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:5/16/2011

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-103
				PIN:	16681.00
Driller:	MaineDOT	Elevation (ft.)	154.1	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/M. Moreau	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/09-5/21/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+92.9, 21.1 Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	
5	1D	24/17	5.00 - 7.00	5/4/4/6	8	11		G#209243 A-4, CL WC=20.5% LL=31 PL=22 PI=9
7	MV		7.00 - 7.00					
10	2D	24/24	10.00 - 12.00	5/4/8/8	12	17	20	G#209244 A-4, ML WC=13.0%
11							32	
12	MV		12.00 - 12.00				53	
13							88	
14							21	
15	3D	24/16	15.00 - 17.00	25/16/14/16	30	42	43	G#209245 A-4, ML WC=10.7%
16							55	
17							69	
18							69	
19							45	
20	4D	24/5	20.00 - 22.00	6/6/7/9	13	18	43	
21							42	
22							43	
23							44	
24							21	
25	5D	24/20	25.00 - 27.00	9/8/9/9	17	24	26	G#209246 A-4, ML WC=12.8%
26							27	
27							27	
28							27	
29							21	
30	6D	24/5	30.00 - 32.00	10/13/14/21	27	38	25	
31							27	
32							29	
33							34	
34							32	
35	7D	21.6/20	35.00 - 36.80	11/17/23/50(3.6)	40	56	43	
36							50	
37							49	
38							50	
39							41	
40	8D	24/10	40.00 - 42.00	20/32/40/45	72	101	44	
41							39	
42							40	
43							45	
44							85	
45	9D	24/15	45.30 - 47.30	11/17/19/50	36	50	63	
46							65	
47							131	
48							184	
49							53	
50	10D	24/18	50.00 - 52.00	11/43/29/26	72	101	43	
51							52	
52							100	
53	R1	45.6/27	53.00 - 56.80	RQD = N/A%			91 NQ-2	
54							69	
55							212	
56							131	
57							110	
58	R2	55.2/55.2	58.40 - 63.00	RQD = 7%			a20 NQ-2	
63	R3	60/60	63.00 - 68.00	RQD = 15%				

Visual Description and Remarks (Elevation (ft.) / Depth):

- 0.00–0.60 (153.50): PAVEMENT.
- 0.60–1.10 (153.00): Brown, damp, gravelly, fine to medium SAND, trace silt.
- 1.10–2.00 (152.10): Concrete with re-bar from 1.1-2.0' bgs.
- 2.00–4.50 (149.60): Brown, damp, gravelly, fine to medium SAND, trace silt.
- 4.50–9.00 (145.10): Olive, moist, stiff, CLAY-SILT, trace fine sand, trace fine gravel, (Glaciomarine). Failed 16x32 mm vane attempt, would not push.
- 9.00–20.00 (134.10): Brown, wet, very stiff to hard, SILT, some fine to coarse sand, trace to little gravel, (Till). Failed 16x32 mm vane attempt, would not push.
- 20.00–30.00 (124.10): Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, (Till). Roller Coned ahead to 25.0' bgs. Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, (Till). Roller Coned ahead to 6D,7D,8D,9D and 10D, took samples, then drove casing to 53.0' bgs.
- 30.00–53.00 (101.10): Grey, wet, hard, well bonded or cemented, SILT, some fine to coarse sand, trace gravel, with occasional cobbles, (Till). Similar to above. Cobble from 36.8-37.0' bgs. Similar to above. Cobble from 45.0-45.3' bgs. Similar to above. Grey, wet, hard, well bonded or cemented, SILT, some fine to coarse sand, trace gravel, with occasional cobbles, (Till). Roller Coned ahead to 53.0' bgs.
- 53.00–58.40 (95.70 / 95.30): R1:Cemented Till and Cobbles. R1:Core Times (min:sec) 53.0-54.0' (3:15) 54.0-55.0' (3:08) 55.0-56.0' (3:28) 56.0-56.8' (2:52) 60% Recovery Core Blocked. a20 blows for 0.2'. Casing REFUSAL at 58.2' bgs. Washed ahead to 58.4' bgs.
- 58.40: Top of Bedrock at Elev. 95.7'.
- 58.40–58.80: R2 Bedrock at 58.4' to 58.8' bgs: Grey, fine-grained PHYLLITE, moderately hard, slightly weathered, fractures from horizontal to vertical, close, open, with minor silt in-filling, numerous quartz and calcite seams 2-5 mm thick often along foliation. Rock Mass Quality is Very Poor. [Vassalboro Formation]
- 58.80–68.00 (86.10): R2 Bedrock at 58.8' to 63.0' bgs: Brown, fine-grained, highly weathered, soft, METASANDSTONE, very close fractures from horizontal to vertical, friable from 61.6'-63.0' bgs, open, with significant silt in filling, significant iron and manganese staining. Rock Mass Quality is Very Poor. [Vassalboro Formation] R2:Core Times (min:sec) 58.4-59.4' (5:52) 59.4-60.4' (5:16) 60.4-61.4' (4:34) 61.4-62.4' (4:33) 62.4-63.0' (6:44) 100% Recovery. R3 Bedrock: Same as above, but no friable zone, less silt in-filling. Rock Mass Quality is Very Poor. [Vassalboro Formation] R3:Core Times (min:sec) 63.0-64.0' (7:53) 64.0-65.0' (3:43) 65.0-66.0' (7:03) 66.0-67.0' (5:39) 67.0-68.0' (5:38) 100% Recovery
- Bottom of Exploration at 68.00 feet below ground surface.

Remarks:

Less than 100 psi down pressure on core barrel.

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-BUS-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-104
				PIN:	16681.00
Driller:	MaineDOT	Elevation (ft.)	154.2	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/18/09-5/18/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+91.4, 18.7 Rt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	
1	1D	12/12	1.00 - 2.00	13/50	---			
5	2D	24/18	5.00 - 7.00	4/3/2/2	5	7		G#209247 A-4, ML WC=13.4%
10	V1 / 3D	24/20	10.00 - 10.11 / 10.00 - 12.00	Su=4323 psf push thru vane / WOH/3/4/5	7	10	6	G#209248 A-4, ML WC=16.2%
11							18	
12							64	
13							72	
14							29	
15	4D	24/20	15.00 - 17.00	8/8/8/12	16	22	23	
16							21	
17							19	
18							21	
19							18	
20	5D	24/24	20.00 - 22.00	7/10/12/18	22	31	20	G#209249 A-4, ML WC=11.1%
21							24	
22							18	
23							24	
24							22	
25	6D	24/18	25.00 - 27.00	9/8/10/16	18	25	30	
26							28	
27							32	
28							38	
29							25	
30	7D	24/16	30.00 - 32.00	7/12/13/17	25	35	30	G#209250 A-4, ML WC=11.8%
31							31	
32							42	
33							48	
34							OPEN HOLE	
35	8D	24/18	35.00 - 37.00	14/20/35/52	55	77		
40	9D	24/24	40.00 - 42.00	12/22/29/38	51	71		G#246276 A-4, ML WC=11.2%
43	R1	60/60	43.00 - 48.00	RQD = 100%			NQ-2	

Visual Description and Remarks (Elevation (ft.) / Depth):

- 0.00–0.70 (153.50): PAVEMENT.
- 0.70–2.00 (152.20): Brown, damp, gravelly, fine to coarse SAND, little silt, (Fill).
- 2.00–2.80 (151.40): Concrete with re-bar from 2.0-2.8' bgs.
- 2.80–4.50 (149.70): Brown, damp, gravelly, fine to coarse SAND, little silt (Fill).
- 4.50–18.50 (135.70): Olive, moist to wet, medium stiff to hard, SILT, trace to some fine to coarse sand, trace to little gravel, (Till). 16x32 mm raw torque readings: V1: 27.5 ft-lbs. Went Open Hole, took samples every 5.0': 4D,5D,6D and 7D then drove casing to 35.0' bgs.
- 18.50–42.60 (111.60): Grey, wet, very stiff to hard, well bonded, SILT, some fine to coarse sand, trace to little gravel, (Till). Grey, wet, very stiff to hard, well bonded, SILT, some fine to coarse sand, trace to little gravel, (Till). Similar to above. Similar to above. Similar to above.
- 42.60–43.00 (111.20): Weathered ROCK.
- 43.00: Top of Intact Bedrock at Elev. 111.2'.
- 43.00–48.00 (106.20): R1 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, slightly weathered, fractures are moderately close, from horizontal to vertical, with minor silt in-filling, quartz and calcite seams 2-4 mm thick. Rock Mass Quality is Excellent. [Vassalboro Formation] R1:Core Times (min:sec) 43.0-44.0' (2:47) 44.0-45.0' (2:00) 45.0-46.0' (2:30) 46.0-47.0' (2:35) 47.0-48.0' (2:46) 100% Recovery
- Bottom of Exploration at 48.00 feet below ground surface.

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-BUS-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-102
				PIN:	16681.00
Driller:	Northern Test Boring, Inc.	Elevation (ft.)	134.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick, Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau	Rig Type:	Dietrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/20/09-5/20/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+90.9, 35.6 Lt.	Casing ID/OD:	HW	Water Level*:	11.4' bgs.
Hammer Efficiency Factor: 0.68		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	
2	1D	24/12	2.00 - 4.00	15/15/23/10	38	43		G#209238 A-1-b, SM WC=5.3%
5	2D/AB	24/20	5.00 - 7.00	10/13/13/15	26	29		G#209239 A-4, ML WC=10.2%
10	3D	24/15	10.00 - 12.00	6/5/6/7	11	12	62	G#209240 A-4, ML WC=12.8%
11							67	
12							99	
13							112	
14							121	
15	4D	24/17	15.00 - 17.00	8/7/12/36	19	22	53	
16							90	
17							128	
18							137	
19							145	
20	5D	24/24	20.00 - 22.00	16/15/20/31	35	40	49	G#209241 A-4, ML WC=11.8%
21							84	
22							97	
23							91	
24							118	
25	6D	24/7	25.00 - 27.00	11/30/46/63	76	86	145	
26							243	
27							277	
28							262	
29							OPEN HOLE	
30	7D	24/8	30.00 - 32.00	15/20/29/39	49	56		G#209242 A-4, SM WC=9.3%
35	MD	2/0	35.00 - 35.17	50(1.2")	---			
40							RC	
41	R1	56.4/41	41.50 - 46.20	RQD = 34%			NQ-2	

Visual Description and Remarks (Elevation (ft.) / Depth):

- 0.00–0.70 (134.10): PAVEMENT.
- 0.70–5.90 (128.90): Brown, damp, medium dense to dense, fine to coarse SAND, some gravel, little silt, (Fill). (2D/A) 5.0-5.9 bgs.
- 5.90–14.50 (120.30): (2D/B) 5.9-7.0' bgs. Grey, moist, stiff to very stiff SILT, some fine to coarse sand, trace gravel, mottled, (Till). Same as above, but wet after 11.4' bgs.
- 14.50: Same as above but numerous cobbles and boulders after 14.5' bgs. Washed ahead to 20.0' bgs, then drove casing.
- Grey, wet, hard, well bonded, SILT, trace fine to coarse sand and gravel, (Till). Washed ahead to 25.0' and 30.0', took samples, then drove casing to 29.0' bgs.
- Grey, wet, hard, well bonded, SILT, trace fine to coarse sand and gravel, (Till).
- Grey, wet, hard, well bonded, SILT, some fine to coarse sand and gravel, (Till). Very dense cemented Till at 32.8' bgs, as evidenced by drill attitude. Failed sample attempt. Washed ahead to bedrock at 40.5' bgs.
- 40.50–41.50 (94.30 / 93.30): Roller Coned ahead through Weathered BEDROCK from 40.5-41.5' bgs.
- 41.50: Top of Intact Bedrock at Elev. 93.3'.
- 41.50–46.20 (88.60): R1 Bedrock: Grey, fine-grained, metasedimentary PHYLLITE, moderately hard, slightly weathered, but highly weathered with iron and manganese staining between 42.6-43.6', fractures from horizontal to vertical, close, open, with minor silt in-filling, numerous quartz and calcite seams 2-5 mm thick along foliation. Rock Mass Quality is Poor. [Vassalboro Formation] R1: Core Times (min:sec) 41.5-42.5' (3:55) 42.5-43.5' (2:29) may be mud seam 43.5-44.5' (4:29) 44.5-45.5' (7:10) 45.5-46.2' (4:40) 73% Recovery
- Bottom of Exploration at 46.20 feet below ground surface.

Remarks:

Auto Hammer #149

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-BUS-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-105
				PIN:	16681.00
Driller:	MaineDOT	Elevation (ft.)	135.5	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/20/09; 10:00-14:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+91.3, 38.6 Rt.	Casing ID/OD:	NW	Water Level*:	6.0' bgs.
Hammer Efficiency Factor: 0.84		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	
2	1D	24/15	2.00 - 4.00	5/6/7/7	13	18		G#246277 A-1-a, GP-GM WC=2.4%
5	2D	24/16	5.00 - 7.00	7/9/14/11	23	32		G#246278 A-4, ML WC=11.1%
10	3D	24/14	10.00 - 12.00	5/13/14/14	27	38	22	G#246279 A-4, ML WC=11.8%
11							36	
12							60	
13							73	
14							61	
15	4D	24/24	15.00 - 17.00	5/13/16/24	29	41	65	
16							139	
17							203	
18							370	
19	5D	24/24	19.00 - 21.00	13/18/27/30	45	63	104	
20							156	
21							117	
22	R1	60/60	22.30 - 27.30	RQD = 66%			NQ-2	
27	R2	60/60	27.30 - 32.30	RQD = 65%				

Visual Description and Remarks (Elevation (ft.) / Depth):

- 0.00–0.70 (134.80): PAVEMENT.
- 0.70–4.50 (131.00): Brown, damp, medium dense, fine to coarse sandy GRAVEL, trace silt, (Fill).
- 4.50–9.00 (126.50): Grey, wet, hard, SILT, some fine to coarse sand, trace gravel, (Till).
- 9.00–21.80 (113.70): Grey, wet, hard, SILT, some fine to coarse sand, little gravel, (Till). Similar to above. Roller Coned ahead to 19.0' bgs. Similar to above. Roller Coned ahead to 22.3' bgs. Bedrock encountered at 21.8' bgs.
- 21.80: Top of Bedrock at Elev. 113.7'.
- 21.80–32.30 (103.20): R1 and R2 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, fresh, fractures are close, from horizontal to vertical, with minor silt in-filling and iron staining. Two large calcite seams approximately 5 cm thick between 22.3' and 23.2' bgs, two 3 cm thick calcite seams at 30.0' and 30.3' bgs, otherwise numerous quartz and calcite seams 2-4 mm thick often following foliation. Rock Mass Quality is Fair. [Vassalboro Formation] R1:Core Times (min:sec) 22.3-23.3' (4:15) 23.3-24.3' (4:00) 24.3-25.3' (4:03) 25.3-26.3' (3:55) 26.3-27.3' (3:10) 100% Recovery. R2:Core Times (min:sec) 27.3-28.3' (3:03) 28.3-29.3' (3:05) 29.3-30.3' (3:07) 30.3-31.3' (3:57) 31.3-32.3' (3:50) 100% Recovery
- Bottom of Exploration at 32.30 feet below ground surface.

Remarks:

300-400 psi down pressure on core barrel.

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-BUS-105

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

IM-1668(100)E

BRIDGE PLANS

PIN 16681.00

BRIDGE NO. 5797

SIGNATURE | P.E. NUMBER | DATE

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M.MOREAU	MAY 2011
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

UNION STREET BRIDGE
OVERPASS I-95
BANGOR PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER 4 OF 5

Filename: ...\msta\009_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:5/16/2011

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

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-101
				PIN:	16681.00
Driller:	Northern Test Boring, Inc.	Elevation (ft.)	154.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick, Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau	Rig Type:	Dietrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/09-5/19/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	15+00.5, 21.6 Lt.	Casing ID/OD:	HW	Water Level*:	10.0' bgs.
Hammer Efficiency Factor: 0.68		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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
0							SSA
5	1D	24/3	5.00 - 7.00	8/8/7/7	15	17	58
							54
							54
							70
							52
10	2D	24/4	10.00 - 12.00	8/8/5/6	13	15	37
							37
							28
							21
							21
15	3D	24/20	15.00 - 17.00	4/7/9/9	16	18	23
							26
	4D	24/16	17.50 - 19.50	9/12/14/15	26	29	23
							105
							71
20	5D	24/18	20.00 - 22.00	10/13/10/17	23	26	31
							ɑ79 OPEN
							HOLE
25	6D	24/20	25.00 - 27.00	14/9/14/20	23	26	

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
154.27	0.53	PAVEMENT.	
153.00	1.80	Brown, gravelly SAND, little silt, (Fill).	
151.60	3.20	Concrete Approach Slab.	
141.50	13.30	Brown, damp, medium dense, fine to coarse SAND, some gravel to gravelly, little silt, (Fill). Same as above, but siltier and wet after 11.0' bgs.	
		Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, occasional cobbles, (Till).	G#209232 A-4, ML WC=16.1%
		Cobble obstruction at 18.6' bgs, back out casing to 15.0' bgs, and continued with open hole to 25.0' bgs, then drove casing to 20.0' bgs.	G#209233 A-4, ML WC=14.5%
		More frequent cobbles between 18.0-25.0' bgs. ɑ79 blows for 0.6'.	G#209234 A-4, ML WC=14.5%
127.80	27.00	Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, occasional cobbles, (Till).	

Bottom of Exploration at 27.00 feet below ground surface.
Casing broke during advance to 25.0', moved to Boring BB-BUS-101A

Remarks:
Auto Hammer #149

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-BUS-101

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

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-101A
				PIN:	16681.00
Driller:	Northern Test Boring, Inc.	Elevation (ft.)	154.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick, Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau	Rig Type:	Dietrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/09-5/19/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	15+02.7, 21.7 Lt.	Casing ID/OD:	NX - 3 in	Water Level*:	None Observed
Hammer Efficiency Factor: 0.68		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer; MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); N-uncorrected = Raw field SPT N-value; Hammer Efficiency Factor = Annual Calibration Value; N_{60} = SPT N-uncorrected corrected for hammer efficiency; N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected; $S_{u(lab)}$ = Lab Vane Shear Strength (psf); WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks
	0	Augered through Pavement, Soil, Concrete and Approach Slab, drove and washed to 15.0' bgs.
		Attempted to wash ahead to 20.0' bgs.
136.80	18.00	Rubble Fill and Cobbles from 18.0-20.9' bgs.
133.90	20.90	Bottom of Exploration at 20.90 feet below ground surface. Could not advance boring. No Refusal.

Remarks:
Auto Hammer #149

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-BUS-101A

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

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-101B
				PIN:	16681.00
Driller:	Northern Test Boring, Inc.	Elevation (ft.)	154.9	Auger ID/OD:	5" Solid Stem
Operator:	Nick, Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau/B. Wilder	Rig Type:	Dietrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/09, 5/21/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+16.5, 20.6 Lt.	Casing ID/OD:	NW	Water Level*:	7.0' bgs.
Hammer Efficiency Factor: 0.68		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer; MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); N-uncorrected = Raw field SPT N-value; Hammer Efficiency Factor = Annual Calibration Value; N_{60} = SPT N-uncorrected corrected for hammer efficiency; N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected; $S_{u(lab)}$ = Lab Vane 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
0							SSA
15	1D	24/13	15.00 - 17.00	15/11/14/15	25	28	13
							31
							69
							94
							110
20	2D	24/17	20.00 - 22.00	10/10/15/18	25	28	72
							73
							66
							68
							65
25	3D	24/18	25.00 - 27.00	6/7/7/11	14	16	42
							59
							60
							58
							55
30	4D	24/4	30.00 - 32.00	5/19/36/10	55	62	49
							54
							79
							78
							75
35	5D	3.6/3	35.00 - 35.30	55(3.6")	---		
	R1	57.6/53	38.00 - 42.80	RQD = 49%			NQ-2
	R2	60/60	42.80 - 47.80	RQD = 26%			

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
154.40	0.50	PAVEMENT.	
147.90	7.00	Gravelly SAND, (Fill). Auger through pavement, soil, concrete approach slab, and drive and wash casing to 15.0'.	
141.90	13.00	Olive brown, wet, SILT, (Glaciomarine).	
		Grey, wet, very stiff, SILT, some fine to coarse sand and gravel, (Till).	G#209235 A-4, ML WC=10.0%
		Grey, wet, very stiff, SILT, little fine to coarse sand and gravel, (Till). Roller Coned ahead to 25.0' bgs.	G#209236 A-4, ML WC=11.8%
		Grey, wet, very stiff to hard, SILT, little to some fine to coarse sand and gravel, (Till).	
		Grey, wet, very stiff to hard, SILT, little to some fine to coarse sand and gravel, (Till). Roller Coned ahead to 30.0' bgs.	
		Roller Coned ahead to 35.0' bgs.	G#209237 A-4, ML WC=11.5%
119.90	35.00	Similar to above, but with cobbles. Roller Coned ahead to 38.0' bgs. Bedrock encountered at 37.5' bgs.	
117.40	37.50	Top of Bedrock at Elev. 117.4'. R1 and R2 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, slightly weathered, fractures are close, from horizontal to vertical, with minor silt in-filling, numerous quartz and calcite seams 2-4 mm thick often following foliation. Rock Mass Quality is Poor. [Vassalboro Formation] R1:Core Times (min:sec) 38.0-39.0' (5:30) 39.0-40.0' (4:20) 40.0-41.0' (4:40) 41.0-42.0' (5:50) 42.0-42.8' (4:20) 92% Recovery Core Blocked. R2:Core Times (min:sec) 42.8-43.8' (5:50) 43.8-44.8' (6:30) 44.8-45.8' (6:15) 45.8-46.8' (8:50) 46.8-47.8' (7:00) 100% Recovery	
107.10	47.80	Bottom of Exploration at 47.80 feet below ground surface.	

Remarks:
Auto Hammer #149
400 lbs down pressure on Core Barrel.

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-BUS-101B

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

Project:	Union Street Overpass I-95	Location:	Bangor, Maine	Boring No.:	BB-BUS-106
				PIN:	16681.00
Driller:	Northern Test Boring, Inc.	Elevation (ft.)	154.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick, Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau	Rig Type:	Dietrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/18/09-5/18/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	15+06.5, 18.9 Rt.	Casing ID/OD:	HW	Water Level*:	13.0' bgs.
Hammer Efficiency Factor: 0.68		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer; MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_p = Unconfined Compressive Strength (ksf); N-uncorrected = Raw field SPT N-value; Hammer Efficiency Factor = Annual Calibration Value; N_{60} = SPT N-uncorrected corrected for hammer efficiency; N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected; $S_{u(lab)}$ = Lab Vane 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
0	1D	14.4/8	0.50 - 1.70	40/29/31(2.4")	---		SSA
							139
							17
							61
5	2D	24/9	5.00 - 7.00	7/10/9/12	19	22	40
							37
							48
							44
							38
10	3D	24/3	10.00 - 12.00	2/2/3/4	5	6	39
							53
							72
							73
	4D	24/12	14.70 - 16.70	6/8/11/11	19	22	ɑ58 OPEN
15							HOLE
	5D	24/12	18.00 - 20.00	5/6/6/10	12	14	
20	6D	24/4	20.00 - 22.00	4/5/6/8	11	12	
25	7D	24/18	25.00 - 27.00	7/9/11/21	20	23	
30	8D	1.2/1	30.00 - 30.10	64(1.2")	---		RC
	R1	60/60	31.10 - 36.10	RQD = 18%			NQ-2

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
154.30	0.50	PAVEMENT.	
153.10	1.70	Grey and brown, damp, loose to very dense, fine to coarse SAND and GRAVEL, little silt, (Fill).	
152.00	2.80	Concrete Approach Slab from 1.7-2.8' bgs.	
		Same as above, but all brown.	
141.80	13.00	Same as above.	
		Grey, wet, very stiff, SILT, trace coarse sand and gravel, (Glaciomarine). qp=2.2 ksf Tv=0.65x2.5=1.625 psf ɑ58 blows for 0.7'.	G#246280 A-4, ML WC=17.5%
137.30	17.50		
		Grey, wet, stiff to very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	G#246281 A-4, ML WC=13.5%
		Grey, wet, stiff to very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	G#246282 A-4, ML WC=12.6%
125.20	29.60	Roller Cone ahead to 30.0' bgs. Bedrock encountered at 29.6' bgs.	
123.70	31.10	Spoon REFUSAL at 30.1' bgs. Weathered ROCK in spoon tip. Roller Coned ahead into weathered rock to 31.1' bgs.	
118.70	36.10	Top of Intact Bedrock at Elev. 123.7'. R1 Bedrock: Grey, fine-grained, moderately hard, PHYLLITE, slightly weathered, fractures horizontal to vertical, close, tight to open with minor silt in-filling. Rock Mass Quality is Very Poor. [Vassalboro Formation] R1:Core Times (min:sec) 31.1-32.1' (5:44) 32.1-33.1' (5:14) 33.1-34.1' (4:45) 34.1-35.1' (4:42) 35.1-36.1' (4:30) 100% Recovery	

Bottom of Exploration at 36.10 feet below ground surface.

Remarks:
Auto Hammer #149

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-BUS-106

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
IM-1668(100)E
PIN 16681.00
BRIDGE NO. 5797
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M.MOREAU	T.WHITE MAY 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

UNION STREET BRIDGE
OVERPASS I-95
BANGOR PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

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)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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 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 thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*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)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

PIN	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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-101
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/19/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	15+00.5, 21.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' bgs.
Hammer Efficiency Factor:	0.68	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	154.27	PAVEMENT. 0.53	
									Brown, gravelly SAND, little silt, (Fill).	
								153.00	1.80	
									Concrete Approach Slab.	
								151.60	3.20	
									Brown, damp, medium dense, fine to coarse SAND, some gravel to gravelly, little silt, (Fill).	
5	1D	24/3	5.00 - 7.00	8/8/7/7	15	17	58			
							54			
							54			
							70			
							52			
10	2D	24/4	10.00 - 12.00	8/8/5/6	13	15	37		Same as above, but siltier and wet after 11.0' bgs.	
							37			
							28			
							21	141.50	13.30	
							21			
15	3D	24/20	15.00 - 17.00	4/7/9/9	16	18	23		Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, occasional cobbles, (Till).	G#209232 A-4, ML WC=16.1%
							26			
	4D	24/16	17.50 - 19.50	9/12/14/15	26	29	23		Cobble obstruction at 18.6' bgs, back out casing to 15.0' bgs. and continued with open hole to 25.0' bgs, then drove casing to 20.0' bgs.	G#209233 A-4, ML WC=14.5%
							105			
							71			
20	5D	24/18	20.00 - 22.00	10/13/10/17	23	26	31		More frequent cobbles between 18.0-25.0' bgs.	G#209234 A-4, ML WC=14.5%
							a79 OPEN HOLE		a79 blows for 0.6'.	
25										

Remarks:

Auto Hammer #149

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

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

Boring No.: BB-BUS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-101
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/19/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	15+00.5, 21.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/20	25.00 - 27.00	14/9/14/20	23	26		127.80		Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, occasional cobbles, (Till).	
										27.00	
										Bottom of Exploration at 27.00 feet below ground surface. Casing broke during advance to 25.0', moved to Boring BB-BUS-101A	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #149

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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-101A
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/19/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	15+02.7, 21.7 Lt.	**Casing ID/OD:**	NX - 3 in	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										Augered through Pavement, Soil, Concrete and Approach Slab, drove and washed to 15.0' bgs.	
5											
10											
15										Attempted to wash ahead to 20.0' bgs.	
								136.80		18.00	
										Rubble Fill and Cobbles from 18.0-20.9' bgs.	
20								133.90		20.90	
										Bottom of Exploration at 20.90 feet below ground surface. Could not advance boring. No Refusal.	
25											

Remarks:

Auto Hammer #149

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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-101B
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau/B. Wilder	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09, 5/21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+16.5, 20.6 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	154.40	PAVEMENT.	
									0.50 Gravelly SAND, (Fill).	
									Auger through pavement, soil, concrete approach slab, and drive and wash casing to 15.0'.	
5										
								147.90	7.00 Olive brown, wet, SILT, (Glaciomarine).	
10										
								141.90	13.00	
15	1D	24/13	15.00 - 17.00	15/11/14/15	25	28	13		Grey, wet, very stiff, SILT, some fine to coarse sand and gravel, (Till).	G#209235 A-4, ML WC=10.0%
							31			
							69			
							94			
							110			
20	2D	24/17	20.00 - 22.00	10/10/15/18	25	28	72		Grey, wet, very stiff, SILT, little fine to coarse sand and gravel, (Till). Roller Coned ahead to 25.0' bgs.	G#209236 A-4, ML WC=11.8%
							73			
							66			
							68		Grey, wet, very stiff to hard, SILT, little to some fine to coarse sand and gravel, (Till).	
25							65			

Remarks:

Auto Hammer #149
400 lbs down pressure on Core Barrel.

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

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

Boring No.: BB-BUS-101B

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-101B
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau/B. Wilder	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09, 5/21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+16.5, 20.6 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	3D	24/18	25.00 - 27.00	6/7/7/11	14	16	42		Grey, wet, very stiff to hard, SILT, little to some fine to coarse sand and gravel, (Till).	
							59		Roller Coned ahead to 30.0' bgs.	
							60			
							58			
							55			
30	4D	24/4	30.00 - 32.00	5/19/36/10	55	62	49		Roller Coned ahead to 35.0' bgs.	G#209237 A-4, ML WC=11.5%
							54			
							79			
							78			
							75			
35	5D	3.6/3	35.00 - 35.30	55(3.6")	---			119.90	35.00 — Similar to above, but with cobbles.	
									Roller Coned ahead to 38.0' bgs. Bedrock encountered at 37.5' bgs.	
								117.40	37.50 — Top of Bedrock at Elev. 117.4'.	
	R1	57.6/53	38.00 - 42.80	RQD = 49%			NQ-2		R1 and R2 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, slightly weathered, fractures are close, from horizontal to vertical, with minor silt in-filling, numerous quartz and calcite seams 2-4 mm thick often following foliation. Rock Mass Quality is Poor. [Vassalboro Formation]	
40									R1:Core Times (min:sec) 38.0-39.0' (5:30) 39.0-40.0' (4:20) 40.0-41.0' (4:40) 41.0-42.0' (5:50) 42.0-42.8' (4:20) 92% Recovery Core Blocked.	
	R2	60/60	42.80 - 47.80	RQD = 26%					R2:Core Times (min:sec) 42.8-43.8' (5:50) 43.8-44.8' (6:30) 44.8-45.8' (6:15) 45.8-46.8' (8:50) 46.8-47.8' (7:00) 100% Recovery	
45								107.10	47.80 — **Bottom of Exploration at 47.80 feet below ground surface.**	
50										

Remarks:

Auto Hammer #149
400 lbs down pressure on Core Barrel.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-102
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	134.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/20/09-5/20/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+90.9, 35.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.4' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	134.10	PAVEMENT. 0.70	
	1D	24/12	2.00 - 4.00	15/15/23/10	38	43			Brown, damp, medium dense to dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#209238 A-1-b, SM WC=5.3%
5	2D/AB	24/20	5.00 - 7.00	10/13/13/15	26	29		128.90	(2D/A) 5.0-5.9 bgs. 5.90	
									(2D/B) 5.9-7.0' bgs. Grey, moist, stiff to very stiff SILT, some fine to coarse sand, trace gravel, mottled, (Till).	G#209239 A-4, ML WC=10.2%
10	3D	24/15	10.00 - 12.00	6/5/6/7	11	12	62		Same as above, but wet after 11.4' bgs.	G#209240 A-4, ML WC=12.8%
							67			
							99			
							112			
							121	120.30	14.50	
15	4D	24/17	15.00 - 17.00	8/7/12/36	19	22	53		Same as above but numerous cobbles and boulders after 14.5' bgs. Washed ahead to 20.0' bgs, then drove casing.	
							90			
							128			
							137			
							145		Grey, wet, hard, well bonded, SILT, trace fine to coarse sand and gravel, (Till).	
20	5D	24/24	20.00 - 22.00	16/15/20/31	35	40	49		Washed ahead to 25.0' and 30.0', took samples, then drove casing to 29.0' bgs.	G#209241 A-4, ML WC=11.8%
							84			
							97			
							91			
25							118			

Remarks:

Auto Hammer #149

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Boring No.: BB-BUS-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-102
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	134.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/20/09-5/20/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+90.9, 35.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.4' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/7	25.00 - 27.00	11/30/46/63	76	86	145		Grey, wet, hard, well bonded, SILT, trace fine to coarse sand and gravel, (Till).	
							243			
							277			
							262		Grey, wet, hard, well bonded, SILT, some fine to coarse sand and gravel, (Till).	
							OPEN HOLE			
30	7D	24/8	30.00 - 32.00	15/20/29/39	49	56				G#209242 A-4, SM WC=9.3%
									Very dense cemented Till at 32.8' bgs, as evidenced by drill attitude.	
35	MD	2/0	35.00 - 35.17	50(1.2")	---				Failed sample attempt. Washed ahead to bedrock at 40.5' bgs.	
40							RC	94.30	40.50 Roller Coned ahead through Weathered BEDROCK from 40.5-41.5' bgs.	
	R1	56.4/41	41.50 - 46.20	RQD = 34%			NQ-2	93.30	41.50 Top of Intact Bedrock at Elev. 93.3'. R1 Bedrock: Grey, fine-grained, metasedimentary PHYLLITE, moderately hard, slightly weathered, but highly weathered with iron and manganese staining between 42.6-43.6', fractures from horizontal to vertical, close, open, with minor silt in-filling, numerous quartz and calcite seams 2-5 mm thick along foliation. Rock Mass Quality is Poor. [Vassalboro Formation]	
45								88.60	R1: Core Times (min:sec) 41.5-42.5' (3:55) 42.5-43.5' (2:29) may be mud seam 43.5-44.5' (4:29) 44.5-45.5' (7:10) 45.5-46.2' (4:40) 73% Recovery 46.20	
50									**Bottom of Exploration at 46.20 feet below ground surface.**	

Remarks:

Auto Hammer #149

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-103
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	154.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/M. Moreau	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+92.9, 21.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	153.50	PAVEMENT. 0.60	
								153.00	Brown, damp, gravelly, fine to medium SAND, trace silt. 1.10	
								152.10	Concrete with re-bar from 1.1-2.0' bgs. 2.00	
									Brown, damp, gravelly, fine to medium SAND, trace silt.	
								149.60	4.50	
5	1D	24/17	5.00 - 7.00	5/4/4/6	8	11			Olive, moist, stiff, CLAY-SILT, trace fine sand, trace fine gravel, (Glaciomarine).	G#209243 A-4, CL WC=20.5% LL=31 PL=22 PI=9
	MV		7.00 - 7.00						Failed 16x32 mm vane attempt, would not push.	
								145.10	9.00	
10	2D	24/24	10.00 - 12.00	5/4/8/8	12	17	20		Brown, wet, very stiff to hard, SILT, some fine to coarse sand, trace to little gravel, (Till).	G#209244 A-4, ML WC=13.0%
							32			
	MV		12.00 - 12.00				53		Failed 16x32 mm vane attempt, would not push.	
							88			
							21			
15	3D	24/16	15.00 - 17.00	25/16/14/16	30	42	43			G#209245 A-4, ML WC=10.7%
							55			
							69			
							69			
							45			
20	4D	24/5	20.00 - 22.00	6/6/7/9	13	18	43	134.10	20.00 Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	
							42		Roller Coned ahead to 25.0' bgs.	
							43			
							44			
25							21			

Remarks:

Less than 100 psi down pressure on core barrel.

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Boring No.: BB-BUS-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-103
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	154.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/M. Moreau	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+92.9, 21.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
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N-uncorrected = Raw field SPT N-value
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/20	25.00 - 27.00	9/8/9/9	17	24	26			Grey, wet, very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	G#209246 A-4, ML WC=12.8%
							27			Roller Coned ahead to 6D,7D,8D,9D and 10D, took samples, then drove casing to 53.0' bgs.	
							27				
							27				
							21				
30								124.10		30.00	
	6D	24/5	30.00 - 32.00	10/13/14/21	27	38	25			Grey, wet, hard, well bonded or cemented, SILT, some fine to coarse sand, trace gravel, with occasional cobbles, (Till).	
							27				
							29				
							34				
							32				
35	7D	21.6/20	35.00 - 36.80	11/17/23/50(3.6)	40	56	43			Similar to above.	
							50				
							49			Cobble from 36.8-37.0' bgs.	
							50				
							41				
40	8D	24/10	40.00 - 42.00	20/32/40/45	72	101	44			Similar to above.	
							39				
							40				
							45				
							85				
45	9D	24/15	45.30 - 47.30	11/17/19/50	36	50	63			Cobble from 45.0-45.3' bgs. Similar to above.	
							65				
							131				
							184				
50							53				

Remarks:

Less than 100 psi down pressure on core barrel.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-103
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	154.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/M. Moreau	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/09-5/21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+92.9, 21.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/18	50.00 - 52.00	11/43/29/26	72	101	43		Grey, wet, hard, well bonded or cemented, SILT, some fine to coarse sand, trace gravel, with occasional cobbles, (Till).	
							52		Roller Coned ahead to 53.0' bgs.	
							100			
	R1	45.6/27	53.00 - 56.80	RQD = N/A%			91	101.10	53.00	
							NQ-2		R1:Cemented Till and Cobbles.	
							69		R1:Core Times (min:sec)	
55									53.0-54.0' (3:15)	
							212		54.0-55.0' (3:08)	
									55.0-56.0' (3:28)	
							131		56.0-56.8' (2:52) 60% Recovery	
									Core Blocked	
							110			
	R2	55.2/55.2	58.40 - 63.00	RQD = 7%			a20	95.70	a20 blows for 0.2'.	
							NQ-2	95.30	Casing REFUSAL at 58.2' bgs. Washed ahead to 58.4' bgs.	
									58.40	
60									Top of Bedrock at Elev. 95.7'.	
									R2 Bedrock at 58.4' to 58.8' bgs: Grey, fine-grained PHYLLITE, moderately hard, slightly weathered, fractures from horizontal to vertical, close, open, with minor silt in-filling, numerous quartz and calcite seams 2-5 mm thick often along foliation. Rock Mass Quality is Very Poor. [Vassalboro Formation]	
									58.80	
	R3	60/60	63.00 - 68.00	RQD = 15%					R2 Bedrock at 58.8' to 63.0' bgs: Brown, fine-grained, highly weathered, soft, METASANDSTONE, very close fractures from horizontal to vertical, friable from 61.6'-63.0' bgs, open, with significant silt in filling, significant iron and manganese staining. Rock Mass Quality is Very Poor. [Vassalboro Formation]	
65									R2:Core Times (min:sec)	
									58.4-59.4' (5:52)	
									59.4-60.4' (5:16)	
									60.4-61.4' (4:34)	
									61.4-62.4' (4:33)	
									62.4-63.0' (6:44) 100% Recovery	
								86.10	R3 Bedrock: Same as above, but no friable zone, less silt in-filling. Rock Mass Quality is Very Poor. [Vassalboro Formation]	
									R3:Core Times (min:sec)	
70									63.0-64.0' (7:53)	
									64.0-65.0' (3:43)	
									65.0-66.0' (7:03)	
									66.0-67.0' (5:39)	
									67.0-68.0' (5:38) 100% Recovery	
									68.00	
75									**Bottom of Exploration at 68.00 feet below ground surface.**	

Remarks:

Less than 100 psi down pressure on core barrel.

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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-104
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	154.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/18/09-5/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+91.4, 18.7 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	153.50	PAVEMENT. 0.70	
	1D	12/12	1.00 - 2.00	13/50	---				Brown, damp, gravelly, fine to coarse SAND, little silt, (Fill).	
								152.20	2.00	
									Concrete with re-bar from 2.0-2.8' bgs.	
								151.40	2.80	
									Brown, damp, gravelly, fine to coarse SAND, little silt (Fill).	
								149.70	4.50	
5	2D	24/18	5.00 - 7.00	4/3/2/2	5	7			Olive, moist to wet, medium stiff to hard, SILT, trace to some fine to coarse sand, trace to little gravel, (Till).	G#209247 A-4, ML WC=13.4%
10	V1 3D	24/20	10.00 - 10.11 10.00 - 12.00	Su=4323 psf push thru vane WOH/3/4/5	7	10	6		16x32 mm raw torque readings: V1: 27.5 ft-lbs	G#209248 A-4, ML WC=16.2%
							18			
							64			
							72			
							29		Went Open Hole, took samples every 5.0': 4D,5D,6D and 7D then drove casing to 35.0' bgs.	
15	4D	24/20	15.00 - 17.00	8/8/8/12	16	22	23			
							21			
							19			
							21	135.70	18.50	
							18			
20	5D	24/24	20.00 - 22.00	7/10/12/18	22	31	20		Grey, wet, very stiff to hard, well bonded, SILT, some fine to coarse sand, trace to little gravel, (Till).	G#209249 A-4, ML WC=11.1%
							24			
							18			
							24			
25							22			

Remarks:

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

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

Boring No.: BB-BUS-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-104
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	154.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/18/09-5/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+91.4, 18.7 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/18	25.00 - 27.00	9/8/10/16	18	25	30		Grey, wet, very stiff to hard, well bonded, SILT, some fine to coarse sand, trace to little gravel, (Till).	
							28			
							32			
							38			
							25			
30	7D	24/16	30.00 - 32.00	7/12/13/17	25	35	30		Similar to above.	G#209250 A-4, ML WC=11.8%
							31			
							42			
							48			
							OPEN HOLE			
35	8D	24/18	35.00 - 37.00	14/20/35/52	55	77			Similar to above.	
40	9D	24/24	40.00 - 42.00	12/22/29/38	51	71			Similar to above.	G#246276 A-4, ML WC=11.2%
								111.60	42.60	
								111.20	Weathered ROCK. 43.00	
	R1	60/60	43.00 - 48.00	RQD = 100%			NQ-2		R1 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, slightly weathered, fractures are moderately close, from horizontal to vertical, with minor silt in-filling, quartz and calcite seams 2-4 mm thick. Rock Mass Quality is Excellent. [Vassalboro Formation] R1:Core Times (min:sec) 43.0-44.0' (2:47) 44.0-45.0' (2:00) 45.0-46.0' (2:30) 46.0-47.0' (2:35) 47.0-48.0' (2:46) 100% Recovery	
45								106.20	48.00	
50									**Bottom of Exploration at 48.00 feet below ground surface.**	

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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-105
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	135.5	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/20/09; 10:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+91.3, 38.6 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	6.0' bgs.
Hammer Efficiency Factor:	0.84	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	134.80	PAVEMENT. 0.70	
	1D	24/15	2.00 - 4.00	5/6/7/7	13	18			Brown, damp, medium dense, fine to coarse sandy GRAVEL, trace silt, (Fill).	G#246277 A-1-a, GP-GM WC=2.4%
								131.00	4.50	
5	2D	24/16	5.00 - 7.00	7/9/14/11	23	32			Grey, wet, hard, SILT, some fine to coarse sand, trace gravel, (Till).	G#246278 A-4, ML WC=11.1%
								126.50	9.00	
10	3D	24/14	10.00 - 12.00	5/13/14/14	27	38	22		Grey, wet, hard, SILT, some fine to coarse sand, little gravel, (Till).	G#246279 A-4, ML WC=11.8%
							36			
							60			
							73			
							61			
15	4D	24/24	15.00 - 17.00	5/13/16/24	29	41	65		Similar to above.	
							139		Roller Coned ahead to 19.0' bgs.	
							203			
							370			
	5D	24/24	19.00 - 21.00	13/18/27/30	45	63	104		Similar to above.	
20							156		Roller Coned ahead to 22.3' bgs. Bedrock encountered at 21.8' bgs.	
							117			
	R1	60/60	22.30 - 27.30	RQD = 66%			NQ-2	113.70	21.80 Top of Bedrock at Elev. 113.7'. R1 and R2 Bedrock: Grey to grey-black fine-grained, metasedimentary PHYLLITE, soft to moderately hard, fresh, fractures are close, from horizontal to vertical, with minor silt in-filling and iron staining. Two large calcite seams approximately 5 cm thick between 22.3' and 23.2'	
25										

Remarks:

300-400 psi down pressure on core barrel.

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Boring No.: BB-BUS-105

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-105
PIN: 16681.00

Driller:	MaineDOT	**Elevation (ft.)**	135.5	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/20/09; 10:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+91.3, 38.6 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	6.0' bgs.
Hammer Efficiency Factor:	0.84	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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										bgs, two 3 cm thick calcite seams at 30.0' and 30.3' bgs, otherwise numerous quartz and calcite seams 2-4 mm thick often following foliation. Rock Mass Quality is Fair. [Vassalboro Formation]	
										R1:Core Times (min:sec) 22.3-23.3' (4:15) 23.3-24.3' (4:00) 24.3-25.3' (4:03) 25.3-26.3' (3:55) 26.3-27.3' (3:10) 100% Recovery	
	R2	60/60	27.30 - 32.30	RQD = 65%							
30										R2:Core Times (min:sec) 27.3-28.3' (3:03) 28.3-29.3' (3:05) 29.3-30.3' (3:07) 30.3-31.3' (3:57) 31.3-32.3' (3:50) 100% Recovery	
								103.20			
										32.30	
										Bottom of Exploration at 32.30 feet below ground surface.	
35											
40											
45											
50											

Remarks:

300-400 psi down pressure on core barrel.

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: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-106
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/18/09-5/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	15+06.5, 18.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	13.0' bgs.
Hammer Efficiency Factor:	0.68	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	14.4/8	0.50 - 1.70	40/29/31(2.4")	---		SSA	154.30	PAVEMENT. 0.50	
								153.10	Grey and brown, damp, loose to very dense, fine to coarse SAND and GRAVEL, little silt, (Fill). 1.70	
							139	152.00	Concrete Approach Slab from 1.7-2.8' bgs. 2.80	
							17			
							61			
5	2D	24/9	5.00 - 7.00	7/10/9/12	19	22	40		Same as above, but all brown.	
							37			
							48			
							44			
							38			
10	3D	24/3	10.00 - 12.00	2/2/3/4	5	6	39		Same as above.	
							53			
							72			
							73	141.80	13.00	
	4D	24/12	14.70 - 16.70	6/8/11/11	19	22	[a]58		Grey, wet, very stiff, SILT, trace coarse sand and gravel, (Glaciomarine).	
15							OPEN HOLE		qp=2.2 ksf Tv=0.65x2.5=1,625 psf [a]58 blows for 0.7'.	G#246280 A-4, ML WC=17.5%
								137.30	17.50	
	5D	24/12	18.00 - 20.00	5/6/6/10	12	14			Grey, wet, stiff to very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	G#246281 A-4, ML WC=13.5%
20	6D	24/4	20.00 - 22.00	4/5/6/8	11	12				
25										

Remarks:

Auto Hammer #149

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

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Boring No.: BB-BUS-106

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Union Street Overpass I-95
Location: Bangor, Maine

Boring No.: BB-BUS-106
PIN: 16681.00

Driller:	Northern Test Boring, Inc.	**Elevation (ft.)**	154.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick, Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau	**Rig Type:**	Dietrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/18/09-5/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	15+06.5, 18.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	13.0' bgs.
Hammer Efficiency Factor:	0.68	**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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/18	25.00 - 27.00	7/9/11/21	20	23			Grey, wet, stiff to very stiff, SILT, some fine to coarse sand, trace gravel, (Till).	G#246282 A-4, ML WC=12.6%
									Roller Cone ahead to 30.0' bgs. Bedrock encountered at 29.6' bgs.	
								125.20	29.60	
30	8D	1.2/1	30.00 - 30.10	64(1.2")	---		RC		Spoon REFUSAL at 30.1' bgs, Weathered ROCK in spoon tip. Roller Coned ahead into weathered rock to 31.1' bgs.	
								123.70	31.10	
	R1	60/60	31.10 - 36.10	RQD = 18%			NQ-2		Top of Intact Bedrock at Elev. 123.7'.	
									R1 Bedrock: Grey, fine-grained, moderately hard, PHYLLITE, slightly weathered, fractures horizontal to vertical, close, tight to open with minor silt in-filling. Rock Mass Quality is Very Poor. [Vassalboro Formation]	
35									R1:Core Times (min:sec) 31.1-32.1' (5:44) 32.1-33.1' (5:14) 33.1-34.1' (4:45) 34.1-35.1' (4:42) 35.1-36.1' (4:30) 100% Recovery	
								118.70	36.10	
									Bottom of Exploration at 36.10 feet below ground surface.	
40										
45										
50										

Remarks:

Auto Hammer #149

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 Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Bangor Project Number: 16681.00

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-BUS-101, 3D	15+00.5	21.6 Lt.	15.0-17.0	209232	1	16.1			ML	A-4	IV
BB-BUS-101, 4D	15+00.5	21.6 Lt.	17.5-19.5	209233	1	14.5			ML	A-4	IV
BB-BUS-101, 5D	15+00.5	21.6 Lt.	20.0-22.0	209234	1	14.5			ML	A-4	IV
BB-BUS-101B, 1D	15+16.5	20.6 Lt.	15.0-17.0	209235	1	10.0			ML	A-4	IV
BB-BUS-101B, 2D	15+16.5	20.6 Lt.	20.0-22.0	209236	1	11.8			ML	A-4	IV
BB-BUS-101B, 4D	15+16.5	20.6 Lt.	30.0-32.0	209237	1	11.5			ML	A-4	IV
BB-BUS-102, 1D	13+90.9	35.6 Lt.	2.0-4.0	209238	2	5.3			SM	A-1-b	II
BB-BUS-102, 2D/B	13+90.9	35.6 Lt.	5.9-7.0	209239	2	10.2			ML	A-4	IV
BB-BUS-102, 3D	13+90.9	35.6 Lt.	10.0-12.0	209240	2	12.8			ML	A-4	IV
BB-BUS-102, 5D	13+90.9	35.6 Lt.	20.0-22.0	209241	2	11.8			ML	A-4	IV
BB-BUS-102, 7D	13+90.9	35.6 Lt.	30.0-32.0	209242	2	9.3			SM	A-4	III
BB-BUS-103, 1D	12+92.9	21.1 Lt.	5.0-7.0	209243	3	20.5	31	9	CL	A-4	IV
BB-BUS-103, 2D	12+92.9	21.1 Lt.	10.0-12.0	209244	3	13.0			ML	A-4	IV
BB-BUS-103, 3D	12+92.9	21.1 Lt.	15.0-17.0	209245	3	10.7			ML	A-4	IV
BB-BUS-103, 5D	12+92.9	21.1 Lt.	25.0-27.0	209246	3	12.8			ML	A-4	IV
BB-BUS-104, 2D	12+91.4	18.7 Rt.	5.0-7.0	209247	4	13.4			ML	A-4	IV
BB-BUS-104, 3D	12+91.4	18.7 Rt.	10.0-12.0	209248	4	16.2			ML	A-4	IV
BB-BUS-104, 5D	12+91.4	18.7 Rt.	20.0-22.0	209249	4	11.1			ML	A-4	IV
BB-BUS-104, 7D	12+91.4	18.7 Rt.	30.0-32.0	209250	4	11.8			ML	A-4	IV
BB-BUS-104, 9D	12+91.4	18.7 Rt.	40.0-42.0	246276	4	11.2			ML	A-4	IV
BB-BUS-105, 1D	13+91.3	38.6 Rt.	2.0-4.0	246277	5	2.4			GP-GM	A-1-a	0
BB-BUS-105, 2D	13+91.3	38.6 Rt.	5.0-7.0	246278	5	11.1			ML	A-4	IV
BB-BUS-105, 3D	13+91.3	38.6 Rt.	10.0-12.0	246279	5	11.8			ML	A-4	IV
BB-BUS-106, 4D	15+06.5	18.9 Rt.	14.7-16.7	246280	5	17.5			ML	A-4	IV
BB-BUS-106, 5D	15+06.5	18.9 Rt.	18.0-20.0	246281	5	13.5			ML	A-4	IV
BB-BUS-106, 7D	15+06.5	18.9 Rt.	25.0-27.0	246282	5	12.6			ML	A-4	IV

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

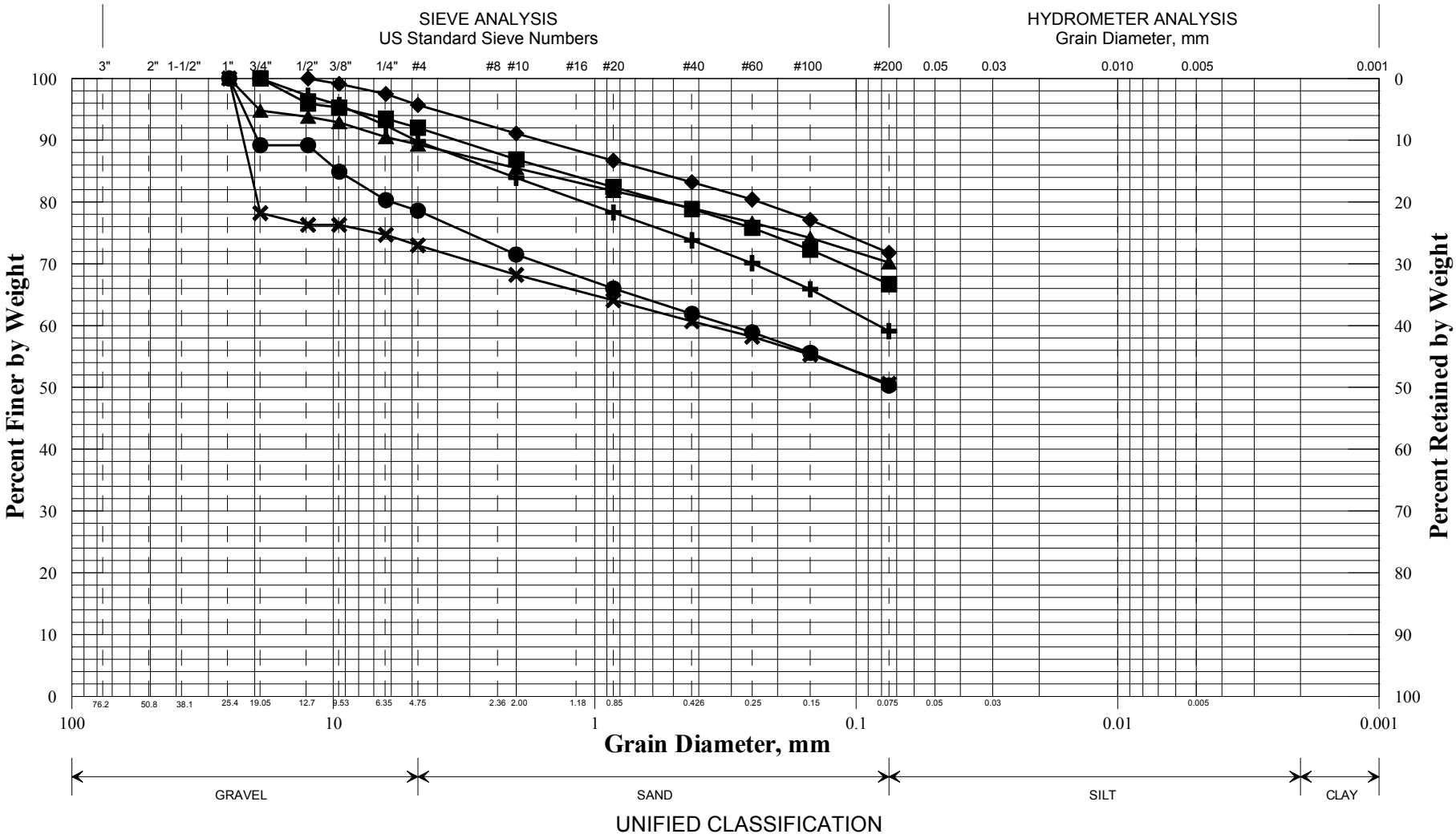
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

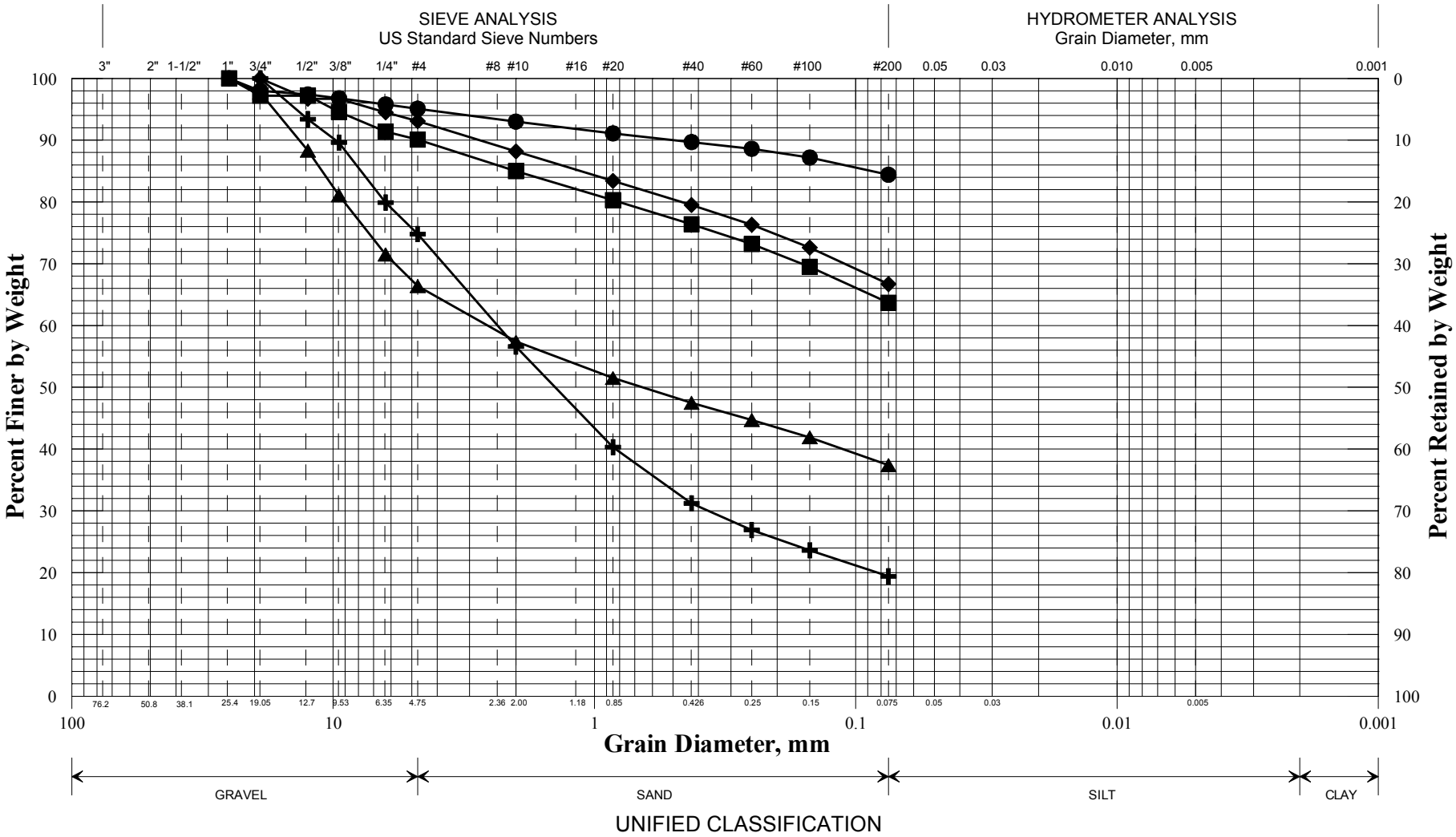
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BUS-101/3D	15+00.5	21.6 LT	15.0-17.0	SILT, some sand, trace gravel.	16.1			
◆	BB-BUS-101/4D	15+00.5	21.6 LT	17.5-19.5	SILT, some sand, trace gravel.	14.5			
■	BB-BUS-101/5D	15+00.5	21.6 LT	20.0-22.0	SILT, some sand, trace gravel.	14.5			
●	BB-BUS-101B/1D	15+16.5	20.6 LT	15.0-17.0	SILT, some sand, some gravel.	10.0			
▲	BB-BUS-101B/2D	15+16.5	20.6 LT	20.0-22.0	SILT, little sand, little gravel.	11.8			
✖	BB-BUS-101B/4D	15+16.5	20.6 LT	30.0-32.0	SILT, some gravel, some sand.	11.5			

PIN
016681.00
Town
Bangor
Reported by/Date
WHITE, TERRY A 8/31/2009

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-BUS-102/1D	13+90.9	35.6 LT	2.0-4.0	SAND, some gravel, little silt.	5.3			
◆	BB-BUS-102/2D/B	13+90.9	35.6 LT	5.9-7.0	SILT, some sand, trace gravel.	10.2			
■	BB-BUS-102/3D	13+90.9	35.6 LT	10.0-12.0	SILT, some sand, trace gravel.	12.8			
●	BB-BUS-102/5D	13+90.9	35.6 LT	20.0-22.0	SILT, trace sand, trace gravel.	11.8			
▲	BB-BUS-102/7D	13+90.9	35.6 LT	30.0-32.0	SILT, some gravel, some sand.	9.3			
✖									

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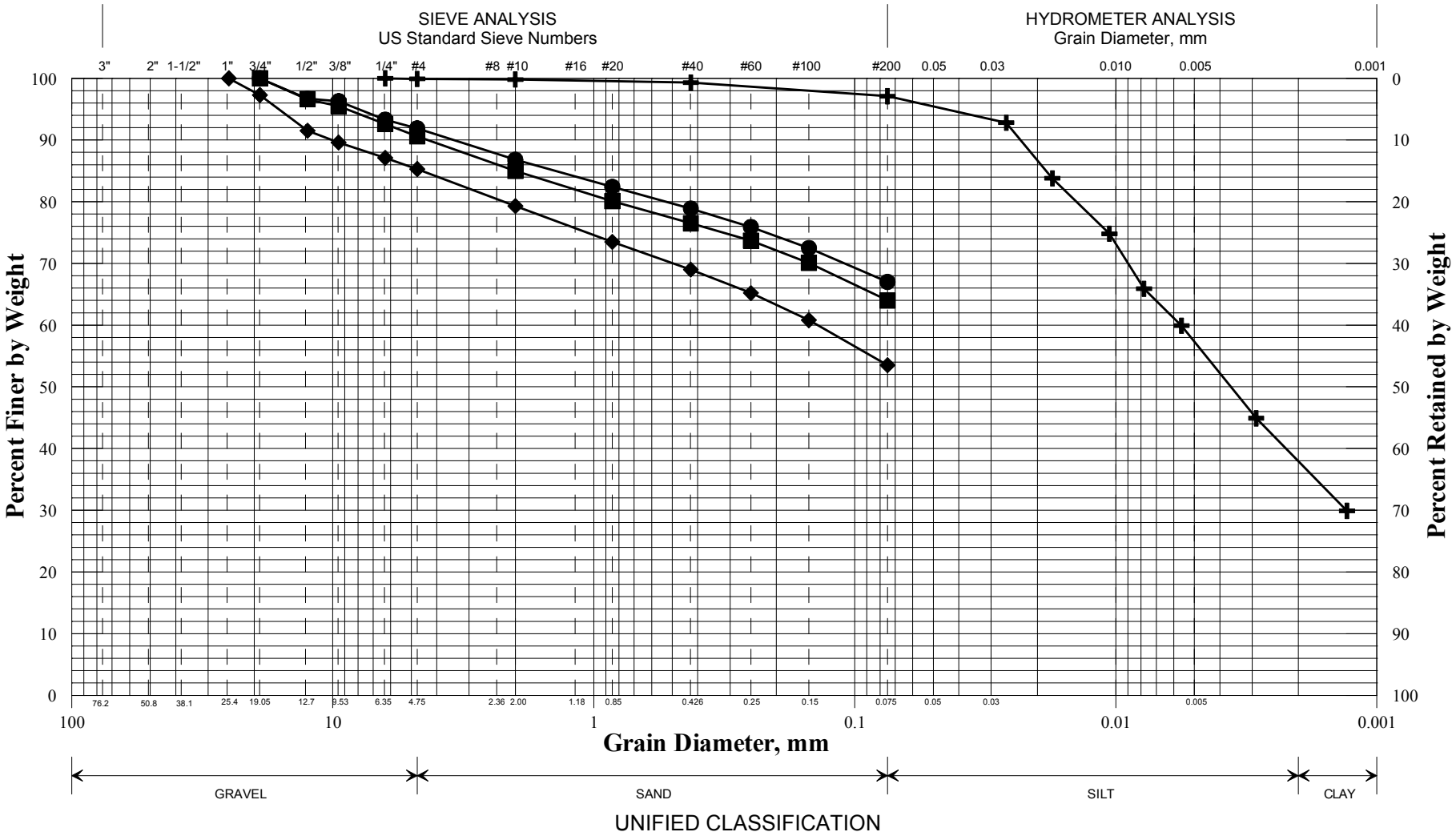
WHITE, TERRY A 8/31/2009

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-BUS-103/1D	12+92.9	21.1 LT	5.0-7.0	CLAY-SILT, trace sand, trace gravel.	20.5	31	22	9
◆	BB-BUS-103/2D	12+92.9	21.1 LT	10.0-12.0	SILT, some sand, little gravel.	13.0			
■	BB-BUS-103/3D	12+92.9	21.1 LT	15.0-17.0	SILT, some sand, trace gravel.	10.7			
●	BB-BUS-103/5D	12+92.9	21.1 LT	25.0-27.0	SILT, some sand, trace gravel.	12.8			
▲									
✖									

PIN
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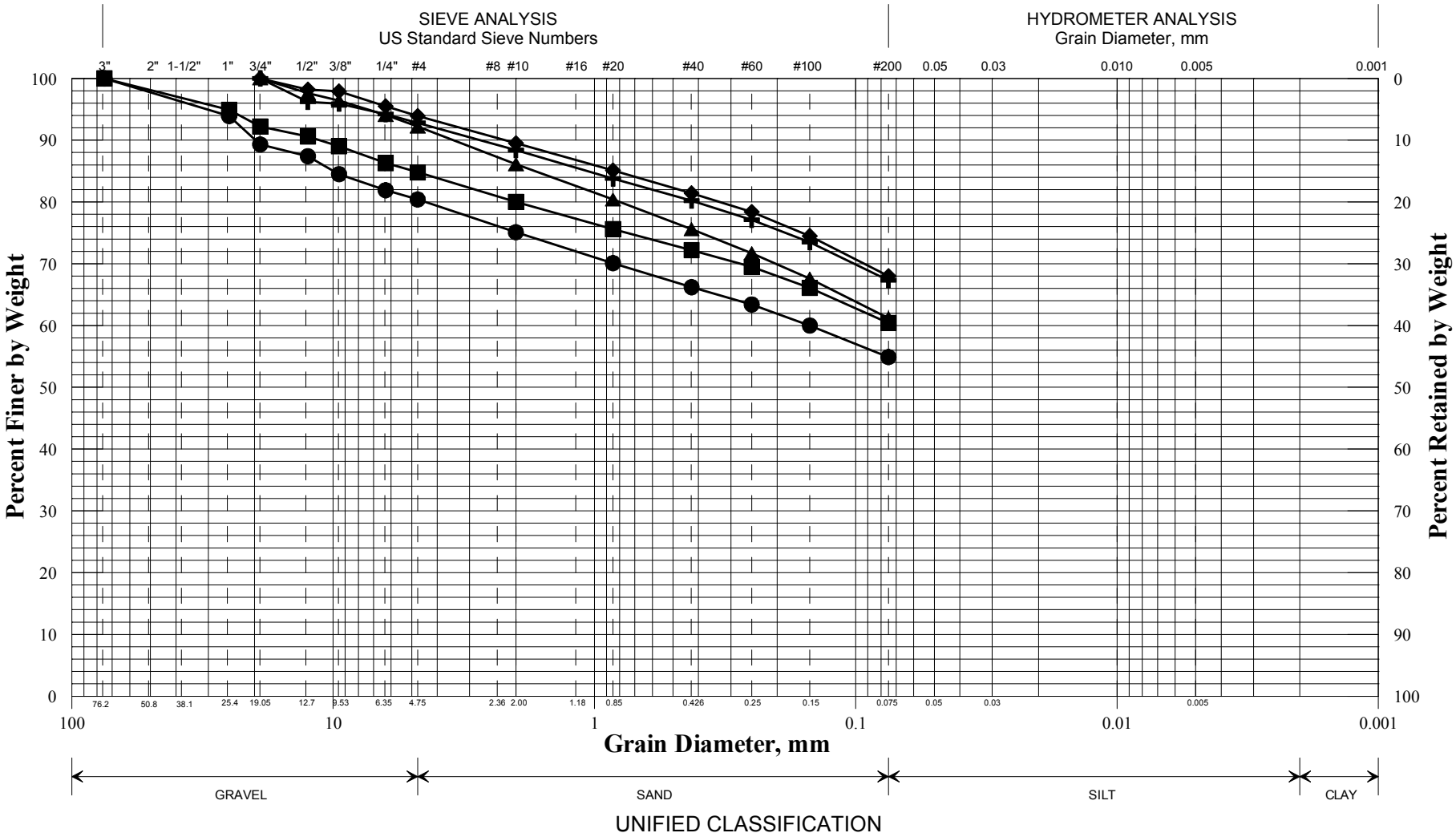
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BUS-104/2D	12+91.4	18.7 RT	5.0-7.0	SILT, some sand, trace gravel.	13.4			
◆	BB-BUS-104/3D	12+91.4	18.7 RT	10.0-12.0	SILT, some sand, trace gravel.	16.2			
■	BB-BUS-104/5D	12+91.4	18.7 RT	20.0-22.0	SILT, some sand, little gravel.	11.1			
●	BB-BUS-104/7D	12+91.4	18.7 RT	30.0-32.0	SILT, some sand, little gravel.	11.8			
▲	BB-BUS-104/9D	12+91.4	18.7 RT	40.0-42.0	SILT, some sand, trace gravel.	11.2			
✖									

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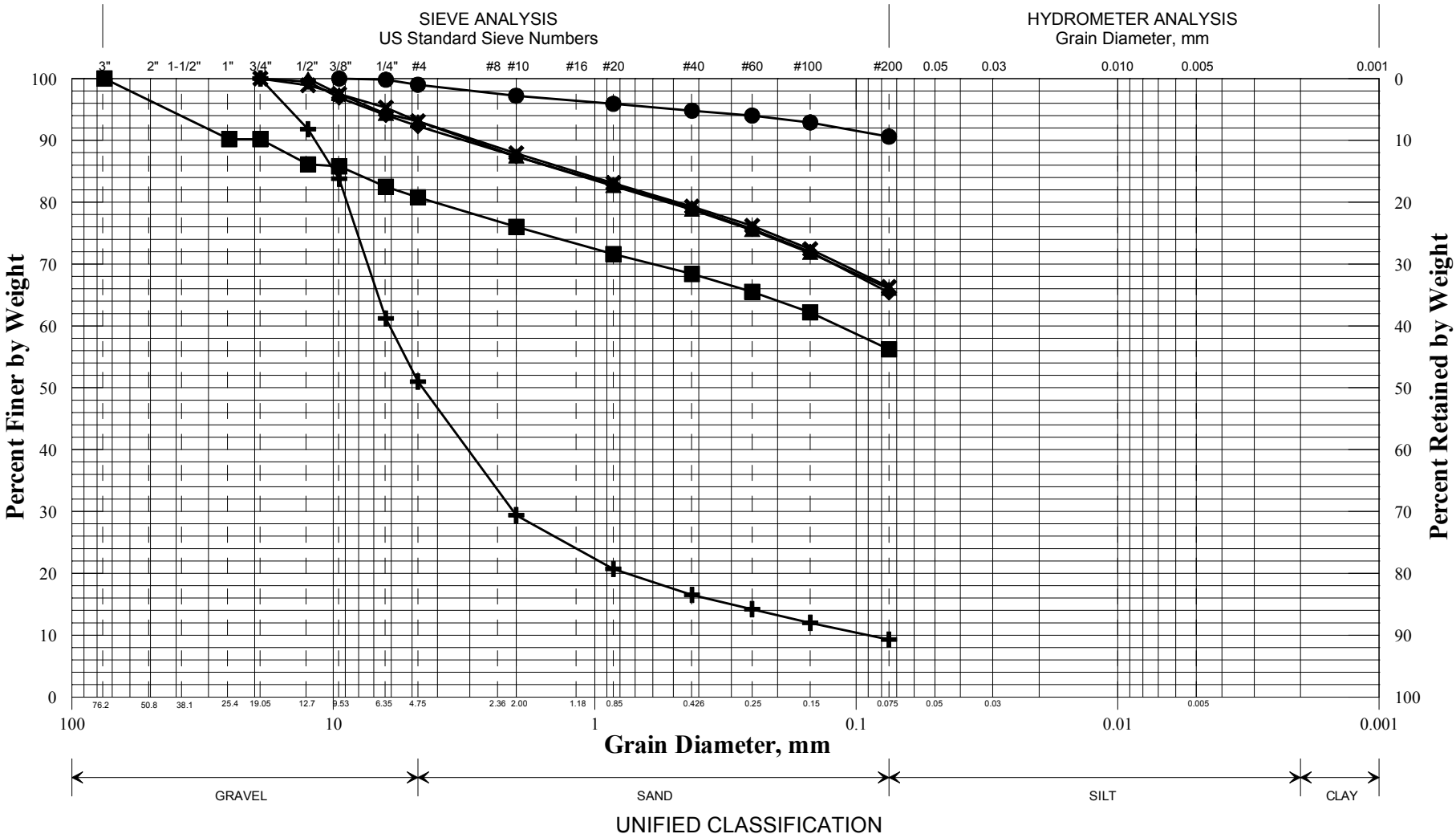
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BUS-105/1D	13+91.3	38.6 RT	2.0-4.0	Sandy GRAVEL, trace silt.	2.4			
◆	BB-BUS-105/2D	13+91.3	38.6 RT	5.0-7.0	SILT, some sand, trace gravel.	11.1			
■	BB-BUS-105/3D	13+91.3	38.6 RT	10.0-12.0	SILT, some sand, little gravel.	11.8			
●	BB-BUS-106/4D	15+06.5	18.9 RT	14.7-16.7	SILT, trace sand, trace gravel.	17.5			
▲	BB-BUS-106/5D	15+06.5	18.9 RT	18.0-20.0	SILT, some sand, trace gravel.	13.5			
✖	BB-BUS-106/7D	15+06.5	18.9 RT	25.0-27.0	SILT, some sand, trace gravel.	12.6			

PIN
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Town
Bangor
Reported by/Date
WHITE, TERRY A 8/31/2009

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
209243	**BB-BUS-103/1D**	**GEOTECHNICAL (DISTURBED)**	**5/19/2009**	**8/18/2009**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **12+92.9** Offset, ft: **21.1 LT** Dbfg, ft: **5.0-7.0**

PIN: **016681.00** Town: **Bangor** Sampler: **WILDER, BRUCE H**

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]	**100.0**
No. 4 [4.75 mm]	**99.9**
No. 10 [2.00 mm]	**99.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.3**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**97.1**
[0.0263 mm]	**92.8**
[0.0175 mm]	**83.8**
[0.0106 mm]	**74.8**
[0.0078 mm]	**65.9**
[0.0056 mm]	**59.9**
[0.0029 mm]	**44.9**
[0.0013 mm]	**29.9**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

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		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
31
Plastic Limit (T 90), %
22
Plasticity Index (T 90), %
9
Specific Gravity, Corrected to 20°C (T 100)
2.67
Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %
20.5

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		

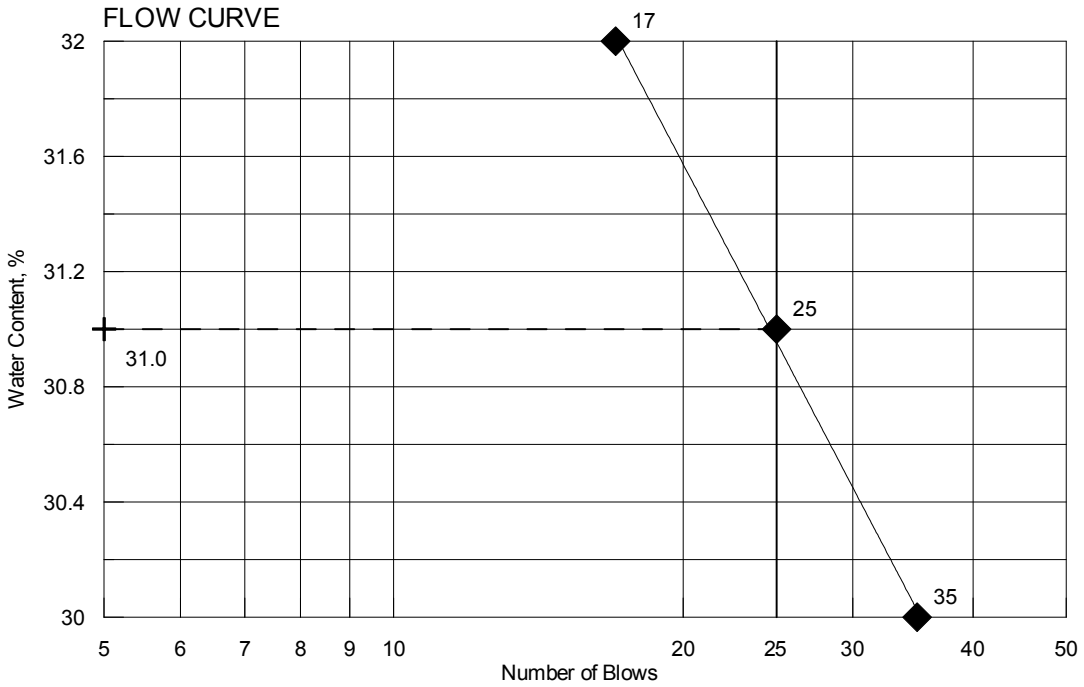
Comments:

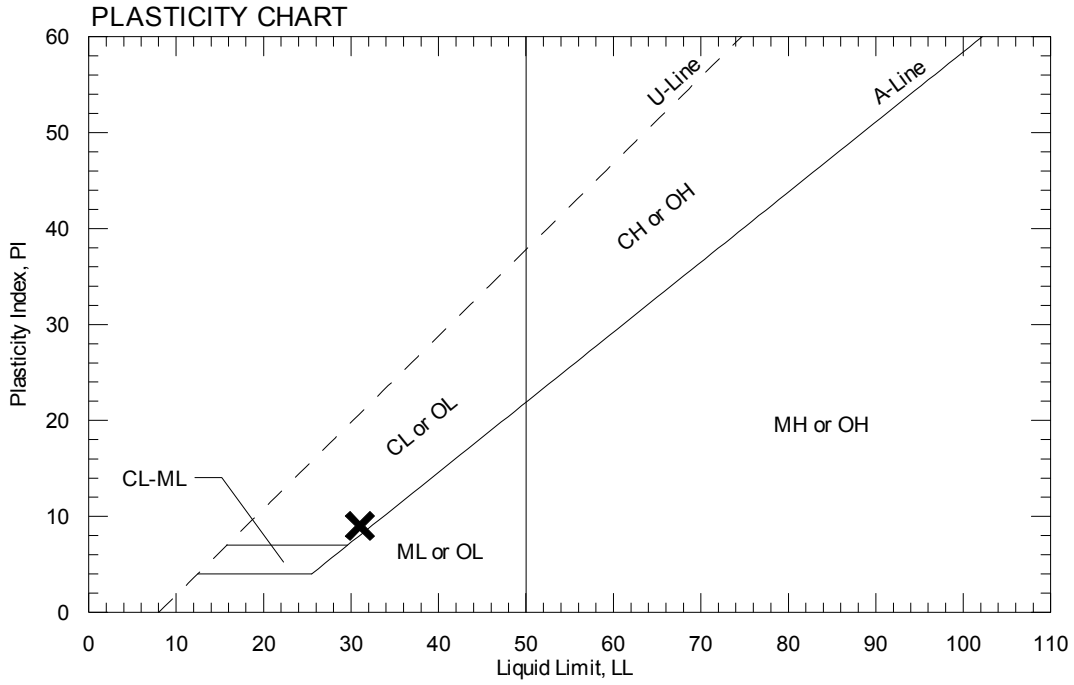
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/25/2009**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Bangor	Reference No.	209243
PIN	016681.00	Water Content, %	20.5
Sampled	5/19/2009	Plastic Limit	22
Boring No./Sample No.	BB-BUS-103/1D	Liquid Limit	31
Station	12+92.9	Plasticity Index	9
Depth	5.0-7.0	Tested By	BBURR





Appendix C

Calculations

ABUTMENT AND WINGWALL ACTIVE AND PASSIVE EARTH PRESSURE:

Rankine Theory - Active Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.5.2, pg. 3-7

Either Rankine or Coulomb may be used for long-heeled cantilever walls where the failure surface is uninterrupted by the top of the wall stem. In general, use Rankine though.

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$$K_a := \tan\left[45deg - \left(\frac{\phi}{2}\right)\right]^2$$

$K_a = 0.31$

Rankine Theory - Passive Earth Pressure from Bowles 5th Edition Section 11-5, pg 602

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

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

$K_{p\_rank} = 3.25$

Coulomb Theory - Active Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.5.2, pg. 3-7

For gravity walls , semi-gravity walls, prefabricated modular walls, and cantilever walls and abutments with short heels where wall and backfill interface friction is considered, use Coulomb Theory

Angle of back face of wall: $\alpha := 90deg$

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$\delta = \beta$ $\delta := \beta$

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

$K_a = 0.31$

Coulomb Theory - Passive Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.6, pg. 3-8

Angle of back face of wall: $\alpha := 90deg$

Soil angle of internal friction: $\phi := 32deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1, pg. 3-74, δ ranges from 17 to 22 $\delta := 20deg$

Angle of backfill from horizontal: $\beta := 0deg$

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

$K_p = 6.89$

FROST PROTECTION

Method 1:

From the Maine Design Freezing Index Map:
DFI = 1730 degree-days
Any Foundations Will Be Backfilled With Coarse-Grained Soils With Wn = 10%

From the 2003 Bridge Design Guide Table 5-1:

$Frost\_depth := [0.3 \cdot (90.1in - 87.5in) + 87.5in]$

$Frost\_depth = 88.28 \cdot in$

$Frost\_depth = 7.36 \cdot ft$

Method 2:

```
------------------------
--- ModBerg Results ---
------------------------

Project Location: Orono, Maine

Air Design Freezing Index        =  1588 F-days
N-Factor                         =  0.80
Surface Design Freezing Index    =  1270 F-days
Mean Annual Temperature          =  43.5 deg F
Design Length of Freezing Season =  132 days

-------------------------------------------------------
Layer
#:Type            t    w%    d    Cf  Cu   Kf   Ku     L
-------------------------------------------------------
1-Coarse        77.4 10.0 125.0  28  34   2.0  1.6  1,800
-------------------------------------------------------

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

*******************************************************
  Total Depth of Frost Penetration = 6.45 ft = 77.4 in.
*******************************************************
```

Use 6.5 feet

BEARING RESISTANCE ON COMPACTED FILL OR TILL SOILS:

SERVICE LIMIT STATE:

LRFD Table C10.6.2.6.1-1, Pg 10-66 (Based on NAVFAC DM 7.2) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State"

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommend Value
Inorganic Silt, Sandy or Clayey Silt (ML, MH)	Very stiff to hard	4 to 8	6 ksf
	Medium stiff to stiff	2 to 6	3 ksf
	Loose	1 to 2	1 ksf

Recommend **Service Limit State** bearing resistance of **4.0 ksf** to control settlements and for preliminary footing sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for spread footings on glacial till at the Strength Limit State:

Assumptions:

1. Footings will be embedded 6.5 feet for frost protection.

   $D_f := 6.5ft$

2. Assumed parameters for soils:
   Assume till

   Moist unit weight: $\gamma_m := 120pcf$

   Saturated unit weight: $\gamma_{sat} := 130pcf$

   Soil angle of internal friction: $\phi_{ns} := 34$

   Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

Unit weight of water: $\gamma_w := 62.4pcf$

Effective Stress at the footing bearing level:

$q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.44 \cdot ksf$

Assume footing width:

$B := \begin{pmatrix} 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 34 deg

Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 42.14$ $N_q := 29.4$ $N_\gamma := 31.1$

Nominal Bearing Resistance per Terzaghi equation

Bowles 5th Ed. Table 4-1 pg. 220

$q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5(\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$

$q_{nom} = \begin{pmatrix} 23.4 \\ 25.5 \\ 27.6 \\ 29.7 \end{pmatrix} \cdot ksf$

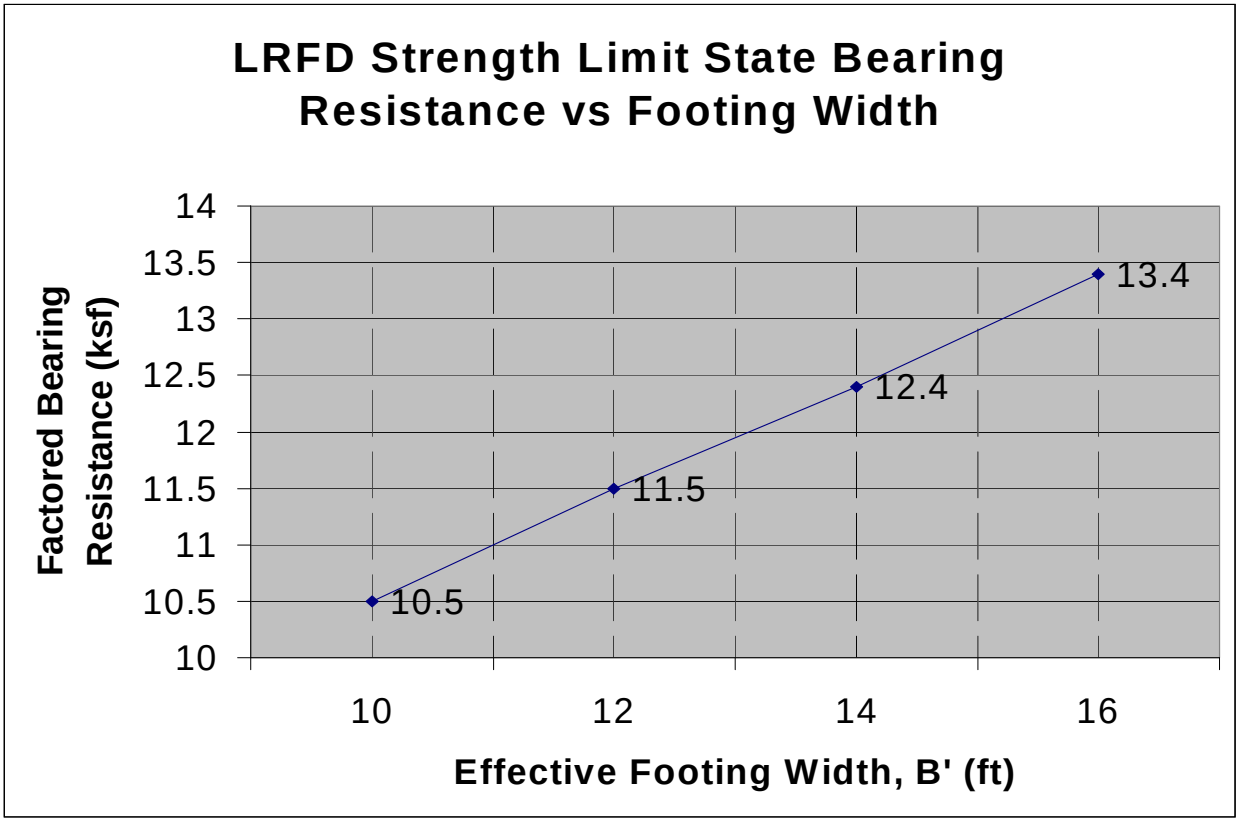
Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32:

$\phi_b := 0.45$

$q_{fac} := q_{nom} \cdot \phi_b$

$q_{fac} = \begin{pmatrix} 10.5 \\ 11.5 \\ 12.4 \\ 13.4 \end{pmatrix} \cdot ksf$

The **Strength Limit State** Factored Bearing Resistances for Abutment and Pier Footings 10-16 feet wide.



EXTREME LIMIT STATE:

Nominal and Factored Bearing Resistance for spread footings on glacial till at the Extreme Limit State considering vehicle collison force CT:

q_{nom} from above:

$$q_{nom} = \begin{pmatrix} 23.4 \\ 25.5 \\ 27.6 \\ 29.7 \end{pmatrix} \cdot ksf \quad \text{for} \quad B := \begin{pmatrix} 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$$

Resistance Factor from LRFD Article 11.5.7 pg. 11-12: $\phi := 1.0$

$$q_{fac} := q_{nom} \cdot \phi$$

$$q_{fac} = \begin{pmatrix} 23.4 \\ 25.5 \\ 27.6 \\ 29.7 \end{pmatrix} \cdot ksf \quad \text{for} \quad B := \begin{pmatrix} 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$$

SEISMIC EVALUATION:

Determine Site Class in Accordance with LRFD Section 3.10
Use N_{avg} Method, Table C3.10.3.1-1
Weakest Column has Most Soil: BB-BUS-103 (except Avg fill thickness)

Fill Layer - 7 ft Avg, N_{60} from other bores
Determine N_{60avg} for Fill:

$$\frac{17+15+43+29+18}{5} = 24.4$$

Silt Layer - 4 ft
Determine N_{60avg}:

11

Upper Till Layer - 21 ft
Determine N_{60avg}:

$$\frac{17+42+18+24}{4} = 25.25$$

Lower Till Layer - 28 ft
Determine N_{60avg}:

$$\frac{38+56+101+50+101}{5} = 69.2$$

Rock Layer - 40 ft
N_{avg} for Rock = 100
Table C3.10.3.1-1

100

Determine N_{bar} for 100 ft Column

$$\frac{100}{\left(\frac{7}{24}\right)+\left(\frac{4}{11}\right)+\left(\frac{21}{25}\right)+\left(\frac{28}{69}\right)+\left(\frac{40}{100}\right)} = 43$$

$15 < N_{bar} < 50$, Site Class D, LRFD Table 3.10.3.1-1, Pg. 3-85

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

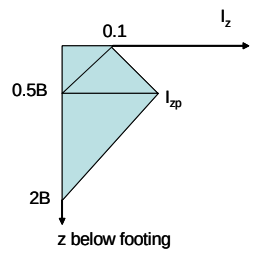
```
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
  State - Maine
  Zip Code - 04401
  Zip Code Latitude     =    44.821200
  Zip Code Longitude  = -068.785700
  Site Class B
  Data are based on a 0.05 deg grid spacing.
    Period        Sa
    (sec)         (g)
     0.0         0.067    PGA - Site Class B
     0.2         0.146    Ss   - Site Class B
     1.0         0.044    S1   - Site Class B
```

```
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
  State - Maine
  Zip Code - 04401
  Zip Code Latitude     =    44.821200
  Zip Code Longitude  = -068.785700
  As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
  Site Class D  -  Fpga =  1.60,  Fa =  1.60,  Fv =  2.40
  Data are based on a 0.05 deg grid spacing.
    Period        Sa
    (sec)         (g)
     0.0         0.108    As   - Site Class D
     0.2         0.233    SDs - Site Class D
     1.0         0.105    SD1 - Site Class D
```

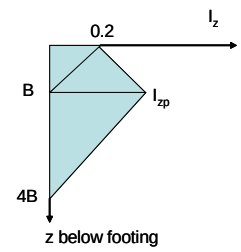
SETTLEMENT ANALYSIS:

Settlement at Abutments and Pier, Granular Method 1:

Schmertmann's Method,
Foundation Engineering Handbook,
2nd Ed. p.179



Square Or Circular Footing



Rectangular Footing, For L/B > 10

$$S_e = C_1 C_2 (q_{contact} - q)\left[\sum (I_z/E_s) \times \Delta z\right]$$

E_s = 320 (N+15) kPa, Say 300ksf

Bowles 5th Edition, p.316

$q_{allow} := 4ksf$ $\quad B = \begin{pmatrix} 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$ $\quad E_s := 300ksf$

$d := 6.5ft$ $\quad \gamma := 130pcf$

$q := d \cdot \gamma$ $\quad q = 845 \cdot psf$

$C_1 := 1 - 0.5 \cdot \left(\frac{q}{q_{allow} - q}\right)$ $\quad C_1 = 0.87$

$C_2 := 1$ For all Maine Soils

Note: $\sigma'_{vp} = \gamma B$ for Rectangular and $\gamma 0.5B$ for Square or Circular Footing
This case:

$\sigma'_{vp} := \gamma \cdot B$ $\quad \sigma'_{vp} = \begin{pmatrix} 1300 \\ 1560 \\ 1820 \\ 2080 \end{pmatrix} \cdot psf$

$\Delta q := q_{allow} - q$ $\quad \Delta q = 3155 \cdot psf$

$I_{zp} := 0.5 + 0.1 \cdot \sqrt{\frac{\Delta q}{\sigma'_{vp}}}$ $\quad I_{zp} = \begin{pmatrix} 0.66 \\ 0.64 \\ 0.63 \\ 0.62 \end{pmatrix}$

Calculate $\sum (I_z/E_s) \times \Delta z$

Assume this project has average uniform E_s, L/B>10

$E_{s1} := E_s$

$E_{s1} = 300 \cdot ksf$ $\quad E_{s2} := E_{s1}$ $\quad E_{s3} := E_{s1}$ $\quad E_{s4} := 0$ $\quad E_{s5} := 0$

$I_{z1} := \frac{0.2 + I_{zp}}{2}$ $\quad I_{z1} = \begin{pmatrix} 0.43 \\ 0.42 \\ 0.42 \\ 0.41 \end{pmatrix}$ $\quad \Delta z1 := 12ft$ $\quad IzEs\Delta z_1 := \overrightarrow{\left(\frac{I_{z1}}{E_{s1}} \cdot \Delta z1\right)}$

$I_{z2} := \frac{I_{zp}}{2}$ $\quad I_{z2} = \begin{pmatrix} 0.33 \\ 0.32 \\ 0.32 \\ 0.31 \end{pmatrix}$ $\quad \Delta z2 := 36ft$ $\quad IzEs\Delta z_2 := \overrightarrow{\left(\frac{I_{z2}}{E_{s2}} \cdot \Delta z2\right)}$

$I_{z3} := 0$ $\quad I_{z3} = 0$ $\quad \Delta z3 := 0ft$ $\quad IzEs\Delta z_3 := \overrightarrow{\left(\frac{I_{z3}}{E_{s3}} \cdot \Delta z3\right)}$

$IzEs\Delta z_1 = \begin{pmatrix} 0.02 \\ 0.02 \\ 0.02 \\ 0.02 \end{pmatrix} \cdot \frac{ft^3}{kip}$ $\quad IzEs\Delta z_2 = \begin{pmatrix} 0.04 \\ 0.04 \\ 0.04 \\ 0.04 \end{pmatrix} \cdot \frac{ft^3}{kip}$ $\quad IzEs\Delta z_3 = 0 \cdot \frac{ft^3}{kip}$

Calculate Settlement

$S_e := \left[C_1 \cdot C_2 \cdot (q_{allow} - q) \cdot (IzEs\Delta z_1 + IzEs\Delta z_2 + IzEs\Delta z_3)\right] \cdot 12 \frac{in}{ft}$

$S_e = \begin{pmatrix} 1.85 \\ 1.82 \\ 1.79 \\ 1.77 \end{pmatrix} \cdot in$

Settlement at Abutments and Pier, Granular Method 2:

Settlement at Abutments

Estimate Settlement for Footing On Soil Using Hough Method:
Ref. LRFD Section 10.6.2.4.2, pg. 10-56

Abutment 1 Assumptions Based on BB-BUS-103, BB-BUS-104:
1) Assume footing will be constructed 20 feet below ground surface (bgs) (~ 7ft below breastwall Finish Grade, Elev. 134 ft)
2) No ground water observed; Assume ground water at footing base elev 134 ft
3) Soil depth below Abut 1 footing Elev 134 ranges from 23 to 38 ft - Use 38 ft (more settlement)
4) Use four 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000
5) Assume Footing Width = 12ft

SPT N_{60} values from BB-BUS-103, and 104:

At 21 feet bgs: 18, 31 => $N_{60Avg} = 25$
At 26 feet bgs: 24, 25 => $N_{60Avg} = 25$
At 31 feet bgs: 38, 35 => $N_{60Avg} = 37$
At 36 feet bgs: 56, 77 => $N_{60Avg} = 67$
At 41 feet bgs: 101, 71 => $N_{60Avg} = 86$
At 46 feet bgs: 50 => $N_{60Avg} = 50$
At 51 feet bgs: 101 => $N_{60Avg} = 101$

$\gamma_t := 130pcf$ $\gamma_w := 62.4pcf$ $(\gamma') := \gamma_t - \gamma_w$ $\gamma' = 67.6 \cdot pcf$

At 21 feet: $\sigma_{o1} := \gamma_t \cdot 20ft + \gamma' \cdot 1ft$ $\sigma_{o1} = 2.67 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 0.91$ $N1_{60Avg} := C_{N1} \cdot 25 = 23$

At 26 feet: $\sigma_{o2} := \gamma_t \cdot 20ft + \gamma' \cdot 6ft$ $\sigma_{o2} = 3.01 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 0.87$ $N1_{60Avg} := C_{N2} \cdot 25 = 22$

For Layer 1: $N1_{60Avg1} := \frac{(23 + 22)}{2} = 23$

At 31 feet: $\sigma_{o3} := \gamma_t \cdot 20ft + \gamma' \cdot 11ft$ $\sigma_{o3} = 3.34 \cdot ksf$ $C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N3} = 0.83$ $N1_{60Avg} := C_{N3} \cdot 37 = 31$

At 36 feet: $\sigma_{o4} := \gamma_t \cdot 20ft + \gamma' \cdot 16ft$ $\sigma_{o4} = 3.68 \cdot ksf$ $C_{N4} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o4}}\right)$

$C_{N4} = 0.8$ $N1_{60Avg} := C_{N4} \cdot 67 = 53$

For Layer 2: $N1_{60Avg2} := \frac{(31 + 53)}{2} = 42$

At 41 feet: $\sigma_{o5} := \gamma_t \cdot 20ft + \gamma' \cdot 21ft$ $\sigma_{o5} = 4.02 \cdot ksf$ $C_{N5} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o5}}\right)$

$C_{N5} = 0.77$ $N1_{60Avg} := C_{N5} \cdot 86 = 66$

At 46 feet: $\sigma_{o6} := \gamma_t \cdot 20ft + \gamma' \cdot 26ft$ $\sigma_{o6} = 4.36 \cdot ksf$ $C_{N6} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o6}}\right)$

$C_{N6} = 0.74$ $N1_{60Avg} := C_{N6} \cdot 50 = 37$

For Layer 3: $N1_{60Avg3} := \frac{(66 + 37)}{2} = 52$

At 51 feet: $\sigma_{o7} := \gamma_t \cdot 20ft + \gamma' \cdot 31ft$ $\sigma_{o7} = 4.7 \cdot ksf$ $C_{N7} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o7}}\right)$

$C_{N7} = 0.72$ $N1_{60Avg} := C_{N7} \cdot 101 = 72$

For Layer 4: $N1_{60Avg4} := 72$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59

Layer 1: $N1_{60Avg1} = 23$ $C'_1 := 82$

Layer 2: $N1_{60Avg2} = 42$ $C'_2 := 138$

Layer 3: $N1_{60Avg3} = 52$ $C'_1 := 177$

Layer 4: $N1_{60Avg4} = 72$ $C'_2 := 280$

$$z := \begin{pmatrix} 5 \\ 15 \\ 25 \\ 35 \end{pmatrix} ft \qquad H := \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} \cdot ft \qquad I := \begin{pmatrix} 0.8 \\ 0.45 \\ 0.3 \\ 0.2 \end{pmatrix} \qquad C' := \begin{pmatrix} 82 \\ 138 \\ 177 \\ 280 \end{pmatrix}$$

$\Delta\sigma_v := 4ksf \cdot I$ $\qquad \Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \\ 1.2 \\ 0.8 \end{pmatrix} \cdot ksf$

For the First Layer:

$\sigma_o := \gamma_t \cdot 20ft + \gamma' \cdot z$

Similarly: $\sigma_o = \begin{pmatrix} 2.94 \\ 3.61 \\ 4.29 \\ 4.97 \end{pmatrix} \cdot ksf$

$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o}\right)\right]}$ $\qquad \Delta H = \begin{pmatrix} 0.46826 \\ 0.15263 \\ 0.07262 \\ 0.0278 \end{pmatrix} \cdot in$

$\Delta H_{TOTAL} := 0.47 \cdot in + 0.15 \cdot in + 0.07in + 0.03in$

$\Delta H_{TOTAL} = 0.72 \cdot in$

<u>Abutment 2 Assumptions Based on BB-BUS-101, BB-BUS-101B, BB-BUS-106:</u>

1) Assume footing will be constructed 20 feet bgs (~ 7ft below breastwall Finish Grade, Elev. 134 ft)
2) Ground water ranges between 7 ft and 13 ft; Use 7 ft bgs (worst case)
3) Soil depth below Abut 1 footing Elev 134 ranges from 10 to 18 ft - Use 18 ft (more settlement)
4) Use two 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000
5) Assume Footing Width = 12ft

SPT N_{60} values from BB-BUS-101, 101B and 106:

At 21 feet bgs: 26, 28, 12 => $N_{60Avg} = 22$
At 26 feet bgs: 26, 16, 23 => $N_{60Avg} = 22$
At 31 feet bgs:31, 62 => $N_{60Avg} = 47$

$\gamma_t := 130pcf$ $\qquad \gamma_w := 62.4pcf$ $\qquad (\gamma') := \gamma_t - \gamma_w$ $\qquad \gamma' = 67.6 \cdot pcf$

At 21 feet: $\sigma_{o1} := \gamma_t \cdot 7ft + \gamma' \cdot 14ft$ $\qquad \sigma_{o1} = 1.86 \cdot ksf$ $\qquad C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 1.03$ $\qquad N1_{60Avg} := C_{N1} \cdot 22 = 23$

At 26 feet: $\sigma_{o2} := \gamma_t \cdot 7ft + \gamma' \cdot 19ft$ $\qquad \sigma_{o2} = 2.19 \cdot ksf$ $\qquad C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 0.97$ $\qquad N1_{60Avg} := C_{N2} \cdot 22 = 21$

For Layer 1: $N1_{60Avg1} := \frac{(23+21)}{2} = 22$

At 31 feet: $\sigma_{o3} := \gamma_t \cdot 7ft + \gamma' \cdot 24ft$ $\sigma_{o3} = 2.53 \cdot ksf$ $C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N3} = 0.92$ $N1_{60Avg} := C_{N3} \cdot 47 = 43$

For Layer 2: $N1_{60Avg2} := 43$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59

Layer 1: $N1_{60Avg1} = 22$ $C'_1 := 82$

Layer 2: $N1_{60Avg2} = 43$ $C'_2 := 142$

$z := \begin{pmatrix} 5 \\ 15 \end{pmatrix} ft$ $H := \begin{pmatrix} 10 \\ 10 \end{pmatrix} \cdot ft$ $I := \begin{pmatrix} 0.8 \\ 0.45 \end{pmatrix}$ $C' := \begin{pmatrix} 82 \\ 142 \end{pmatrix}$

$\Delta\sigma_v := 4ksf \cdot I$ $\Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \end{pmatrix} \cdot ksf$

For the First Layer:

$\sigma_o := \gamma_t \cdot 7ft + \gamma' \cdot 13ft + \gamma' \cdot z$

Similarly:

$\sigma_o = \begin{pmatrix} 2.13 \\ 2.8 \end{pmatrix} \cdot ksf$

$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o}\right)\right]}$ $\Delta H = \begin{pmatrix} 0.58352 \\ 0.18205 \end{pmatrix} \cdot in$

$\Delta H_{TOTAL} := 0.58 \cdot in + 0.18 \cdot in$

$\Delta H_{TOTAL} = 0.76 \cdot in$

Pier Assumptions Based on BB-BUS-102, BB-BUS-105:

1) Assume footing will be constructed ~ 7 feet bgs (~ Finish Grade, Elev. 134 ft)
2) Ground water ranges between 6 ft and 11 ft; Use 6 ft bgs (worst case)
3) Soil depth below Pier footing Elev 129 ranges from 15 to 35 ft - Use 35 ft (more settlement)
4) Use four 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000
5) Assume Footing Width = 12ft

SPT N_{60} values from BB-BUS-102 and 105:

At 6 feet bgs: 29, 32 => $N_{60Avg} = 31$
At 11 feet bgs: 12, 38 => $N_{60Avg} = 25$
At 16 feet bgs: 22, 41 => $N_{60Avg} = 32$
At 21 feet bgs: 40, 63 => $N_{60Avg} = 52$
At 26 feet bgs: 86 => $N_{60Avg} = 86$
At 31 feet bgs: 56 => $N_{60Avg} = 56$
At 36 feet bgs: Missed SPT Use => $N_{60Avg} = 56$

$\gamma_t := 130pcf$ $\gamma_w := 62.4pcf$ $(\gamma') := \gamma_t - \gamma_w$ $\gamma' = 67.6 \cdot pcf$

At 6 feet: $\sigma_{o1} := \gamma_t \cdot 6ft$ $\sigma_{o1} = 0.78 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 1.32$ $N1_{60Avg} := C_{N1} \cdot 31 = 41$

At 11 feet: $\sigma_{o2} := \gamma_t \cdot 6ft + \gamma' \cdot 5ft$ $\sigma_{o2} = 1.12 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 1.2$ $N1_{60Avg} := C_{N2} \cdot 25 = 30$

For Layer 1: $N1_{60Avg1} := \frac{(41 + 30)}{2} = 36$

At 16 feet: $\sigma_{o3} := \gamma_t \cdot 6ft + \gamma' \cdot 10ft$ $\sigma_{o3} = 1.46 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N1} = 1.11$ $N1_{60Avg} := C_{N1} \cdot 32 = 35$

At 21 feet: $\sigma_{o4} := \gamma_t \cdot 6ft + \gamma' \cdot 15ft$ $\sigma_{o4} = 1.79 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o4}}\right)$

$C_{N2} = 1.04$ $N1_{60Avg} := C_{N2} \cdot 52 = 54$

For Layer 2: $N1_{60Avg2} := \frac{(35 + 54)}{2} = 45$

At 26 feet: $\sigma_{o5} := \gamma_t \cdot 6ft + \gamma' \cdot 20ft$ $\sigma_{o5} = 2.13 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o5}}\right)$

$C_{N1} = 0.98$ $\quad N1_{60Avg} := C_{N1} \cdot 86 = 84$

At 31 feet: $\sigma_{o6} := \gamma_t \cdot 6ft + \gamma' \cdot 25ft$ $\quad \sigma_{o6} = 2.47 \cdot ksf$ $\quad C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o6}}\right)$

$C_{N2} = 0.93$ $\quad N1_{60Avg} := C_{N2} \cdot 56 = 52$

For Layer 3: $\quad N1_{60Avg3} := \frac{(84 + 52)}{2} = 68$

At 36 feet: $\sigma_{o7} := \gamma_t \cdot 6ft + \gamma' \cdot 30ft$ $\quad \sigma_{o7} = 2.81 \cdot ksf$ $\quad C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o7}}\right)$

$C_{N3} = 0.89$ $\quad N1_{60Avg} := C_{N3} \cdot 56 = 50$

For Layer 4: $\quad N1_{60Avg4} := 50$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59

Layer 1: $N1_{60Avg1} = 36$ $\quad C'_1 := 120$

Layer 2: $N1_{60Avg2} = 45$ $\quad C'_2 := 147$

Layer 3: $N1_{60Avg3} = 68$ $\quad C'_1 := 252$

Layer 4: $N1_{60Avg4} = 50$ $\quad C'_2 := 168$

$$z := \begin{pmatrix} 5 \\ 15 \\ 25 \\ 35 \end{pmatrix} ft \qquad H := \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} \cdot ft \qquad I := \begin{pmatrix} 0.8 \\ 0.45 \\ 0.3 \\ 0.2 \end{pmatrix} \qquad C' := \begin{pmatrix} 120 \\ 147 \\ 252 \\ 168 \end{pmatrix}$$

$$\Delta\sigma_v := 4ksf \cdot I \qquad \Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \\ 1.2 \\ 0.8 \end{pmatrix} \cdot ksf$$

For the First Layer:

$\sigma_o := \gamma_t \cdot 6.5ft + \gamma' \cdot z$

Similarly:

$$\sigma_o = \begin{pmatrix} 1.18 \\ 1.86 \\ 2.54 \\ 3.21 \end{pmatrix} \cdot ksf$$

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'} \right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o} \right) \right]} \qquad \Delta H = \begin{pmatrix} 0.56879 \\ 0.24007 \\ 0.08015 \\ 0.06901 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.57 \cdot in + 0.24 \cdot in + 0.08in + 0.07in$$

$$\Delta H_{TOTAL} = 0.96 \cdot in$$

OK, Say 1 to 2 inches of immediate elastic settlement below footing on soil.

MaineDOT

Department of Transportation
Bridge Program – Geotechnical Section
16 State House Station
Augusta, Maine 04333-0016

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Soils Report 2011-10 Addendum #1

To:	Bryson Welch, PE, Steve Bodge, PE
Cc:	Laura Krusinski, PE, TEDOCS
Author:	Mike Moreau, PE
Doc Type:	24
Date:	May 22, 2013
Bridge #:	5797
Route:	222
PIN:	16681.00
Town:	Bangor
Subject:	Soils Report No. 2011-10 Revised Service Limit State Bearing Resistance

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The following revisions are made to the Geotechnical Design Report for the Replacement of Union Street Bridge over I-95, Bangor, Maine, Soils Report No. 2011-10.

Background

Becker Structural Engineers, Inc., Portland, Maine, have been contracted by the Department to provide final design services for the Union Street Bridge. During the final design process, Becker used the presumptive factored service limit bearing resistance of 4 ksf presented in our preliminary Soils Report No. 2011-10, dated May 12, 2011 to size foundation spread footings. Becker determined that a footing size of 16.5 feet was required based on that bearing resistance. Becker asked whether a higher factored service limit bearing resistance might be used to reduce the required footing width. Although a footing width of 16.5 feet is acceptable, a footing this size will be more difficult to construct in the restricted I-95 median area.

Revised Bearing Resistance Analysis

Allowable settlement controls spread footing foundation design at this site. We reviewed and refined our settlement analysis to determine what bearing resistances resulted in acceptable levels of settlement. Based on our refined analysis, we estimate that the total settlement of a prepared subgrade consisting of compacted fill or native glacial till at the elevations proposed in the 90% plan submittal will be on the order of 1 to 1-1/2 inches for conventional spreads footings with factored service limit bearing resistances up to 6.0 ksf. This settlement is acceptable and will occur during construction. Revised supporting calculations are attached.

Recommendations

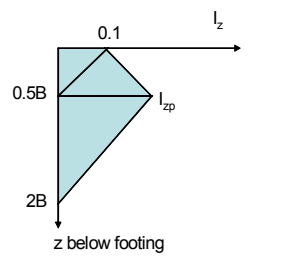
Based on our refined analysis, we recommend that a factored bearing resistance of 6.0 ksf be used to analyze the service limit state and for footing sizing to control settlement.

Attach: Revised Calculations

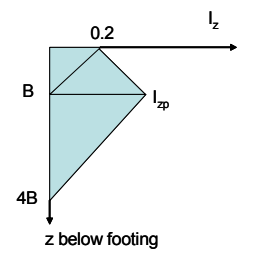
SETTLEMENT ANALYSIS:

Settlement at Abutments and Pier, Granular Method 1:

Schmertmann's Method,
Foundation Engineering Handbook,
2nd Ed. p.179



Square or Circular Footing, For L/B = 1



Rectangular Footing, For L/B > 10

$$S_e = C_1 C_2 (q_{contact} - q)\left[\sum (I_z/E_s) \times \Delta z\right]$$

$q_{allow} := 6.0ksf$ $\qquad$ $B := \begin{pmatrix} 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$

$d := 6.5ft$ $\qquad$ $\gamma := 130pcf$

$q := d \cdot \gamma$ $\qquad$ $q = 845 \cdot psf$

$C_1 := 1 - 0.5 \cdot \left(\frac{q}{q_{allow} - q}\right)$ $\qquad$ $C_1 = 0.92$

$C_2 := 1$ $\qquad$ For all Maine Soils

Note: $\sigma'_{vp} = \gamma B + q$ for Rectangular and $\gamma 0.5B + q$ for Square or Circular Footing
This case, discount sloping fill in front of abutment (predicts most settlement)

$\sigma'_{vp} := \gamma \cdot B$ $\qquad$ $\sigma'_{vp} = \begin{pmatrix} 1300 \\ 1560 \\ 1820 \\ 2080 \end{pmatrix} \cdot psf$

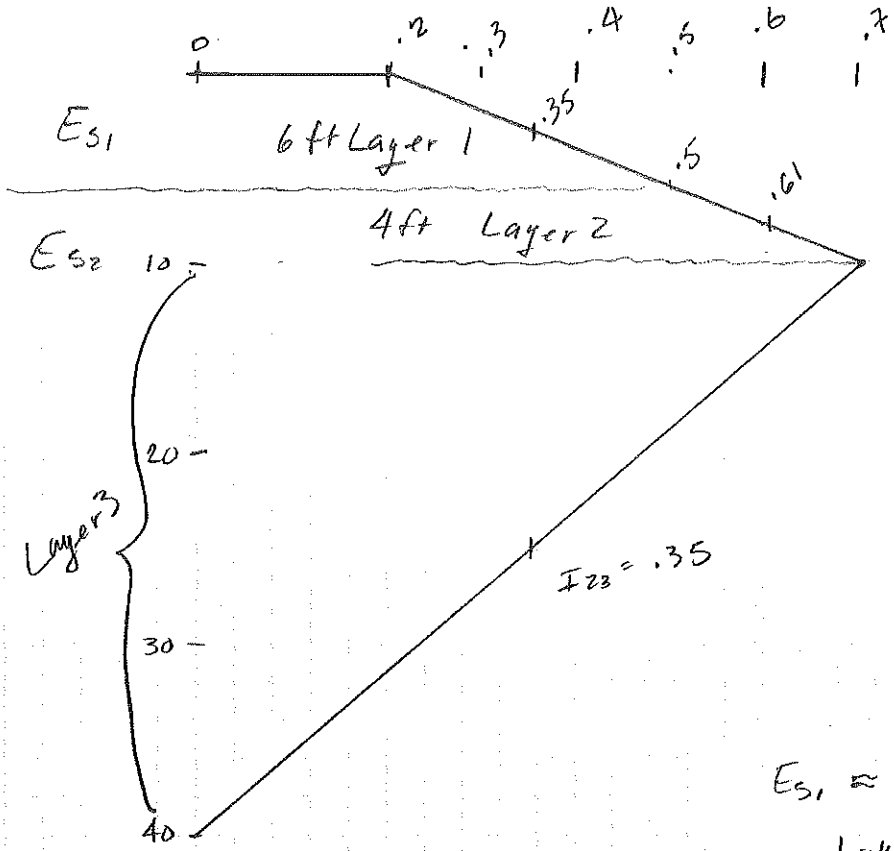
$\Delta q := q_{allow} - q$ $\qquad$ $\Delta q = 5155 \cdot psf$

$I_{zp} := 0.5 + 0.1 \cdot \sqrt{\frac{\Delta q}{\sigma'_{vp}}}$ $\qquad$ $I_{zp} = \begin{pmatrix} 0.7 \\ 0.68 \\ 0.67 \\ 0.66 \end{pmatrix}$

PRELIM. BY M. Morean DATE 3/1/13 PROJ. NO. 16681 FILE NO. ____ OF ____
FINAL CHK. BY ____ DATE ____ LOCATION BANGOR SH. NO. ____ OF ____
ITEM NO. ____ SUBJECT Union St. Bridge

Assume B = 10
4B = 40

For Rect. Ftg I_{zp} @ B = 10



$E_{s1} \approx 500$ ksf

Looked @ Several Eqn ~ Bowles p 316

$E_{s2} \approx 40000 + 1050(22)$ Bowles
N = 22 is most conservative value in all profiles

$E_{s2} \approx 1260$ ksf

Calculate $\sum (I_z/E_s) \times \Delta z$

Consevative Profile on This project has Two E_s Layers:

E_s = 2750 (N) kPa, N = 12 =>Say 500ksf Bowles p.316 (Note Several Eqns Evaluated)

So 6 Feet of E_s = 500 ksf Under Footing

E_s = 40000 + 1050 (N) kPa, N = 22 => Say 1260ksf Bowles p.316

So 34 Feet of E_s = 1260 ksf Below Weaker Till Above

(Note Some Locations Have Thinner Till Above Bedrock, So This Is Conservative)

Use Schemertmann Curve For Rectangular Footing L/B>10
Use I_{zp} = 0.7 (Predicts Most Settlement)

$E_{s1} := 500ksf$ $E_{s2} := 1260ksf$ $E_{s3} := 1260ksf$ $E_{s4} := 0$ $E_{s5} := 0$

$I_{z1} := \frac{0.2 + 0.50}{2}$ $I_{z1} = 0.35$ $\Delta z1 := 6ft$ $IzEs\Delta z_1 := \overrightarrow{\left(\frac{I_{z1}}{E_{s1}} \cdot \Delta z1\right)}$

$I_{z2} := 0.61$ $I_{z2} = 0.61$ $\Delta z2 := 4ft$ $IzEs\Delta z_2 := \overrightarrow{\left(\frac{I_{z2}}{E_{s2}} \cdot \Delta z2\right)}$

$I_{z3} := 0.35$ $I_{z3} = 0.35$ $\Delta z3 := 30ft$ $IzEs\Delta z_3 := \overrightarrow{\left(\frac{I_{z3}}{E_{s3}} \cdot \Delta z3\right)}$

$IzEs\Delta z_1 = 0.004 \cdot \frac{ft^3}{kip}$ $IzEs\Delta z_2 = 0.002 \cdot \frac{ft^3}{kip}$ $IzEs\Delta z_3 = 0.01 \cdot \frac{ft^3}{kip}$

Calculate Settlement

$$S_e := \left[C_1 \cdot C_2 \cdot (q_{allow} - q) \cdot (IzEs\Delta z_1 + IzEs\Delta z_2 + IzEs\Delta z_3)\right] \cdot 12 \frac{in}{ft}$$

$S_e = 0.82 \cdot in$ Say 1 inch

Settlement at Abutments and Pier, Granular Method 2:

Settlement at Abutments

Estimate Settlement for Footing On Soil Using Hough Method:
Ref. LRFD Section 10.6.2.4.2, pg. 10-56

Abutment 1 Assumptions Based on BB-BUS-103, BB-BUS-104:
1) Assume footing will be constructed 20 feet below ground surface (bgs) (~ 7ft below breastwall Finish Grade, Elev. 131 ft)
2) No ground water observed; Assume ground water at footing base elev 131 ft
3) Soil depth below Abut 1 footing Elev 131 ranges from 23 to 38 ft - Use 38 ft (more settlement)
4) Use four 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000

SPT N_{60} values from BB-BUS-103, and 104:

At 21 feet bgs: 18, 31 => $N_{60Avg} = 25$
At 26 feet bgs: 24, 25 => $N_{60Avg} = 25$
At 31 feet bgs: 38, 35 => $N_{60Avg} = 37$
At 36 feet bgs: 56, 77 => $N_{60Avg} = 67$
At 41 feet bgs: 101, 71 => $N_{60Avg} = 86$
At 46 feet bgs: 50 => $N_{60Avg} = 50$
At 51 feet bgs: 101 => $N_{60Avg} = 101$

$\gamma_t := 130pcf$ $\gamma_w := 62.4pcf$ $(\gamma') := \gamma_t - \gamma_w$ $\gamma' = 67.6 \cdot pcf$

At 21 feet: $\sigma_{o1} := \gamma_t \cdot 20ft + \gamma' \cdot 1ft$ $\sigma_{o1} = 2.67 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 0.91$ $N1_{60Avg} := C_{N1} \cdot 25 = 23$

At 26 feet: $\sigma_{o2} := \gamma_t \cdot 20ft + \gamma' \cdot 6ft$ $\sigma_{o2} = 3.01 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 0.87$ $N1_{60Avg} := C_{N2} \cdot 25 = 22$

For Layer 1: $N1_{60Avg1} := \frac{(23 + 22)}{2} = 23$

At 31 feet: $\sigma_{o3} := \gamma_t \cdot 20ft + \gamma' \cdot 11ft$ $\sigma_{o3} = 3.34 \cdot ksf$ $C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N3} = 0.83$ $N1_{60Avg} := C_{N3} \cdot 37 = 31$

At 36 feet: $\sigma_{o4} := \gamma_t \cdot 20ft + \gamma' \cdot 16ft$ $\quad \sigma_{o4} = 3.68 \cdot ksf$ $\quad C_{N4} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o4}}\right)$

$C_{N4} = 0.8$ $\quad N1_{60Avg} := C_{N4} \cdot 67 = 53$

For Layer 2: $N1_{60Avg2} := \frac{(31 + 53)}{2} = 42$

At 41 feet: $\sigma_{o5} := \gamma_t \cdot 20ft + \gamma' \cdot 21ft$ $\quad \sigma_{o5} = 4.02 \cdot ksf$ $\quad C_{N5} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o5}}\right)$

$C_{N5} = 0.77$ $\quad N1_{60Avg} := C_{N5} \cdot 86 = 66$

At 46 feet: $\sigma_{o6} := \gamma_t \cdot 20ft + \gamma' \cdot 26ft$ $\quad \sigma_{o6} = 4.36 \cdot ksf$ $\quad C_{N6} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o6}}\right)$

$C_{N6} = 0.74$ $\quad N1_{60Avg} := C_{N6} \cdot 50 = 37$

For Layer 3: $N1_{60Avg3} := \frac{(66 + 37)}{2} = 52$

At 51 feet: $\sigma_{o7} := \gamma_t \cdot 20ft + \gamma' \cdot 31ft$ $\quad \sigma_{o7} = 4.7 \cdot ksf$ $\quad C_{N7} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o7}}\right)$

$C_{N7} = 0.72$ $\quad N1_{60Avg} := C_{N7} \cdot 101 = 72$

For Layer 4: $N1_{60Avg4} := 72$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59
Assume B = 12 to reference Boussinesq Stress Contours for I values below per LRFD Fig. 10.6.2.4.1-1

Layer 1: $N1_{60Avg1} = 23$ $\quad C'_1 := 82$

Layer 2: $N1_{60Avg2} = 42$ $\quad C'_2 := 138$

Layer 3: $N1_{60Avg3} = 52$ $\quad C'_1 := 177$

Layer 4: $N1_{60Avg4} = 72$ $\quad C'_2 := 280$

$$z := \begin{pmatrix} 5 \\ 15 \\ 25 \\ 35 \end{pmatrix} ft \qquad H := \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} \cdot ft \qquad I := \begin{pmatrix} 0.8 \\ 0.45 \\ 0.3 \\ 0.2 \end{pmatrix} \qquad C' := \begin{pmatrix} 82 \\ 138 \\ 177 \\ 280 \end{pmatrix}$$

$\Delta\sigma_v := 4ksf \cdot I$ $\qquad \Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \\ 1.2 \\ 0.8 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v4.5} := 4.5ksf \cdot I$ $\qquad \Delta\sigma_{v4.5} = \begin{pmatrix} 3.6 \\ 2.0 \\ 1.4 \\ 0.9 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.0} := 5ksf \cdot I$ $\qquad \Delta\sigma_{v5.0} = \begin{pmatrix} 4.0 \\ 2.3 \\ 1.5 \\ 1.0 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.5} := 5.5ksf \cdot I$ $\qquad \Delta\sigma_{v5.5} = \begin{pmatrix} 4.4 \\ 2.5 \\ 1.7 \\ 1.1 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v6.0} := 6ksf \cdot I$ $\qquad \Delta\sigma_{v6.0} = \begin{pmatrix} 4.8 \\ 2.7 \\ 1.8 \\ 1.2 \end{pmatrix} \cdot ksf$

For the First Layer:

$\sigma_o := \gamma_t \cdot 20ft + \gamma' \cdot z$

Similarly: $\qquad \sigma_o = \begin{pmatrix} 2.94 \\ 3.61 \\ 4.29 \\ 4.97 \end{pmatrix} \cdot ksf$

For $\Delta\sigma_v$ = 4.0ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_v}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.46826\\0.15263\\0.07262\\0.0278\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.47\cdot in + 0.15\cdot in + 0.07in + 0.03in$$

$$\Delta H_{TOTAL} = 0.72\cdot in$$

For $\Delta\sigma_v$ = 4.5ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v4.5}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.50838\\0.16801\\0.08056\\0.031\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.51\cdot in + 0.17\cdot in + 0.08in + 0.03in$$

$$\Delta H_{TOTAL} = 0.79\cdot in$$

For $\Delta\sigma_v$ = 5.0ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v5.0}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.54612\\0.18279\\0.08829\\0.03415\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.55\cdot in + 0.18\cdot in + 0.09in + 0.03in$$

$$\Delta H_{TOTAL} = 0.85\cdot in$$

For $\Delta\sigma_v$ = 5.5ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v5.5}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.58175\\0.19701\\0.09582\\0.03724\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.58\cdot in + 0.20\cdot in + 0.10in + 0.04in$$

$$\Delta H_{TOTAL} = 0.92\cdot in$$

For $\Delta\sigma_v$ = 6.0ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v6.0}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.61548\\0.21071\\0.10316\\0.04028\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.62\cdot in + 0.21\cdot in + 0.10in + 0.04in$$

$$\Delta H_{TOTAL} = 0.97\cdot in$$

Summary Settlements At Abutment 1 Varying $\Delta\sigma_v$ For B ~ 12 ft

$\Delta\sigma_v$ (ksf)	Δ (in)
4.0	0.72
4.5	0.79
5.0	0.85
5.5	0.92
6.0	0.97

Abutment 2 Assumptions Based on BB-BUS-101, BB-BUS-101B, BB-BUS-106:

1) Assume footing will be constructed 20 feet bgs (~ 7ft below breastwall Finish Grade, Elev. 135 ft)
2) Ground water ranges between 7 ft and 13 ft; Use 7 ft bgs (worst case)
3) Soil depth below Abut 1 footing Elev 135 ranges from 10 to 18 ft - Use 18 ft (more settlement)
4) Use two 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000

SPT N_{60} values from BB-BUS-101, 101B and 106:

At 21 feet bgs: 26, 28, 12 => N_{60Avg} = 22
At 26 feet bgs: 26, 16, 23 => N_{60Avg} = 22
At 31 feet bgs:62 => N_{60Avg} = 62

$\gamma_t := 130pcf$ $\quad \gamma_w := 62.4pcf$ $\quad (\gamma') := \gamma_t - \gamma_w$ $\quad \gamma' = 67.6\cdot pcf$

At 21 feet: $\sigma_{o1} := \gamma_t\cdot 7ft + \gamma'\cdot 14ft$ $\quad \sigma_{o1} = 1.86\cdot ksf$ $\quad C_{N1} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 1.03$ $\quad N1_{60Avg} := C_{N1}\cdot 22 = 23$

At 26 feet: $\sigma_{o2} := \gamma_t\cdot 7ft + \gamma'\cdot 19ft$ $\quad \sigma_{o2} = 2.19\cdot ksf$ $\quad C_{N2} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 0.97$ $\quad N1_{60Avg} := C_{N2}\cdot 22 = 21$

For Layer 1: $\quad N1_{60Avg1} := \frac{(23+21)}{2} = 22$

At 31 feet: $\sigma_{o3} := \gamma_t \cdot 7ft + \gamma' \cdot 24ft$ $\sigma_{o3} = 2.53 \cdot ksf$ $C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N3} = 0.92$ $N1_{60Avg} := C_{N3} \cdot 62 = 57$

For Layer 2: $N1_{60Avg2} := 57$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59
Assume B = 12 to reference Boussinesq Stress Contours for I values below per LRFD Fig. 10.6.2.4.1-1

Layer 1: $N1_{60Avg1} = 22$ $C'_1 := 82$

Layer 2: $N1_{60Avg2} = 57$ $C'_2 := 197$

$z := \begin{pmatrix} 5 \\ 15 \end{pmatrix} ft$ $H := \begin{pmatrix} 10 \\ 10 \end{pmatrix} \cdot ft$ $I := \begin{pmatrix} 0.8 \\ 0.45 \end{pmatrix}$ $C' := \begin{pmatrix} 82 \\ 197 \end{pmatrix}$

$\Delta\sigma_v := 4ksf \cdot I$ $\Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v4.5} := 4.5ksf \cdot I$ $\Delta\sigma_{v4.5} = \begin{pmatrix} 3.6 \\ 2.0 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.0} := 5ksf \cdot I$ $\Delta\sigma_{v5.0} = \begin{pmatrix} 4.0 \\ 2.3 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.5} := 5.5ksf \cdot I$ $\Delta\sigma_{v5.5} = \begin{pmatrix} 4.4 \\ 2.5 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v6.0} := 6ksf \cdot I$ $\Delta\sigma_{v6.0} = \begin{pmatrix} 4.8 \\ 2.7 \end{pmatrix} \cdot ksf$

For the First Layer:

$$\sigma_o := \gamma_t \cdot 7ft + \gamma' \cdot 13ft + \gamma' \cdot z$$

Similarly:

$$\sigma_o = \begin{pmatrix} 2.13 \\ 2.8 \end{pmatrix} \cdot ksf$$

For $\Delta\sigma_v$ = 4.0ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'} \right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o} \right) \right]} \qquad \Delta H = \begin{pmatrix} 0.58352 \\ 0.13123 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.58 \cdot in + 0.13 \cdot in$$

$$\Delta H_{TOTAL} = 0.71 \cdot in$$

For $\Delta\sigma_v$ = 4.5ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'} \right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_{v4.5}}{\sigma_o} \right) \right]} \qquad \Delta H = \begin{pmatrix} 0.62954 \\ 0.14385 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.63 \cdot in + 0.14 \cdot in$$

$$\Delta H_{TOTAL} = 0.77 \cdot in$$

For $\Delta\sigma_v$ = 5.0ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'} \right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_{v5.0}}{\sigma_o} \right) \right]} \qquad \Delta H = \begin{pmatrix} 0.67245 \\ 0.1559 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.67 \cdot in + 0.16 \cdot in$$

$$\Delta H_{TOTAL} = 0.83 \cdot in$$

For $\Delta\sigma_v$ = 5.5ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v5.5}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.71264\\0.16743\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.71\cdot in + 0.17\cdot in$$

$$\Delta H_{TOTAL} = 0.88\cdot in$$

For $\Delta\sigma_v$ = 6.0ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o+\Delta\sigma_{v6.0}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix}0.75045\\0.17847\end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.75\cdot in + 0.18\cdot in$$

$$\Delta H_{TOTAL} = 0.93\cdot in$$

Summary Settlements At Abutment 2 Varying $\Delta\sigma_v$ For B ~ 12 ft

$\Delta\sigma_v$ (ksf)	Δ (in)
4.0	0.71
4.5	0.77
5.0	0.83
5.5	0.88
6.0	0.93

Pier Assumptions Based on BB-BUS-102, BB-BUS-105:

1) Assume footing will be constructed ~ 7 feet bgs (~ Finish Grade, Elev. 128.5 ft)
2) Ground water ranges between 6 ft and 11 ft; Use 6 ft bgs (worst case)
3) Soil depth below Pier footing Elev 128.5 ranges from 15 to 35 ft - Use 35 ft (more settlement)
4) Use four 10 ft layers for settlement analysis per FHWA NHI-00-045, August 2000

SPT N_{60} values from BB-BUS-102 and 105:

At 6 feet bgs: 29, 32 => $N_{60Avg} = 31$
At 11 feet bgs: 12, 38 => $N_{60Avg} = 25$
At 16 feet bgs: 22, 41 => $N_{60Avg} = 32$
At 21 feet bgs: 40, 63 => $N_{60Avg} = 52$
At 26 feet bgs: 86 => $N_{60Avg} = 86$
At 31 feet bgs: 56 => $N_{60Avg} = 56$
At 36 feet bgs: Missed SPT Use => $N_{60Avg} = 56$

$\gamma_t := 130pcf$ $\gamma_w := 62.4pcf$ $(\gamma') := \gamma_t - \gamma_w$ $\gamma' = 67.6 \cdot pcf$

At 6 feet: $\sigma_{o1} := \gamma_t \cdot 6ft$ $\sigma_{o1} = 0.78 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o1}}\right)$

$C_{N1} = 1.32$ $N1_{60Avg} := C_{N1} \cdot 31 = 41$

At 11 feet: $\sigma_{o2} := \gamma_t \cdot 6ft + \gamma' \cdot 5ft$ $\sigma_{o2} = 1.12 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o2}}\right)$

$C_{N2} = 1.2$ $N1_{60Avg} := C_{N2} \cdot 25 = 30$

For Layer 1: $N1_{60Avg1} := \frac{(41 + 30)}{2} = 36$

At 16 feet: $\sigma_{o3} := \gamma_t \cdot 6ft + \gamma' \cdot 10ft$ $\sigma_{o3} = 1.46 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o3}}\right)$

$C_{N1} = 1.11$ $N1_{60Avg} := C_{N1} \cdot 32 = 35$

At 21 feet: $\sigma_{o4} := \gamma_t \cdot 6ft + \gamma' \cdot 15ft$ $\sigma_{o4} = 1.79 \cdot ksf$ $C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o4}}\right)$

$C_{N2} = 1.04$ $N1_{60Avg} := C_{N2} \cdot 52 = 54$

For Layer 2: $N1_{60Avg2} := \frac{(35 + 54)}{2} = 45$

At 26 feet: $\sigma_{o5} := \gamma_t \cdot 6ft + \gamma' \cdot 20ft$ $\sigma_{o5} = 2.13 \cdot ksf$ $C_{N1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o5}}\right)$

$C_{N1} = 0.98$ $\quad N1_{60Avg} := C_{N1} \cdot 86 = 84$

At 31 feet: $\sigma_{o6} := \gamma_t \cdot 6ft + \gamma' \cdot 25ft$ $\quad \sigma_{o6} = 2.47 \cdot ksf$ $\quad C_{N2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o6}}\right)$

$C_{N2} = 0.93$ $\quad N1_{60Avg} := C_{N2} \cdot 56 = 52$

For Layer 3: $\quad N1_{60Avg3} := \frac{(84 + 52)}{2} = 68$

At 36 feet: $\sigma_{o7} := \gamma_t \cdot 6ft + \gamma' \cdot 30ft$ $\quad \sigma_{o7} = 2.81 \cdot ksf$ $\quad C_{N3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{o7}}\right)$

$C_{N3} = 0.89$ $\quad N1_{60Avg} := C_{N3} \cdot 56 = 50$

For Layer 4: $\quad N1_{60Avg4} := 50$

Calculate Settlement

Use **Well Graded Silty Sand and Gravel** curve, Figure 10.6.2.4.2-1, pg 10-59
Assume B = 12 to reference Boussinesq Stress Contours for I values below per LRFD Fig. 10.6.2.4.1-1

Layer 1: $N1_{60Avg1} = 36$ $\quad C'_1 := 120$

Layer 2: $N1_{60Avg2} = 45$ $\quad C'_2 := 147$

Layer 3: $N1_{60Avg3} = 68$ $\quad C'_1 := 252$

Layer 4: $N1_{60Avg4} = 50$ $\quad C'_2 := 168$

$$z := \begin{pmatrix} 5 \\ 15 \\ 25 \\ 35 \end{pmatrix} ft \qquad H := \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} \cdot ft \qquad I := \begin{pmatrix} 0.8 \\ 0.45 \\ 0.3 \\ 0.2 \end{pmatrix} \qquad C' := \begin{pmatrix} 120 \\ 147 \\ 252 \\ 168 \end{pmatrix}$$

$$\Delta\sigma_v := 4ksf \cdot I \qquad \Delta\sigma_v = \begin{pmatrix} 3.2 \\ 1.8 \\ 1.2 \\ 0.8 \end{pmatrix} \cdot ksf$$

$\Delta\sigma_{v4.5} := 4.5ksf \cdot I$ $\quad \Delta\sigma_{v4.5} = \begin{pmatrix} 3.6 \\ 2.0 \\ 1.4 \\ 0.9 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.0} := 5ksf \cdot I$ $\quad \Delta\sigma_{v5.0} = \begin{pmatrix} 4.0 \\ 2.3 \\ 1.5 \\ 1.0 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v5.5} := 5.5ksf \cdot I$ $\quad \Delta\sigma_{v5.5} = \begin{pmatrix} 4.4 \\ 2.5 \\ 1.7 \\ 1.1 \end{pmatrix} \cdot ksf$

$\Delta\sigma_{v6.0} := 6ksf \cdot I$ $\quad \Delta\sigma_{v6.0} = \begin{pmatrix} 4.8 \\ 2.7 \\ 1.8 \\ 1.2 \end{pmatrix} \cdot ksf$

For the First Layer:

$\sigma_o := \gamma_t \cdot 6.5ft + \gamma' \cdot z$

Similarly:

$$\sigma_o = \begin{pmatrix} 1.18 \\ 1.86 \\ 2.54 \\ 3.21 \end{pmatrix} \cdot ksf$$

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'} \right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o} \right) \right]} \qquad \Delta H = \begin{pmatrix} 0.56879 \\ 0.24007 \\ 0.08015 \\ 0.06901 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.57 \cdot in + 0.24 \cdot in + 0.08in + 0.07in$$

$$\Delta H_{TOTAL} = 0.96 \cdot in$$

For $\Delta\sigma_v$ = 4.5ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_{v4.5}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix} 0.60672 \\ 0.26122 \\ 0.08829 \\ 0.07665 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.61 \cdot in + 0.26 \cdot in + 0.09in + 0.08in$$

$$\Delta H_{TOTAL} = 1.04 \cdot in$$

For $\Delta\sigma_v$ = 5.0ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_{v5.0}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix} 0.6416 \\ 0.28119 \\ 0.09613 \\ 0.0841 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.64 \cdot in + 0.28 \cdot in + 0.10in + 0.08in$$

$$\Delta H_{TOTAL} = 1.1 \cdot in$$

For $\Delta\sigma_v$ = 5.5ksf

$$\Delta H := \overrightarrow{\left[H \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_{v5.5}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix} 0.67388 \\ 0.30009 \\ 0.10367 \\ 0.09138 \end{pmatrix} \cdot in$$

$$\Delta H_{TOTAL} := 0.67 \cdot in + 0.30 \cdot in + 0.10in + 0.09in$$

$$\Delta H_{TOTAL} = 1.16 \cdot in$$

For $\Delta\sigma_v$ = 6.0ksf

$$\Delta H := \overrightarrow{\left[H\cdot\left(\frac{1}{C'}\right)\cdot\log\left(\frac{\sigma_o + \Delta\sigma_{v6.0}}{\sigma_o}\right)\right]} \qquad \Delta H = \begin{pmatrix} 0.70393 \\ 0.31803 \\ 0.11096 \\ 0.0985 \end{pmatrix}\cdot in$$

$$\Delta H_{TOTAL} := 0.70\cdot in + 0.32\cdot in + 0.11in + 0.10in$$

$$\Delta H_{TOTAL} = 1.23\cdot in$$

Summary Settlements At Pier Varying $\Delta\sigma_v$ For B ~ 12 ft

$\Delta\sigma_v$ (ksf)	Δ (in)
4.0	0.96
4.5	1.04
5.0	1.10
5.5	1.16
6.0	1.23

OK, Say 1 to 1.5 inches of immediate elastic settlement below Abutment or Pier footings on soil.