MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

SEBAGO LAKE ROAD CROSSING BRIDGE
OVER MAINE CENTRAL RAILROAD
STATE ROUTE 35
STANDISH, MAINE

STATE OF MAINE
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7120
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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of Sebago Lake Road Crossing Bridge over Maine Central Railroad in Standish, Maine. The proposed replacement bridge will consist of an approximately 75 foot long, single span, welded steel plate girder bridge on semi-integral concrete abutments on spread footings constructed behind U-shaped Mechanically Stabilized Earth (MSE) wall wrapped fills with crashworthy barriers. The following design recommendations are discussed in detail in the attached report:

Abutment Subgrade Preparation - Abutment spread footings shall be constructed on a bed of compacted ¾ inch crushed stone 3.0 feet thick, placed in 8-inch maximum lifts compacted with at least four (4) passes of a heavy, walk behind vibratory-type compactor.

Semi-integral Stub Abutment Bearing Resistance – It is anticipated that the semi-integral stub abutments at the site will be founded on a bed of crushed stone on select granular fill soils associated with the MSE walls. Applicable permanent and transient loads are specified in AASHTO LFRD Bridge Design Specifications Fourth Edition (LRFD) Article 11.5.5. The design of abutments on MSE walls shall be in accordance with LRFD Article 11.10.11. Abutment footings shall be proportioned to provide stability against bearing capacity failure. Bearing resistance for any structure founded on granular soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 ksf. A factored bearing resistance of 4 ksf based on FHWA Allowable Stress Design recommendations may be used when analyzing the service limit state and for preliminary footing sizing. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure.

Semi-integral Stub Abutments - The bottom of footing elevation for Abutment No. 1 is anticipated to be approximately 287.75 feet. The bottom of footing elevation for Abutment No. 2 is anticipated to be approximately 287.0 feet. Per LRFD Article 11.10.11 the minimum distance from the centerline of the bearing on the abutment to the outer edge of the MSE wall facing shall be 3.5 feet. The minimum distance between the back face of the panel and the footing shall be 6 inches. The footings on MSE walls shall be designed for all applicable load combinations specified in AASHTO LRFD Bridge Design Specifications Fourth Edition (LRFD) Articles 3.4.1 and 11.5.5 and 11.6.2 through 11.6.6. The design of abutments founded on spread footings at the strength limit state shall consider factored bearing resistance, eccentricity, lateral sliding and structural failure. At the service limit state spread footing design shall be assessed for: settlement, horizontal movement, and overall stability. The overall stability of the foundation should be investigated at the Service I Load Combination. Semi-integral abutments should be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the height of soil behind the superstructure/end diaphragm. The superstructure backwall (end diaphragm) should be designed for full passive pressure. In designing for active and passive earth pressures, a Rankine active earth pressure coefficient, K_a, and a Coulomb passive earth pressure coefficient, K_p, are recommended.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

All portions of the proposed MSE walls supporting bridge substructures are within a distance of 50 feet to the centerline of the railroad track. Per LRFD Article 3.6.5.2, the abutment MSE walls should be designed for railway vehicle impact forces or protected by a crashworthy barrier as described in LRFD Article 3.6.5.1.

Mechanically Stabilized Earth Wall Wrapped Abutments –MSE walls constructed on existing granular soils will be used to support the semi-integral stub abutments. The walls shall be designed in accordance with Special Provision 636 and LRFD Article 11.10 by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for any structure founded on native soils at this site should be evaluated at the strength limit state using factored loads and a bearing resistance of 14.0 ksf. A factored bearing resistance of 6.0 ksf may be used when analyzing the service limit state and for preliminary footing sizing. A concrete leveling pad with a width no less than 2.0 feet shall be provided to support the MSE wall face elements. The front face of the wall shall be founded a minimum of 5.0 feet below finished exterior grade for frost protection. The minimum length of reinforcement for MSE walls supporting bridge abutments shall be the greater of 22 feet or 0.6(H+d)+6.5 feet, where H is the wall height as measured from the leveling pad and d is the height of soil above the wall. The reinforcing length shall be uniform throughout the entire height of the wall. An impervious Geomembrane consisting of low-permeability, 2 sided, textured HDPE wit hah minimum thickness of 60 mils shall be installed near the top of the reinforced soil zone to reduce the chance of water infiltration into the reinforced backfill.

The internal and external stability of the MSE walls shall be designed for all additional vertical and horizontal loads and forces imposed by the abutment footing and the bridge superstructure, in addition to the supplemental lateral earth pressures on the abutment and superstructure end diaphragm. It is important that these additional vertical and horizontal forces and loads be included on the Plans for use by the MSE wall designer-supplier.

Settlement – Fills of up to 24 feet will be required to construct the MSE wall wrapped stub abutments. Settlements during construction of the fills are anticipated to range from 2 to 4 inches. Post construction settlements are anticipated to be less than 1.0 inch. Due to the granular nature of the fill soils, the majority of the settlements are anticipated to occur during construction having negligible effect on the finished bridge structure.

Frost Protection - Any foundations placed on native subgrade soil should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies to foundations placed on soil, including MSE wall leveling pads. An alternative foundation construction approach allows founding the abutment footings on a 3 foot bed of crushed stone with an impermeable membrane over the MSE wall reinforced soil and an abutment embedment depth of 3 feet for frost protection.

Seismic Design Considerations – The Sebago Lake Road Crossing Bridge on State Route 35 is not on the National Highway System (NHS). The site is assigned to Site Class D and Seismic Zone 1. The LRFD code states that single span bridges need not be analyzed for seismic loads regardless of their seismic zone. The minimum requirements as specified in LRFD Articles 4.7.4.2 and 3.10.9.2 apply.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of Sebago Lake Road Crossing Bridge over the Maine Central Railroad in Standish, Cumberland County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge was constructed in 1951 and consists of a 179 foot long, three-span, steel superstructure supported on spill through concrete abutments and two concrete column piers. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in “poor” condition (rating of 4), the bridge superstructure is in “fair” condition (rating of 5) and the substructure is in “satisfactory” condition (rating of 6). Year 2008 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 48.9. It is understood that the existing bridge superstructure and substructures will be completely removed and replaced.

The proposed bridge has been designed by Maguire Group, Inc. of Portsmouth, New Hampshire and will consist of a 75 foot long, single span, welded steel plate girder superstructure with a bituminous wearing surface. The bridge will be supported on semi-integral concrete abutments constructed on Mechanically Stabilized Earth (MSE) wall wrapped fills. The proposed MSE walls are located within 50 feet of the centerline of the railroad tracks therefore; crashworthy barriers are required at the base of the MSE walls per LRFD Article 3.6.5. The alignment centerline will be located approximately 40 feet north of the existing bridge centerline. The bridge will have a 30 degree skew with the railroad. Two-way traffic will be maintained during construction on the existing structure while the proposed bridge is constructed on the new alignment.

2.0 GEOLOGIC SETTING

Sebago Lake Road Crossing Bridge on State Route 35 in Standish crosses the Maine Central Railroad approximately 0.6 miles north of State Route 114 as shown on Sheet 1 - Location Map found at the end of this report. The railroad is a service rail owned by MaineDOT and is currently out of service.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits (coarse grained facies). Soils in the site area are generally comprised of sand, gravel and minor amounts of silt. The unit generally is deposited in areas where the topography is flat to moderately sloping. These soils were generally deposited where glacial meltwater streams and currents entered the sea.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as interbedded pelite and sandstone of the Waterville Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Boring BB-SSLX-101 was drilled at track level near the location of existing Pier No. 1 (west). Boring BB-SSLX-102 was drilled at track level near location of existing Pier No. 2 (east). These borings were drilled between February 2 and 5, 2009 by the MaineDOT Materials, Testing and Exploration Department and Northern Test Boring (NTB) of Gorham, Maine prior to the designer's decision to build the bridge on a new alignment. An additional boring, BB-SSLX-201, was drilled on June 29 and 30, 2009 at the track level north of the new alignment in order to obtain information along the new alignment for design and construction. The exploration locations are shown on Sheet 2 – Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheets 4 and 5 - Boring Logs found at the end of this report.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. Both of drill rigs used at the site are equipped with automatic hammers to drive the split spoon. The hammers were calibrated in August of 2007 and again in February 2009. The MaineDOT automatic hammer was found to deliver approximately 30 percent more energy in 2007 and 40 percent more energy in 2009 during driving than the standard rope and cathead system. The NTB automatic hammer was found to deliver approximately 13 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor to the raw field N-values. These hammer efficiency factors (0.77 and 0.84 for MaineDOT and 0.633 for NTB) and both the raw field N-value and the corrected N-value are shown on the boring logs.

The MaineDOT geotechnical team member and/or a Certified Subsurface Inspector selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field testing requirements and logged the subsurface conditions encountered. The borings were located in the field by survey during drilling activities.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of 20 standard grain size analyses and two (2) grain size analyses with hydrometer. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 4 and 5 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered in the borings consisted of sand and gravel with occasional cobbles. Due to the depth of the overburden soils in the area of the bridge and the planned use of MSE wall supported abutments on spread footings, the borings were terminated before reaching bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered at the site:

Brown Sand - A layer of brown sand was encountered as the upper layer in all of the borings. This layer was found to be dark brown, brown and light brown, damp to wet, fine to coarse sand, trace to some gravel, trace silt and occasional cobbles. The thickness of this layer was approximately 73.0 feet in boring BB-SSLX-101 and approximately 58.0 feet in boring BB-SSLX-102. The layer was not fully penetrated in Boring BB-SSLX-201. Corrected SPT N-values in the layer ranged from 8 to 61 blows per foot (bpf) indicating that the soil is loose to very dense in consistency. Water contents from twelve (12) samples obtained within the layer range from approximately 3% to 20%. Twelve (12) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SP-SM or SP by the Unified Soil Classification System.

Rust Colored Sand - A layer of rust colored sand was encountered beneath the brown sand in borings BB-SSLX-101 and BB-SSLX-102. This layer was found to be rust colored, wet, fine to coarse sand, trace to little gravel, trace silt, and trace clay. The thickness of this layer was approximately 6.0 feet in boring BB-SSLX-101 and approximately 10.0 feet in boring BB-SSLX-102. Corrected SPT N-values in the layer ranged from 19 to 53 bpf indicating that the soil is medium dense to very dense in consistency. Water contents from two (2) samples obtained within the layer range from approximately 17% to 20%. Two (12) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SP-SM or SC-SM by the Unified Soil Classification System.

Grey Sand - A layer of grey sand was encountered beneath the rust colored sand in borings BB-SSLX-101 and BB-SSLX-102. This layer was found to be grey, wet, fine to coarse and fine to medium sand, trace gravel, trace to some silt, and trace clay. This layer was not fully penetrated in the borings. Corrected SPT N-values in the layer ranged from 22 to 50 bpf indicating that the soil is medium dense to dense in consistency. Water contents from eight (8) samples obtained within the layer range from approximately 19% to 26%. Eight (8) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-3 or A-2-4 by the AASHTO Classification System and a SP-SM, SM or SC-SM by the Unified Soil Classification System.

Groundwater - Groundwater was observed at a depths ranging from approximately 9.0 feet to 17.0 feet below the existing ground surface. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions.

Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

MaineDOT has contracted Maguire Group, Inc. of Portsmouth, New Hampshire to design the replacement structure for the Sebago Lake Road Crossing Bridge. During the Preliminary Design Report (PDR) development phase of the project, Maguire Group, Inc. evaluated the following foundation alternatives for this project:

- Pile supported integral abutments
- Stub abutments with spread footings on MSE wrapped fills with crashworthy barriers
- Semi-integral abutments with spread footings on MSE wrapped fills with crashworthy barriers

The use of semi-integral abutments with spread footings on MSE wrapped fills was chosen as the most viable foundation for the site. An LRFD criterion requires crashworthy barriers at the base of the MSE walls. This report addresses only this foundation type.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for semi-integral stub abutments founded MSE wall wrapped fills protected by with crashworthy barriers which have been identified as the optimal foundation types for the project.

7.1 Abutment Subgrade Preparation

Abutment spread footings shall be constructed on a bed of compacted ¾-inch crushed stone 3.0 feet thick, placed in 8-inch maximum lifts compacted with at least four (4) passes of a heavy, walk behind vibratory-type compactor (method should approximate compaction to approximately 97% of AASHTO T-180). See Appendix D - Special Provision at the end of this report for specific gradation requirements.

7.2 Semi-integral Stub Abutment Bearing Resistance

It is anticipated that the semi-integral stub abutments at the site will be founded on a crushed stone mat and select granular fill associated with the MSE wall Special Provision. Applicable permanent and transient loads are specified in AASHTO LFRD Bridge Design Specifications Fourth Edition (LRFD) Article 11.5.5. The design of abutments on MSE walls shall be in accordance with LRFD Articles 11.10.11. Abutment footings shall be proportioned to provide stability against bearing capacity failure.

As the semi-integral stub abutments are to be supported on granular soils the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in LRFD Figure 11.6.3.2-1. Bearing resistance for abutment footings bearing on MSE walls shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 ksf. A factored bearing resistance of 4 ksf based on FHWA

Allowable Stress Design recommendations may be used when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as 0.3*f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure.

7.3 Semi-integral Stub Abutments

The cast-in-place, semi-integral stub abutments will be placed on spread footings on MSE walls. The bottom of footing elevation for Abutment No. 1 is anticipated to be approximately 287.75 feet. The bottom of footing elevation for Abutment No. 2 is anticipated to be approximately 287.0 feet. Per LRFD Article 11.10.11 the minimum distance from the centerline of the bearing on the abutment to the outer edge of the MSE wall facing shall be 3.5 feet. The minimum distance between the back face of the panel and the footing shall be 6 inches.

The footings on MSE walls shall be designed for all applicable load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6.2 through 11.6.6. The design of abutments founded on spread footings at the strength limit state shall consider factored bearing resistance, eccentricity, lateral sliding and structural failure.

Per LRFD Table 10.5.5.2.2-1, a sliding resistance factor, ϕ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place concrete footings on sand. Sliding computations for resistances to lateral loads shall assume a maximum frictional coefficient of 0.45 at the footing-soil interface.

For spread footings on soil, the eccentricity of loading at the strength limit state shall not exceed one-fourth ($1/4^{th}$) of the effective footing dimensions.

The resistance factor of 1.0 shall be used to assess spread footing design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Semi-integral abutments should be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the height of soil behind the superstructure/end diaphragm. The superstructure backwall (end diaphragm) should be designed for full passive pressure. In designing for active and passive earth pressures, a Rankine active earth pressure coefficient, K_a, and a Coulomb passive earth pressure coefficient, K_p, are recommended. However, the Designer may elect a more conservative approach and design the abutment stem wall to withstand a passive earth pressure state. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.307 is recommended. In designing for passive earth pressure, the Coulomb state is recommended. Designing semi-integral abutments for Coulomb passive earth pressure, K_p=6.89, may result in uneconomical wall sections. For this reason, consideration may be given to using a

Rankine passive earth pressure, K_p=3.25 when designing semi-integral abutments. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the return wings when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 4 below:

Abutment Height	h_{eq}
5 feet	4.0 feet
10 feet	3.0 feet
≥20 feet	2.0 feet

Table 1 – Equivalent Height of Soil for Vehicular Loading on Abutments

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

All portions of the proposed MSE walls supporting bridge substructures are within a distance of 50 feet to the centerline of the railroad track. Per LRFD Article 3.6.5.2, the abutment MSE walls should be designed for railway vehicle impact forces or protected by a crashworthy barrier as described in LRFD Article 3.6.5.1.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the MaineDOT BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

7.4 Mechanically Stabilized Earth Wall Wrapped Abutments

Mechanically Stabilized Earth (MSE) walls founded on existing granular soils will be used to support the semi-integral stub abutments. The walls shall be designed in accordance with Special Provision 636 and LRFD Article 11.10 by a Professional Engineer subcontracted by the Contractor as a design-build item. Special Provision 636 is included in Appendix D found at the end of this report. No utilities other than highway drainage are to be constructed within the reinforced zone unless access is provided to utilities without disrupting reinforcements and breakage or rupture of utility lines will not have a detrimental effect on the stability of the structure.

The MSE walls shall be designed in accordance with Special Provision 636 and LRFD Article 11.10. The MSE walls shall be designed by the vendor for external and internal stability of the reinforced mass behind the facing. It is the responsibility of MaineDOT to assure the MSE wall and approach embankment adequately meet requirements for global stability and bearing capacity. Special Provision 636 also includes requirements for facing elements, reinforcing strips, backfill material and compaction, impervious membrane, drainage and other wall requirements.

The MSE wall elements should consider the permanent and transient loads as specified in LRFD Articles 3.4.1 and 11.10.5.2. Earth pressures for external stability shall be calculated using an active earth pressure coefficient, K_a=0.31, calculated using Rankine Theory. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 2 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥20	2.0	2.0

Table 2 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Traffic loads shall be treated as uniform surcharge loads in accordance with LRFD Article 3.11.6.2. The live load surcharge pressure shall not be less than 2.0 feet of earth. Parapets or traffic barriers constructed over or in line with the front face of the wall shall be designed to resist overturning moments by their own mass. The upper layers of soil reinforcement shall have sufficient tensile capacity to resist a concentrated horizontal load of γP_H where P_H is 10 kips distributed over the barrier length of 5.0 feet. Parapets and traffic barriers shall satisfy crash testing requirements as specified in LRFD Section 13.

The internal and external stability of the MSE walls shall be designed for all additional vertical and horizontal loads and forces imposed by the abutment footing and the bridge superstructure, in addition to the supplemental lateral earth pressures on the abutment and superstructure end diaphragm. It is important that these additional vertical and horizontal forces and loads be included on the Plans for use by the MSE wall designer-supplier.

MSE walls may be used to retain soil supporting bridge abutments supported on spread footings with the following additional design criteria:

- A minimum distance of 3.5 feet should be provided between the outer edge of the MSE wall facing and the centerline of bearing on the abutment.

- The minimum distance between the back face of the panel and the abutment footing shall be 6.0 inches.

- A minimum distance of 4 feet should be provided between the bottom of the superstructure and the berm in front of the footing behind the MSE top panel. The berm should be surfaced with an impermeable treatment.

- The top of the MSE panel in front of the abutment footings should be set 1 foot above the berm elevation.

The bearing resistance for the reinforced soil volume and leveling pads founded on native soils at this site should be evaluated at the strength limit state using factored loads and a bearing resistance of 14.0 ksf. The bearing resistance factor, ϕ_b, for spread footings on soil is 0.45. A factored bearing resistance of 6.0 ksf may be used when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure. A concrete leveling pad with a width no less than 2.0 feet should be provided to support the wall face elements. Any organic material encountered shall be removed to the full depth and replaced with compacted gravel borrow meeting MaineDOT 703.20.

A resistance factor of ϕ= 1.0 shall be used to assess the MSE volume design at the service limit state including: settlement, horizontal movement and overall stability. The Extreme Event II limit state design check related to collision by rail vehicles for MSE walls supporting bridge abutments footings shall include bearing resistance, eccentricity, sliding and overall stability. A resistance factor of ϕ=1.0 shall be used for the Extreme Event II limit state. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, ϕ, of 0.65.

The MSE wall designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for MSE wall volume backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.58 at the soil-soil interface. A sliding resistance factor of ϕ_τ=0.9 shall be applied to the nominal sliding resistance of MSE walls founded on soil.

The minimum length of reinforcement for MSE walls supporting bridge abutments shall be the greater of 22 feet or 0.6(H+d)+6.5 feet, where H is the wall height as measured from the leveling pad and d is the height of soil above the wall. The reinforcing length shall be uniform throughout the entire height of the wall unless substantiating evidence is presented to indicate that variation in length is satisfactory. Backfill within the reinforced mass shall consist of Gravel Borrow meeting the requirements of MaineDOT 703.20 except the maximum particle size shall be limited to 4 inches. The backfill within the reinforced mass shall meet the additional electrochemical requirements specified in Special Provision 636.

An impervious Geomembrane consisting of low-permeability, 2 sided, textured HDPE wit hah minimum thickness of 60 mils shall be installed near the top of the reinforced soil zone to reduce the chance of water infiltration into the reinforced backfill. The membrane shall be bonded to the back of the abutment. The surface of the membrane and the approach slab shall be sloped to shed water infiltrating from the road surface above.

7.5 Settlement

In order to construct the MSE wall wrapped abutments significant fills are required at each abutment location. Evaluation of potential settlement at each of the abutments resulted in approximately 2 to 4 inches of settlement. Due to the granular nature of the site soils, settlements are anticipated to occur during construction having negligible effect on the finished bridge structure. Post construction settlements are anticipated to be less than 1.0 inch.

7.6 Frost Protection

It is anticipated that the semi-integral stub abutments will be founded MSE wall wrapped fills. All foundations placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the US Army Cold Regions Research and Engineering Laboratory software ModBerg, the site has a design-freezing index of approximately 1200 F-degree days. This correlates to a frost depth of 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. See Appendix C - Calculations at the end of this report for supporting documentation.

This minimum embedment depth applies to foundations placed on soil, including MSE wall leveling pads. An alternative foundation construction approach allows founding the abutment footings on a 3 foot bed of crushed stone with an impermeable membrane over the MSE wall reinforced soil and an abutment embedment depth of 3 feet for frost protection.

7.7 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the Maine DOT BDG, Sebago Lake Road Crossing Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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) = 0.095g
- Short-term (0.2-second period) spectral acceleration coefficient (S_{DS}) = 0.298g
- Long-term (1.0-second period) spectral acceleration coefficient (S_{D1}) = 0.112g
- Site Class D (site soils with average N-value greater than 15 bpf and less than 50 bpf)
- Seismic Zone 1 (based on S_{D1} less than or equal to 0.15g)

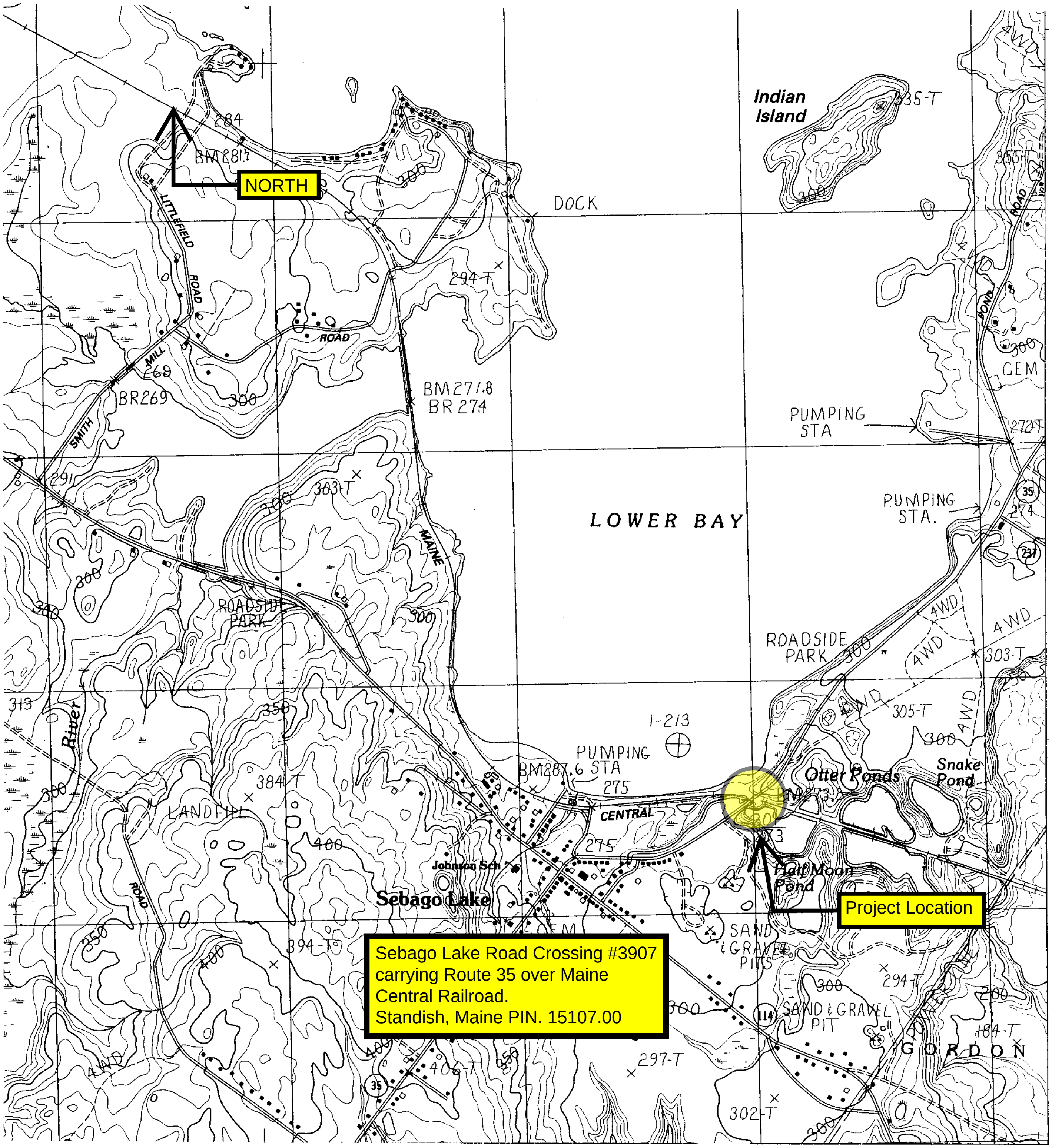
See Appendix C- Calculations at the end of this report for supporting documentation.

8.0 CLOSURE

This report has been prepared for the use of the Maguire Group, Inc. and the MaineDOT Bridge Program for specific application to the proposed replacement of Sebago Lake Road Crossing Bridge in Standish, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is 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 at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets



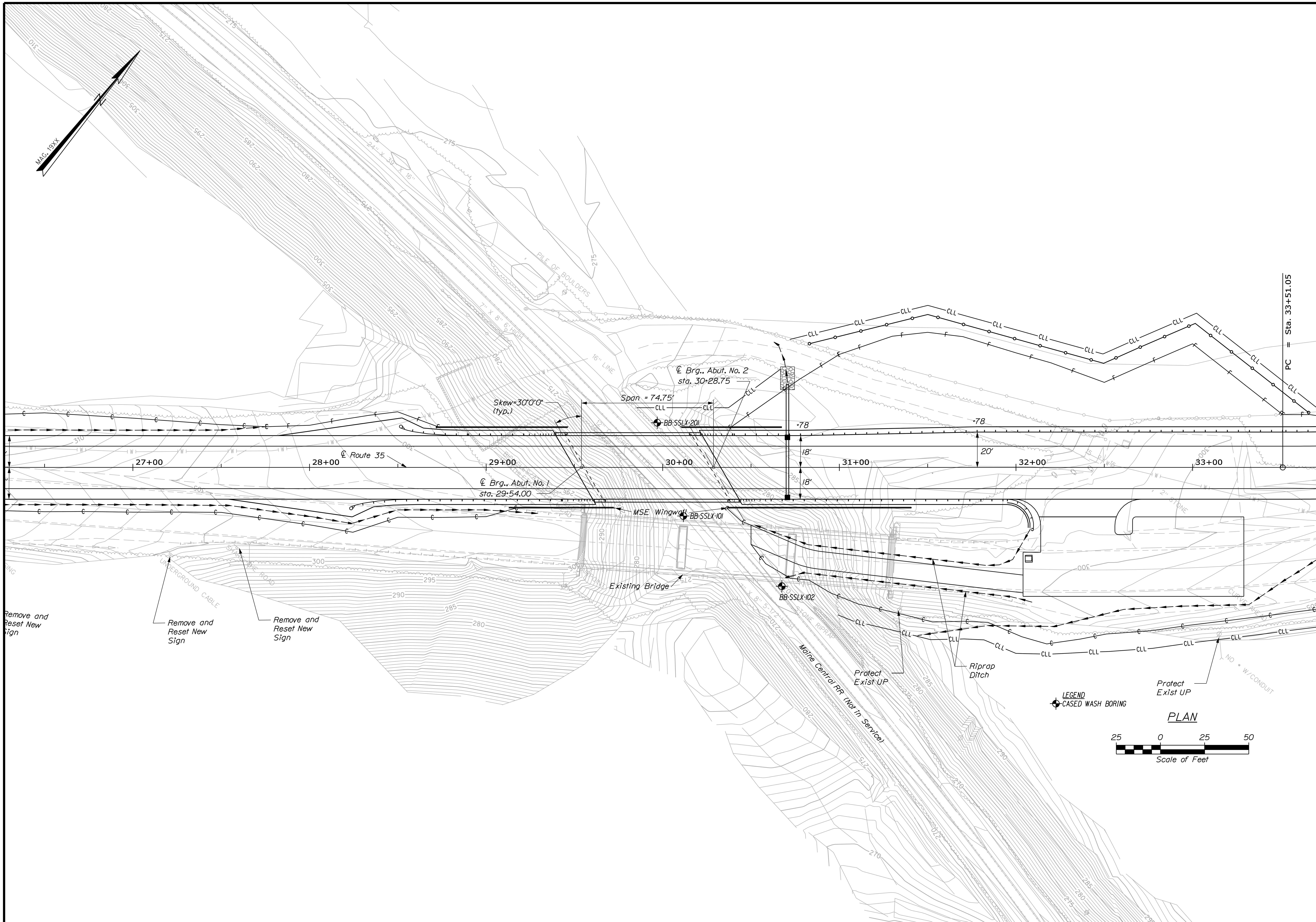
SEBAGO LAKE QUADRANGLE
MAINE – CUMBERLAND CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000

KILOMETERS 1 .5 0 1 2
METERS 1000 0 1000 2000
MILES 1 .5 0 1
FEET 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10 000

CONTOUR INTERVAL 10 FEET

Filename: ...\00\GEOTECH\MSTA\002_BLP1.dgn Division: GEOTECH Username: terry.white Date:7/23/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1510(700)X

PIN 015107.00

BRIDGE NO. 3907

BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

ROUTE 35, SEBAGO LAKE ROAD CROSSING
MAINE CENTRAL RAILROAD
STANDISH ANDROSCOGGIN COUNTY

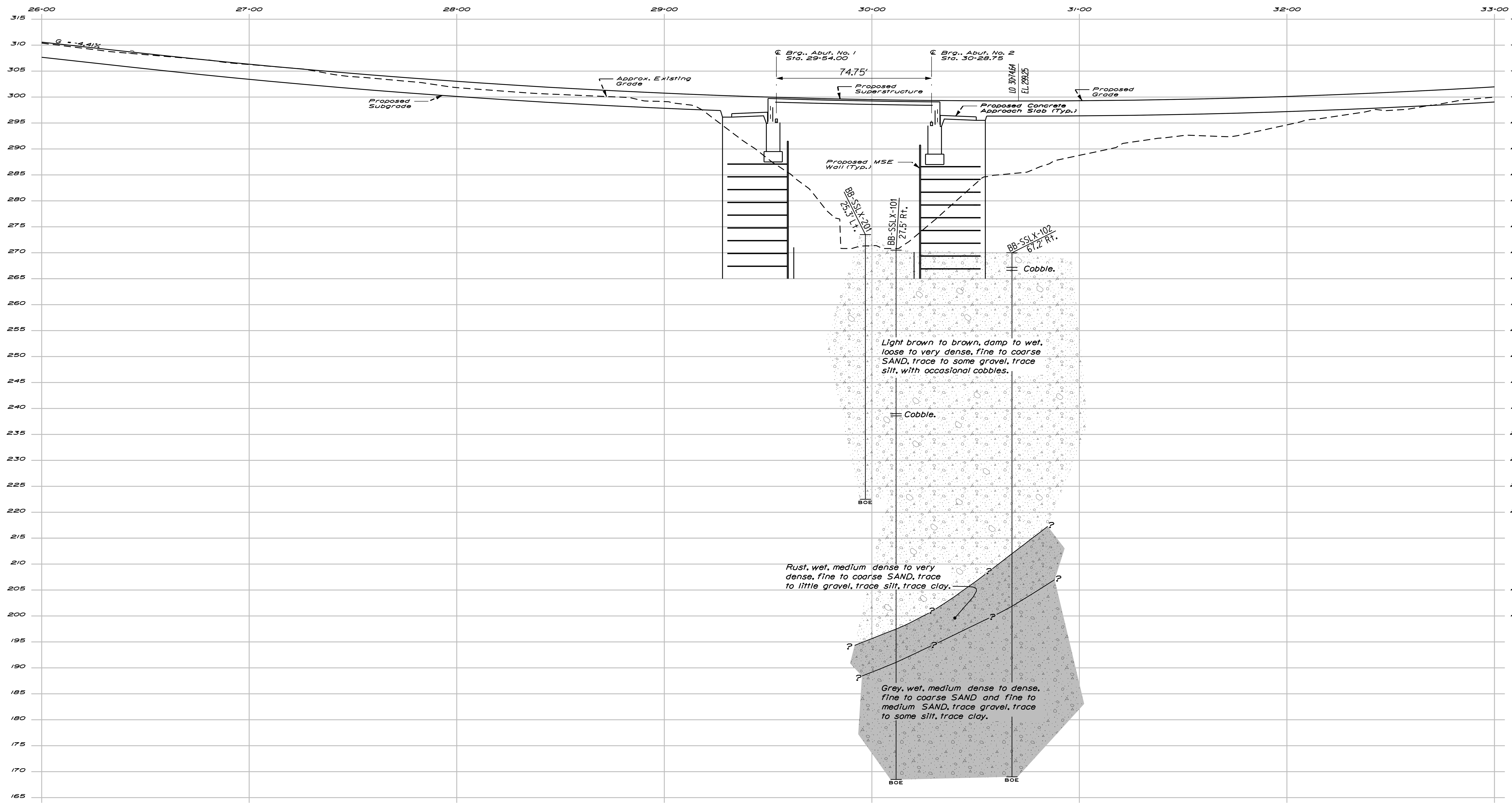
BORING LOCATION PLAN

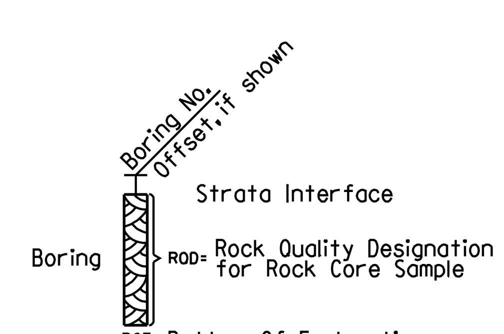
SHEET NUMBER

2

OF 5

Filename: ...\00\geotech\msta\003_ISP1.dgn Division: GEOTECH Username: terry.white Date:7/23/2009





PROFILE

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

BH-1510(700)X

PIN 015107.00

BRIDGE PLANS

BRIDGE NO. 3907

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

SIGNATURE

P.E. NUMBER

DATE

ROUTE 35, SEBAGO LAKE ROAD CROSSING
MAINE CENTRAL RAILROAD
STANDISH ANDROSCOGGIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 5

Filename: ...\msta\004_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:7/23/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine

Location: Standish, Maine

Boring No.: BB-SSLX-201

PIN: 15107.00

Driller:	MaineDOT	Elevation (ft.)	273.5	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:	6/29/09-6/30/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	29+96.9, 25.3 Lt.	Casing ID/OD:	NW	Water Level*:	9.0' 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	1D	24/16	0.00 - 2.00	3/4/5/8	9	13	SSA		Light brown, moist, medium dense, fine to coarse SAND, trace silt, some gravel.	
5	2D	24/15	5.00 - 7.00	7/5/5/5	10	14			Light brown, medium dense, fine to coarse SAND, trace silt, trace gravel.	
10	3D	24/20	10.00 - 12.00	4/4/4/6	8	11	22		Similar to above.	
							25			
							32			
							47			
	4D	24/14	14.00 - 16.00	10/6/4/4	10	14	11		Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
15							17			
							21			
							21			
							25			
	5D	24/10	19.00 - 21.00	11/5/5/5	10	14	10		Similar to above.	
20							19			
							27			
							39			
							45			
	6D	24/9	24.00 - 26.00	11/9/7/6	16	22	23		Similar to above.	
25							OPEN HOLE			
	7D	24/13	29.00 - 31.00	6/5/6/7	11	15			Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
	8D	24/18	34.00 - 36.00	6/6/6/6	12	17			Similar to above.	
	9D	24/18	39.00 - 41.00	6/6/8/10	14	20			Similar to above.	
	10D	24/3	44.00 - 46.00	9/7/9/9	16	22			Similar to above.	
	11D	24/4	49.00 - 51.00	9/7/8/11	15	21			Similar to above.	
51.00								222.50	Bottom of Exploration at 51.00 feet below ground surface. NO REFUSAL	

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-SSLX-201

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine

Location: Standish, Maine

Boring No.: BB-SSLX-101

PIN: 15107.00

Driller:	Northern Test Boring Inc.	Elevation (ft.)	270.5	Auger ID/OD:	2 59/64x6 57/64HSA
Operator:	Mike/Nick	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D50 (Track)	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	2/2/09-2/5/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	30+11.7, 27.5 Rt.	Casing ID/OD:	HW	Water Level*:	16.6' bgs.
Hammer Efficiency Factor: 0.633		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	1D	24/14	0.00 - 2.00	14/12/16/14	28	30	HSA		Dark brown, wet, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt. (2' Frost Depth)	G#212276 A-1-b, SP-SM WC=15.8%
3.00								267.50		
5	2D	24/12	5.00 - 7.00	5/5/7/5	12	13			Light brown, damp, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212277 A-1-b, SP WC=7.2%
10	3D	24/12	10.00 - 12.00	5/4/4/5	8	8			Similar to above, loose.	
15	4D	24/13	15.00 - 17.00	6/8/12/12	20	21			Similar to above, medium dense.	G#212278 A-1-b, SP WC=5.0%
17.00								253.50		
20	5D	24/8	20.00 - 22.00	8/10/10/12	20	21	50		Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212279 A-1-b, SP-SM WC=20.0%
							46			
							55			
							66			
							57			
25	6D	24/14	25.00 - 27.00	4/8/8/6	16	17	43		Similar to above, medium dense.	
							65			
							80			
							107			
							91			
30	7D	18/17	30.00 - 31.50	9/8/50	58	61	52		Similar to above, very dense.	G#212280 A-1-b, SP WC=18.0%
							54		Cobble from 31.5-31.8' bgs.	
							102			
							107			
							102			
35	8D	24/13	35.00 - 37.00	5/7/7/8	14	15	56		Similar to above, medium dense.	
							63			
							91			
							112			
							141			
40	9D	24/12	40.00 - 42.00	8/9/9/8	18	19	80		Similar to above.	G#212281 A-1-b, SP-SM WC=16.6%
							108			
							148			
							174			
							160			
45	MD	24/0	45.00 - 47.00	7/8/11/14	19	20	77		From wash water, similar to above.	
							108			
							147			
							187			
							175			
50	10D	24/14	50.00 - 52.00	4/9/11/14	20	21	62		Brown, wet, medium dense, fine to coarse SAND, little gravel.	
							150			
							173			
							222			
							216			
55	MD	24/0	55.00 - 57.00	7/9/9/10	18	19	125		From wash water, similar to above.	
							136			
							149			
							210			
							230			
60	11D	24/9	60.00 - 62.00	8/12/8/10	20	21	120		Similar to above.	
							175			
							173			
							193			
							207			
65	12D	24/17	65.00 - 67.00	3/9/12/16	21	22	172		Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212282 A-1-b, SP WC=17.0%
							204			
							240			
							238			
							256			
70	MD	24/0	70.00 - 72.00	5/9/10/10	19	20	160		From wash water, similar to above.	
							176			
							251			
73.00							415	197.50		
							605			
75										

Remarks:

Auto Hammer #283

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 2

Boring No.: BB-SSLX-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine

Location: Standish, Maine

Boring No.: BB-SSLX-101

PIN: 15107.00

Driller:	Northern Test Boring Inc.	Elevation (ft.)	270.5	Auger ID/OD:	2 59/64x6 57/64HSA
Operator:	Mike/Nick	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D50 (Track)	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	2/2/09-2/5/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	30+11.7, 27.5 Rt.	Casing ID/OD:	HW	Water Level*:	16.6' bgs.
Hammer Efficiency Factor: 0.633		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
75	13D	24/18	75.00 - 77.00	15/26/24/26	50	53	206		Rust, wet, very dense, fine to coarse SAND, trace silt, trace clay, trace gravel.	G#212283 A-1-b, SC-SM WC=20.3%
							243			
							308			
							432			
79.00							401	191.50		
80	14D	24/9	80.00 - 82.00	13/14/20/17	34	36	183		Grey-brown, wet, dense, fine to coarse SAND, trace gravel.	
							240			
							303			
							300			
84.00							282	186.50		
85	15D	24/16	85.00 - 87.00	8/10/13/21	23	24	275		Light brown, wet, medium dense, fine to medium SAND, trace silt.	G#212284 A-3, SP-SM WC=25.5%
							322			
							336			
							320			
89.00							300	181.50		
90	16D	24/18	90.00 - 92.00	6/12/20/27	32	34	252		Grey, wet, dense, fine to medium SAND, trace silt.	G#212285 A-3, SP-SM WC=25.5%
							305			
							342			
							411			
							426			
95	17D	24/12	95.00 - 97.00	8/9/17/24	26	27	361		Similar to above, medium dense.	
							398			
							402			
							427			
							448			
100	18D	24/13	100.00 - 102.00	6/13/34/19	47	50			Grey, wet, dense, fine to medium SAND, some silt, trace gravel.	G#212286 A-2-4, SM WC=24.4%
102.00								168.50	Bottom of Exploration at 102.00 feet below ground surface. NO REFUSAL	
125										

Remarks:

Auto Hammer #283

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

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

Page 2 of 2

Boring No.: BB-SSLX-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1510(700)X

BRIDGE NO. 3907

PIN 015107.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	________		
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	FEB 2009
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

ROUTE 35, SEBAGO LAKE ROAD CROSSING
MAINE CENTRAL RAILROAD
STANDISH ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER

4

OF 5

Filename: ...\msta\005_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:7/23/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine
Location: Standish, Maine

Boring No.: BB-SSLX-102
PIN: 15107.00

Driller:	MaineDOT	Elevation (ft.)	270.0	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:	2/2/09-2/4/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	30+67.5, 67.2 Rt.	Casing ID/OD:	NW	Water Level*:	17.0' bgs.
Hammer Efficiency Factor: 0.77		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		2.0' Frost Depth.	
									Cobble from 2.8-3.4' bgs.	
5	1D	24/18	5.00 - 7.00	2/5/7/9	12	15			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, occasional cobble.	G#212287 A-1-b, SP-SM WC=4.1%
10	2D	24/4	10.00 - 12.00	3/4/5/4	9	12			Similar to above.	
15	3D	24/16	15.00 - 17.00	4/5/9/11	14	18	89		Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt.	G#212288 A-1-b, SP-SM WC=3.2%
							61			
							48			
							47			
	4D	24/5	19.00 - 21.00	10/15/7/5	22	28	7		Similar to above, wet, medium dense.	
20							36			
							41			
							32			
							33			
	MD	24/1	24.00 - 26.00	10/10/10/8	20	26	8		From wash water, similar to above.	
25							25			
							31			
							42			
							40			
	5D	24/14	29.00 - 31.00	7/4/5/5	9	12	24		Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212289 A-1-b, SP WC=18.5%
30							41			
							40			
							51			
							46			
	MD	24/0	34.00 - 36.00	9/7/8/8	15	19	24		From wash water, similar to above.	
35							50			
							54			
							55			
							63			
	6D	24/12	39.00 - 41.00	12/4/5/5	9	12	27		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#212290 A-1-b, SP WC=15.0%
40							53			
							71			
							70			
							71			
	7D	24/9	44.00 - 46.00	9/4/6/12	10	13	29		Similar to above.	
45							63			
							93			
							89			
							96			
	8D	24/9	49.00 - 51.00	14/6/6/10	12	15	48		Similar to above.	
50							84			
							99			
							97			
							98			
	9D	24/10	54.00 - 56.00	18/13/9/15	22	28	45		Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#212291 A-1-b, SP-SM WC=14.5%
55							47			
							85			
							91			
							87	212.00	58.00	
	10D	24/14	59.00 - 61.00	7/7/8/17	15	19	45		Rust, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#212292 A-1-b, SP-SM WC=17.0%
60							53			
							76			
							75			
							77			
	11D	24/6	64.00 - 66.00	9/7/8/10	15	19	43	206.00	64.00 Rust, wet, medium dense, fine to coarse SAND, little gravel.	
65							69			
							113			
							114			
							117	202.00	68.00	
	12D	24/10	69.00 - 71.00	13/10/9/11	19	24	71		Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#212293 A-1-b, SP-SM WC=19.7%
70							78			
							119			
							121			
							130			
	13D	24/13	74.00 - 76.00	10/7/12/15	19	24	75		Similar to above.	G#212294 A-3, SP-SM
75										

Remarks:

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

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

Page 1 of 2

Boring No.: BB-SSLX-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine
Location: Standish, Maine

Boring No.: BB-SSLX-102
PIN: 15107.00

Driller:	MaineDOT	Elevation (ft.)	270.0	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:	2/2/09-2/4/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	30+67.5, 67.2 Rt.	Casing ID/OD:	NW	Water Level*:	17.0' bgs.
Hammer Efficiency Factor: 0.77		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
75							73			WC=21.2%
							107			
							129			
							138			
	14D	24/15	79.00 - 81.00	16/10/12/16	22	28	77		Similar to above.	G#212295 A-3, SP-SM WC=20.7%
80							99			
							125			
							127			
							128	187.00	83.00	
	15D	24/8	84.00 - 86.00	12/10/11/17	21	27	89		Grey, wet, medium dense, fine to medium SAND, trace silt.	
85							111			
							167			
							171			
							185			
	16D	24/12	89.00 - 91.00	12/12/15/19	27	35	77		Similar to above, dense.	G#212296 A-3, SP-SM WC=22.5%
90							101			
							141			
							145			
							156			
	17D	24/8	94.00 - 96.00	8/9/8/15	17	22	117		Similar to above, medium dense.	
95							116			
							147			
							179			
							189			
	18D	24/16	99.00 - 101.00	15/14/15/16	29	37			Grey, wet, dense, fine SAND, some silt, trace medium sand, trace clay.	G#212297 A-2-4, SC-SM WC=25.5%
100										
								169.00	101.00 Bottom of Exploration at 101.00 feet below ground surface. NO REFUSAL	
105										
110										
115										
120										
125										

Remarks:

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

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Page 2 of 2

Boring No.: BB-SSLX-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1510(700)X

BRIDGE NO. 3907 PIN 015107.00 BRIDGE PLANS

PROJ. MANAGER	______	BY	DATE
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	FEB 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

ROUTE 35, SEBAGO LAKE ROAD CROSSING
MAINE CENTRAL RAILROAD
STANDISH ANDROSCOGGIN 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 thumbnail
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

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
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

January 2008

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-101
PIN: 15107.00

Driller:	Northern Test Boring Inc.	**Elevation (ft.)**	270.5	**Auger ID/OD:**	2½"x6¼" HSA
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 (Track)	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/2/09-2/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+11.7, 27.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.6' bgs.
Hammer Efficiency Factor:	0.633	**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.
0	1D	24/14	0.00 - 2.00	14/12/16/14	28	30	HSA			Dark brown, wet, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt. (2' Frost Depth)	G#212276 A-1-b, SP-SM WC=15.8%
								267.50		3.00	
5	2D	24/12	5.00 - 7.00	5/5/7/5	12	13				Light brown, damp, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212277 A-1-b, SP WC=7.2%
10	3D	24/12	10.00 - 12.00	5/4/4/5	8	8				Similar to above, loose.	
15	4D	24/13	15.00 - 17.00	6/8/12/12	20	21				Similar to above, medium dense.	G#212278 A-1-b, SP WC=5.0%
								253.50		17.00	
20	5D	24/8	20.00 - 22.00	8/10/10/12	20	21	50			Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212279 A-1-b, SP-SM WC=20.0%
							46				
							55				
							66				
25							57				

Remarks:

Auto Hammer #283

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Maine Department of Transportation

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US CUSTOMARY UNITS

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PIN: 15107.00

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Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 (Track)	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/2/09-2/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+11.7, 27.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.6' bgs.
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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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_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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LL = Liquid Limit
PL = Plastic Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/14	25.00 - 27.00	4/8/8/6	16	17	43			Similar to above, medium dense.	
							65				
							80				
							107				
							91				
30	7D	18/17	30.00 - 31.50	9/8/50	58	61	52			Similar to above, very dense.	G#212280 A-1-b, SP WC=18.0%
							54			Cobble from 31.5-31.8' bgs.	
							102				
							107				
							102				
35	8D	24/13	35.00 - 37.00	5/7/7/8	14	15	56			Similar to above, medium dense.	
							63				
							91				
							112				
							141				
40	9D	24/12	40.00 - 42.00	8/9/9/8	18	19	80			Similar to above.	G#212281 A-1-b, SP-SM WC=16.6%
							108				
							148				
							174				
							160				
45	MD	24/0	45.00 - 47.00	7/8/11/14	19	20	77			From wash water, similar to above.	
							108				
							147				
							187				
50							175				

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-101

PIN: 15107.00

Driller:	Northern Test Boring Inc.	**Elevation (ft.)**	270.5	**Auger ID/OD:**	2½"x6¼" HSA
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 (Track)	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/2/09-2/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+11.7, 27.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.6' bgs.
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/14	50.00 - 52.00	4/9/11/14	20	21	62			Brown, wet, medium dense, fine to coarse SAND, little gravel.	
							150				
							173				
							222				
							216				
55	MD	24/0	55.00 - 57.00	7/9/9/10	18	19	125			From wash water, similar to above.	
							136				
							149				
							210				
							230				
60	11D	24/9	60.00 - 62.00	8/12/8/10	20	21	120			Similar to above.	
							175				
							173				
							193				
							207				
65	12D	24/17	65.00 - 67.00	3/9/12/16	21	22	172			Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212282 A-1-b, SP WC=17.0%
							204				
							240				
							238				
							256				
70	MD	24/0	70.00 - 72.00	5/9/10/10	19	20	160			From wash water, similar to above.	
							176				
							251				
							415	197.50		73.00	
							605				
75											

Remarks:

Auto Hammer #283

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Boring Location:	30+11.7, 27.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.6' bgs.
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	13D	24/18	75.00 - 77.00	15/26/24/26	50	53	206			Rust, wet, very dense, fine to coarse SAND, trace silt, trace clay, trace gravel.	G#212283 A-1-b, SC-SM WC=20.3%
							243				
							308				
							432				
							401	191.50		79.00	
80	14D	24/9	80.00 - 82.00	13/14/20/17	34	36	183			Grey-brown, wet, dense, fine to coarse SAND, trace gravel.	
							240				
							303				
							300				
							282	186.50		84.00	
85	15D	24/16	85.00 - 87.00	8/10/13/21	23	24	275			Light brown, wet, medium dense, fine to medium SAND, trace silt.	G#212284 A-3, SP-SM WC=25.5%
							322				
							336				
							320				
							300	181.50		89.00	
90	16D	24/18	90.00 - 92.00	6/12/20/27	32	34	252			Grey, wet, dense, fine to medium SAND, trace silt.	G#212285 A-3, SP-SM WC=25.5%
							305				
							342				
							411				
							426				
95	17D	24/12	95.00 - 97.00	8/9/17/24	26	27	361			Similar to above, medium dense.	
							398				
							402				
							427				
100							448				

Remarks:

Auto Hammer #283

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PIN: 15107.00

Driller:	Northern Test Boring Inc.	**Elevation (ft.)**	270.5	**Auger ID/OD:**	2½"x6¼" HSA
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D50 (Track)	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/2/09-2/5/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+11.7, 27.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.6' bgs.
Hammer Efficiency Factor:	0.633	**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.
100	18D	24/13	100.00 - 102.00	6/13/34/19	47	50				Grey, wet, dense, fine to medium SAND, some silt, trace gravel.	G#212286 A-2-4, SM WC=24.4%
								168.50		102.00	
										Bottom of Exploration at 102.00 feet below ground surface. NO REFUSAL	
105											
110											
115											
120											
125											

Remarks:

Auto Hammer #283

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: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-102
PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	270.0	**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:	2/2/09-2/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+67.5, 67.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	17.0' bgs.
Hammer Efficiency Factor:	0.77	**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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			2.0' Frost Depth.	
										Cobble from 2.8-3.4' bgs.	
5	1D	24/18	5.00 - 7.00	2/5/7/9	12	15				Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, occasional cobble.	G#212287 A-1-b, SP-SM WC=4.1%
10	2D	24/4	10.00 - 12.00	3/4/5/4	9	12				Similar to above.	
15	3D	24/16	15.00 - 17.00	4/5/9/11	14	18	89			Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt.	G#212288 A-1-b, SP-SM WC=3.2%
							61				
							48				
							47				
	4D	24/5	19.00 - 21.00	10/15/7/5	22	28	7			Similar to above, wet, medium dense.	
20							36				
							41				
							32				
							33				
25	MD	24/1	24.00 - 26.00	10/10/10/8	20	26	8			From wash water, similar to above.	

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad **Location:** Standish, Maine	**Boring No.:** BB-SSLX-102 **PIN:** 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	270.0	**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:	2/2/09-2/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+67.5, 67.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	17.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
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							25				
							31				
							42				
							40				
	5D	24/14	29.00 - 31.00	7/4/5/5	9	12	24			Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#212289 A-1-b, SP WC=18.5%
30							41				
							40				
							51				
							46				
	MD	24/0	34.00 - 36.00	9/7/8/8	15	19	24			From wash water, similar to above.	
35							50				
							54				
							55				
							63				
	6D	24/12	39.00 - 41.00	12/4/5/5	9	12	27			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#212290 A-1-b, SP WC=15.0%
40							53				
							71				
							70				
							71				
	7D	24/9	44.00 - 46.00	9/4/6/12	10	13	29			Similar to above.	
45							63				
							93				
							89				
							96				
	8D	24/9	49.00 - 51.00	14/6/6/10	12	15	48			Similar to above.	
50											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad Location: Standish, Maine	Boring No.: BB-SSLX-102 PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	270.0	**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:	2/2/09-2/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+67.5, 67.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	17.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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
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PI = Plasticity Index
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							84				
							99				
							97				
							98				
	9D	24/10	54.00 - 56.00	18/13/9/15	22	28	45			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#212291 A-1-b, SP-SM WC=14.5%
55							47				
							85				
							91				
							87	212.00		58.00	
	10D	24/14	59.00 - 61.00	7/7/8/17	15	19	45			Rust, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#212292 A-1-b, SP-SM WC=17.0%
60							53				
							76				
							75				
							77	206.00		64.00	
	11D	24/6	64.00 - 66.00	9/7/8/10	15	19	43			Rust, wet, medium dense, fine to coarse SAND, little gravel.	
65							69				
							113				
							114	202.00		68.00	
							117				
	12D	24/10	69.00 - 71.00	13/10/9/11	19	24	71			Grey, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#212293 A-1-b, SP-SM WC=19.7%
70							78				
							119				
							121				
							130				
75	13D	24/13	74.00 - 76.00	10/7/12/15	19	24	75			Similar to above.	G#212294 A-3, SP-SM

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-102
PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	270.0	**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:	2/2/09-2/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+67.5, 67.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	17.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
75							73				WC=21.2%
							107				
							129				
							138				
	14D	24/15	79.00 - 81.00	16/10/12/16	22	28	77			Similar to above.	G#212295 A-3, SP-SM WC=20.7%
80							99				
							125				
							127				
							128	187.00		83.00	
	15D	24/8	84.00 - 86.00	12/10/11/17	21	27	89			Grey, wet, medium dense, fine to medium SAND, trace silt.	
85							111				
							167				
							171				
							185				
	16D	24/12	89.00 - 91.00	12/12/15/19	27	35	77			Similar to above, dense.	G#212296 A-3, SP-SM WC=22.5%
90							101				
							141				
							145				
							156				
	17D	24/8	94.00 - 96.00	8/9/8/15	17	22	117			Similar to above, medium dense.	
95							116				
							147				
							179				
							189				
100	18D	24/16	99.00 - 101.00	15/14/15/16	29	37				Grey, wet, dense, fine SAND, some silt, trace medium sand, trace clay.	G#212297 A-2-4, SC-SM

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-102
PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	270.0	**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:	2/2/09-2/4/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	30+67.5, 67.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	17.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100											WC=25.5%
								169.00		101.00 **Bottom of Exploration at 101.00 feet below ground surface.** NO REFUSAL	
105											
110											
115											
120											
125											

Remarks:

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Boring No.: BB-SSLX-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-201
PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	273.5	**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:	6/29/09-6/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	29+96.9, 25.3 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	3/4/5/8	9	13	SSA			Light brown, moist, medium dense, fine to coarse SAND, trace silt, some gravel.	
5	2D	24/15	5.00 - 7.00	7/5/5/5	10	14				Light brown, medium dense, fine to coarse SAND, trace silt, trace gravel.	
10	3D	24/20	10.00 - 12.00	4/4/4/6	8	11	22			Similar to above.	
							25				
							32				
							47				
	4D	24/14	14.00 - 16.00	10/6/4/4	10	14	11			Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
15							17				
							21				
							21				
							25				
	5D	24/10	19.00 - 21.00	11/5/5/5	10	14	10			Similar to above.	
20							19				
							27				
							39				
							45				
25	6D	24/9	24.00 - 26.00	11/9/7/6	16	22	23			Similar to above.	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad
Location: Standish, Maine

Boring No.: BB-SSLX-201
PIN: 15107.00

Driller:	MaineDOT	**Elevation (ft.)**	273.5	**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:	6/29/09-6/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	29+96.9, 25.3 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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SSA = Solid Stem Auger
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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

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							OPEN HOLE				
	7D	24/13	29.00 - 31.00	6/5/6/7	11	15				Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	
30											
	8D	24/18	34.00 - 36.00	6/6/6/6	12	17				Similar to above.	
35											
	9D	24/18	39.00 - 41.00	6/6/8/10	14	20				Similar to above.	
40											
	10D	24/3	44.00 - 46.00	9/7/9/9	16	22				Similar to above.	
45											
	11D	24/4	49.00 - 51.00	9/7/8/11	15	21				Similar to above.	
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 35, Sebago Lake Road Crossing Bridge #3907 over Maine Central Railroad	**Boring No.:**	BB-SSLX-201		
Location:	Standish, Maine	**PIN:**	15107.00		
Driller:	MaineDOT	**Elevation (ft.)**	273.5	**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:	6/29/09-6/30/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	29+96.9, 25.3 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' 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.
50								222.50		51.00 **Bottom of Exploration at 51.00 feet below ground surface.** NO REFUSAL	
55											
60											
65											
70											
75											

Remarks:

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

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

Boring No.: BB-SSLX-201

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Standish   Project Number: 15107.00

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-SSLX-101, 1D	30+11.7	27.5 Rt.	0.0-2.0	212276	1	15.8			SP-SM	A-1-b	0
BB-SSLX-101, 2D	30+11.7	27.5 Rt.	5.0-7.0	212277	1	7.2			SP	A-1-b	0
BB-SSLX-101, 4D	30+11.7	27.5 Rt.	15.0-17.0	212278	1	5.0			SP	A-1-b	0
BB-SSLX-101, 5D	30+11.7	27.5 Rt.	20.0-22.0	212279	1	20.0			SP-SM	A-1-b	0
BB-SSLX-101, 7D	30+11.7	27.5 Rt.	30.0-31.5	212280	1	18.0			SP	A-1-b	0
BB-SSLX-101, 9D	30+11.7	27.5 Rt.	40.0-42.0	212281	1	16.6			SP-SM	A-1-b	0
BB-SSLX-101, 12D	30+11.7	27.5 Rt.	65.0-67.0	212282	2	17.0			SP	A-1-b	0
BB-SSLX-101, 13D	30+11.7	27.5 Rt.	75.0-77.0	212283	2	20.3			SC-SM	A-1-b	II
BB-SSLX-101, 15D	30+11.7	27.5 Rt.	85.0-87.0	212284	2	25.5			SP-SM	A-3	0
BB-SSLX-101, 16D	30+11.7	27.5 Rt.	90.0-92.0	212285	2	25.5			SP-SM	A-3	0
BB-SSLX-101, 18D	30+11.7	27.5 Rt.	100.0-102.0	212286	2	24.4			SM	A-2-4	II
BB-SSLX-102, 1D	30+67.5	67.2 Rt.	5.0-7.0	212287	3	4.1			SP-SM	A-1-b	0
BB-SSLX-102, 3D	30+67.5	67.2 Rt.	15.0-17.0	212288	3	3.2			SP-SM	A-1-b	0
BB-SSLX-102, 5D	30+67.5	67.2 Rt.	29.0-31.0	212289	3	18.5			SP	A-1-b	0
BB-SSLX-102, 6D	30+67.5	67.2 Rt.	39.0-41.0	212290	3	15.0			SP	A-1-b	0
BB-SSLX-102, 9D	30+67.5	67.2 Rt.	54.0-56.0	212291	3	14.5			SP-SM	A-1-b	0
BB-SSLX-102, 10D	30+67.5	67.2 Rt.	59.0-61.0	212292	3	17.0			SP-SM	A-1-b	0
BB-SSLX-102, 12D	30+67.5	67.2 Rt.	69.0-71.0	212293	4	19.7			SP-SM	A-1-b	0
BB-SSLX-102, 13D	30+67.5	67.2 Rt.	74.0-76.0	212294	4	21.2			SP-SM	A-3	0
BB-SSLX-102, 14D	30+67.5	67.2 Rt.	79.0-81.0	212295	4	20.7			SP-SM	A-3	0
BB-SSLX-102, 16D	30+67.5	67.2 Rt.	89.0-91.0	212296	4	22.5			SP-SM	A-3	0
BB-SSLX-102, 18D	30+67.5	67.2 Rt.	99.0-101.0	212297	4	25.5			SC-SM	A-2-4	II

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)

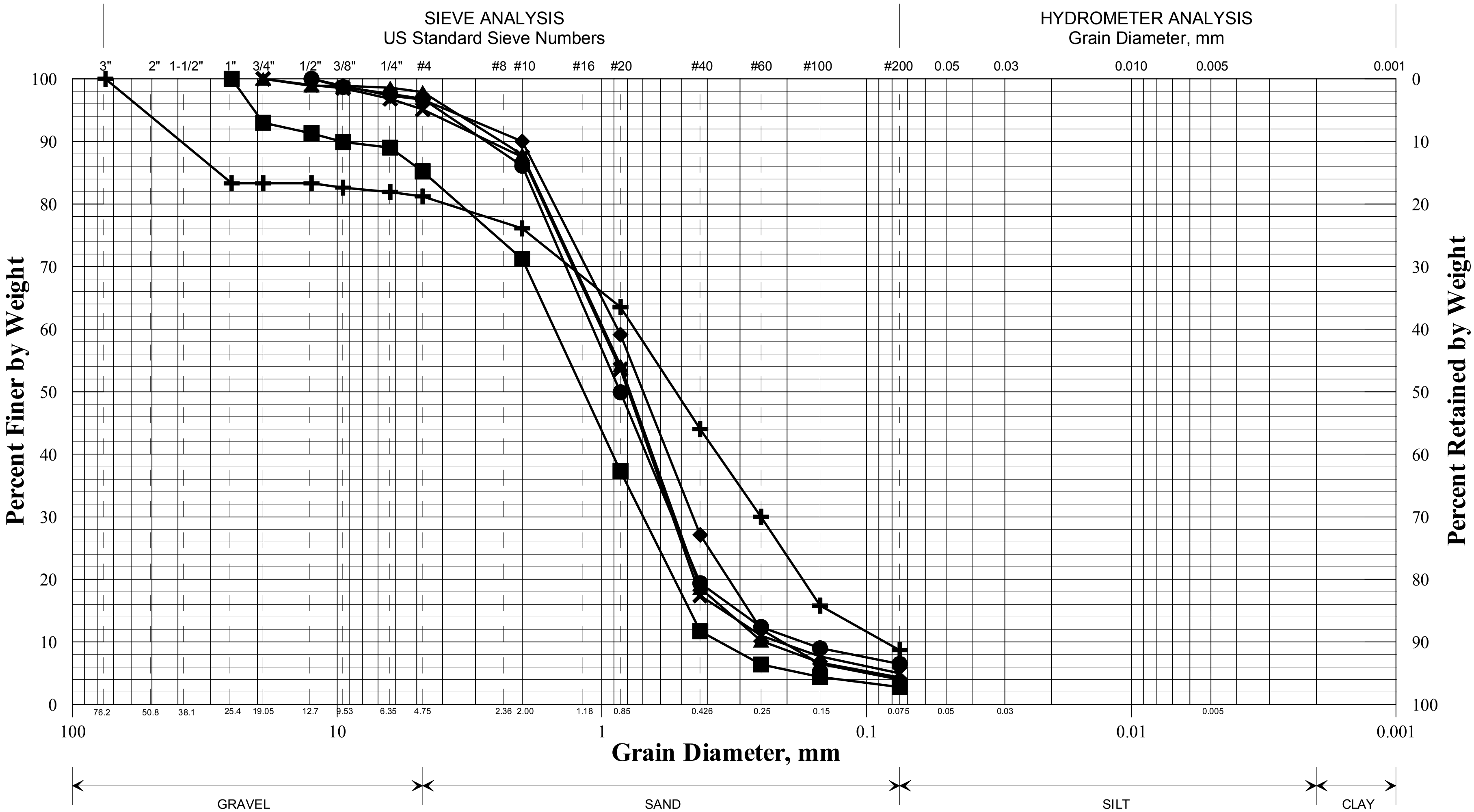
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

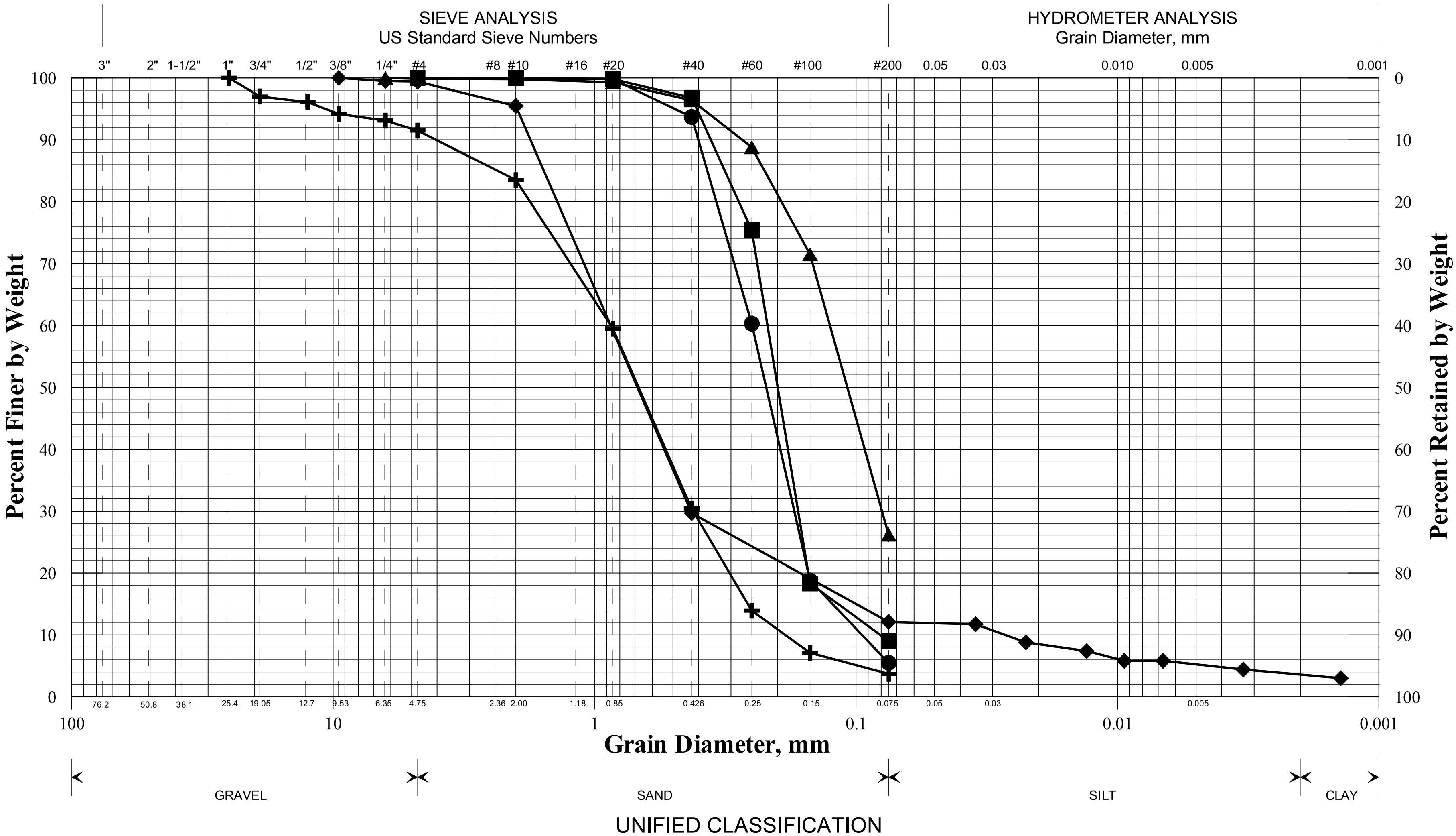


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SSLX-101/1D	30+11.7	27.5 RT	0.0-2.0	SAND, little gravel, trace silt.	15.8			
◆	BB-SSLX-101/2D	30+11.7	27.5 RT	5.0-7.0	SAND, trace silt, trace gravel.	7.2			
■	BB-SSLX-101/4D	30+11.7	27.5 RT	15.0-17.0	SAND, little gravel, trace silt.	5.0			
●	BB-SSLX-101/5D	30+11.7	27.5 RT	20.0-22.0	SAND, trace silt, trace gravel.	20.0			
▲	BB-SSLX-101/7D	30+11.7	27.5 RT	30.0-31.5	SAND, trace silt, trace gravel.	18.0			
✖	BB-SSLX-101/9D	30+11.7	27.5 RT	40.0-42.0	SAND, trace silt, trace gravel.	16.6			

PIN
015107.00
Town
Standish
Reported by/Date
WHITE, TERRY A 2/18/2009

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SSLX-101/12D	30+11.7	27.5 RT	65.0-67.0	SAND, trace gravel, trace silt.	17.0			
◆	BB-SSLX-101/13D	30+11.7	27.5 RT	75.0-77.0	SAND, trace silt, trace clay, trace gravel.	20.3			
■	BB-SSLX-101/15D	30+11.7	27.5 RT	85.0-87.0	SAND, trace silt.	25.5			
●	BB-SSLX-101/16D	30+11.7	27.5 RT	90.0-92.0	SAND, trace silt.	25.5			
▲	BB-SSLX-101/18D	30+11.7	27.5 RT	100.0-102.0	SAND, some silt, trace gravel.	24.4			
✖									

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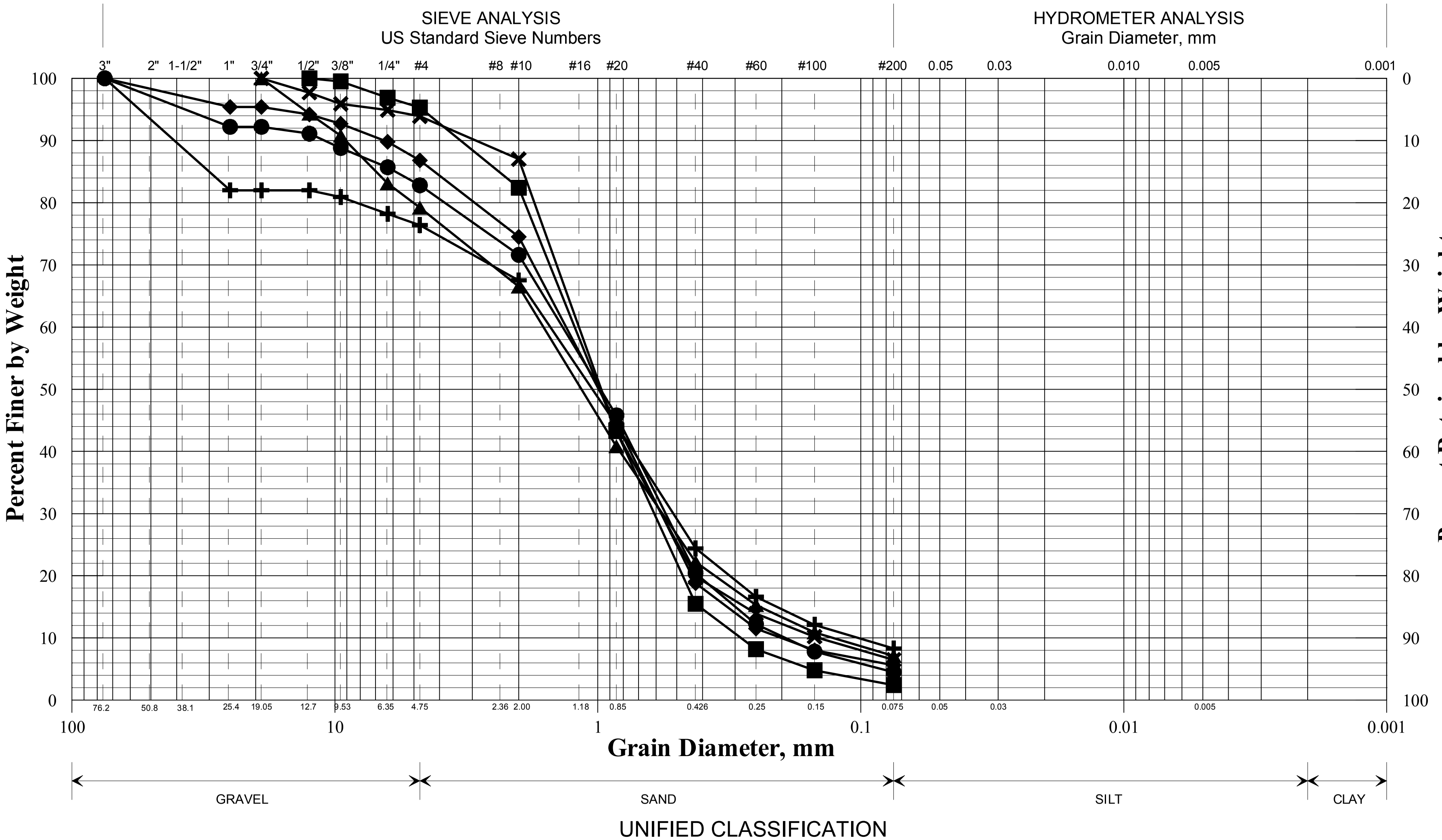
015107.00
Town
Standish
Reported by/Date
WHITE, TERRY A 2/18/2009

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SSLX-102/1D	30+67.5	67.2 RT	5.0-7.0	SAND, some gravel, trace silt.	4.1			
◆	BB-SSLX-102/3D	30+67.5	67.2 RT	15.0-17.0	SAND, little gravel, trace silt.	3.2			
■	BB-SSLX-102/5D	30+67.5	67.2 RT	29.0-31.0	SAND, trace gravel, trace silt.	18.5			
●	BB-SSLX-102/6D	30+67.5	67.2 RT	39.0-41.0	SAND, little gravel, trace silt.	15.0			
▲	BB-SSLX-102/9D	30+67.5	67.2 RT	54.0-56.0	SAND, some gravel, trace silt.	14.5			
✖	BB-SSLX-102/10D	30+67.5	67.2 RT	59.0-61.0	SAND, trace silt, trace gravel.	17.0			

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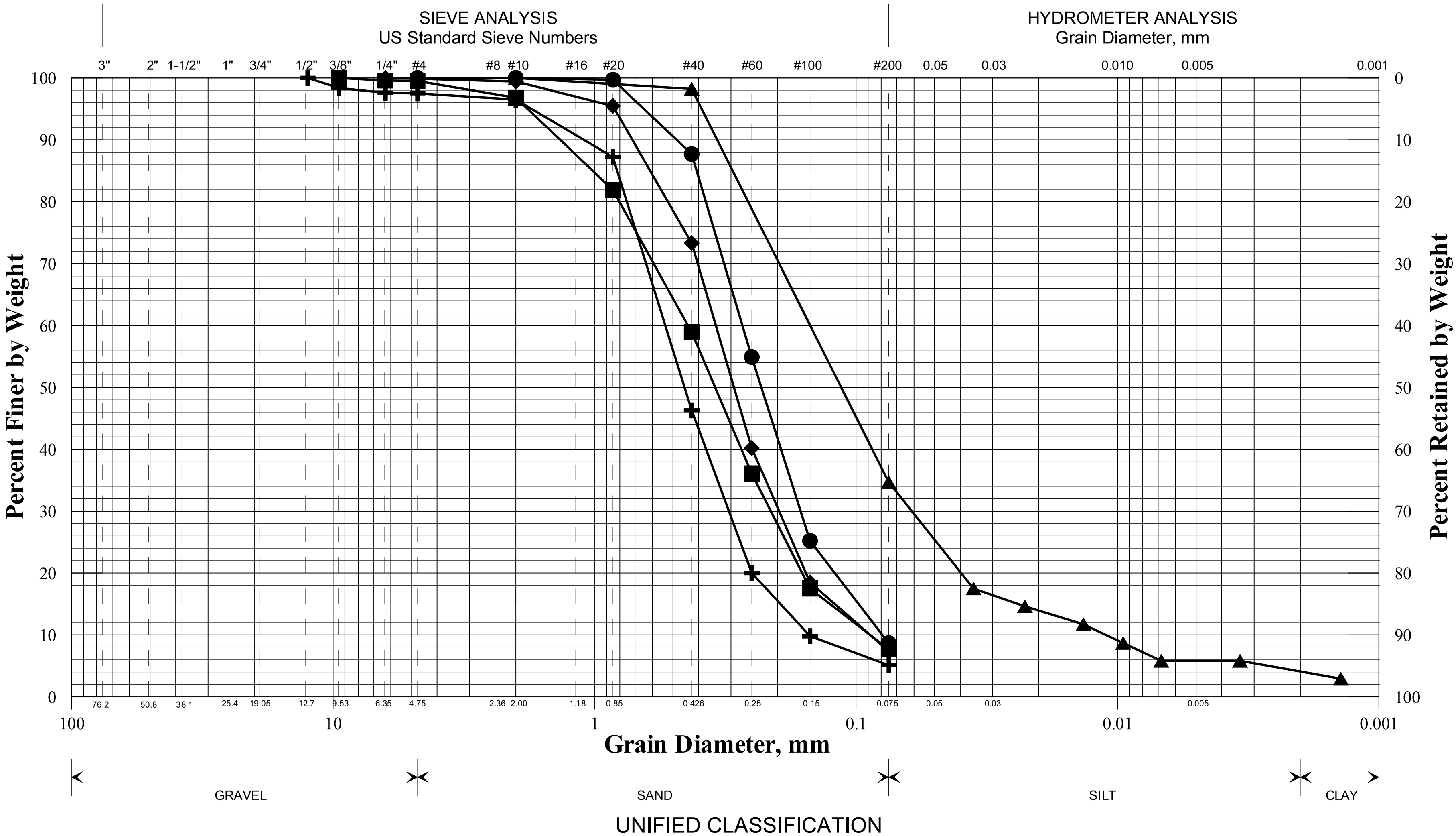
Town
Standish
Reported by/Date
WHITE, TERRY A 2/18/2009

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SSLX-102/12D	30+67.5	67.2 RT	69.0-71.0	SAND, trace silt, trace gravel.	19.7			
◆	BB-SSLX-102/13D	30+67.5	67.2 RT	74.0-76.0	SAND, trace silt, trace gravel.	21.2			
■	BB-SSLX-102/14D	30+67.5	67.2 RT	79.0-81.0	SAND, trace silt, trace gravel.	20.7			
●	BB-SSLX-102/16D	30+67.5	67.2 RT	89.0-91.0	SAND, trace silt.	22.5			
▲	BB-SSLX-102/18D	30+67.5	67.2 RT	99.0-101.0	SAND, some silt, trace clay.	25.5			
✖									

PIN	
015107.00	
Town	
Standish	
Reported by/Date	
WHITE, TERRY A	2/18/2009

SHEET 4

Appendix C

Calculations

Bearing Resistance:

For any foundation on native subgrade soils including MSE Mass and wall leveling pad:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on compacted MSE fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Type of Bearing Material: Coarse to medium sand, with little gravel (SW, SP)

Compacted MSE wall backfill - Soils will be medium dense to dense

Consistency In Place: Medium Dense to Dense

Bearing Resistance: Ordinary Range (ksf) 4 - 8

Recommended Value of Use (ksf): 6 ksf

Recommended Value: $q_{nom} := 6 \cdot ksf$

Resistance factor at the **service limit state** Φ=1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := q_{nom} \cdot 1.0$ $\qquad q_{factored\_bc} = 6 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Reference: Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-2 Bearing Capacity

Assumptions:

1. Footings will be embedded 5.0 feet for frost protection. $D_f := 5.0 \cdot ft$

2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4, pg 163)

   Saturated unit weight: $\gamma_s := 125 \cdot pcf$

   Dry unit weight: $\gamma_d := 120 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 32 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$

3. Use Terzaghi strip equations as L>B

4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 16 \cdot ft$ Based on boring logs

Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1 pg 220

For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=32 deg

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w) \qquad q = 1.231 \cdot ksf$$

$$q_{ult} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w) B \cdot N_\gamma \cdot s_\gamma \qquad q_{ult} = \begin{pmatrix} 32 \\ 34 \\ 35 \\ 37 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Apply 0.45 resistance factor to get factored resistance.

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{ult} \cdot \phi_b$$

$$q_{factored} = \begin{pmatrix} 14.4 \\ 15.3 \\ 16 \\ 16.6 \end{pmatrix} \cdot ksf$$

Based on these footing widths:

$$B = \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 14 ksf

Bearing Resistance: For Bridge Abutment Spread Footings on MSE Select Backfill Soils:

Part 1 - Service Limit State

Based on "Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines
FHWA Publication No. FHWA-NHI-00-043 March 2001

From Section 5.1a MSEW Abutments on Spread Footings

Limit the bearing capacity on the reinforced volume to 4,000 psf (page 172)

Recommended Value: $q_{nom} := 4 \cdot ksf$

Resistance factor at the **service limit state** Φ=1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := q_{nom} \cdot 1.0$ $\qquad$ $q_{factored\_bc} = 4 \cdot ksf$

Part 2 - Strength Limit State

Factored Bearing Resistance - abutment footing on MSEW mass

Based on information from FHWA regarding revisions to the FHWA NHI MSE wall manual the following bearing resistance is recommended at the strength limit state:

$q_{factored\_str} := 7 \cdot ksf$

The revised manual has not been published as of this report.
The revisions to the manual will note that AASHTO does not provide a value of factored bearing resistance at strength limit state and the recommended value is based on the authors' experience.)
("Authors'" refers to the authors of the revised FHWA NHI MSE wall manual.)

Active Earth Pressures:

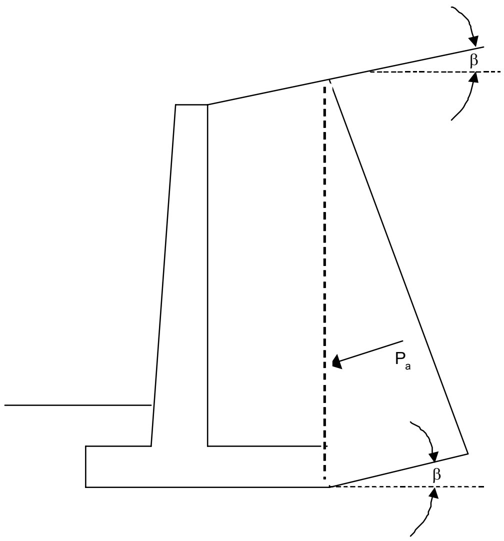
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

unit weight: $\gamma_{type4} := 125 \cdot pcf$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

Cohesion: $c_{sand} := 0 \cdot psf$

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



Generally use Rankine for long heeled cantilever walls where the failure surface is un interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with horizontal backfill surface:

$$K_{a\_rankine} := \tan\left(45 \cdot deg - \frac{\phi_{type4}}{2}\right)^2 \qquad K_{a\_rankine} = 0.307$$

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{a\_rankine\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}} \qquad K_{a\_rankine\_slope} = 0.307$$

Pa is oriented at an angle of β to the vertical plane.

Passive Earth Pressure:

Coulomb Theory - Passive Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.6 pg 3-8

Angle of back face of wall to the horizontal: $\alpha := 90 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

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

$K_p = 6.89$

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

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

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

$K_{p\_rank} = 3.25$

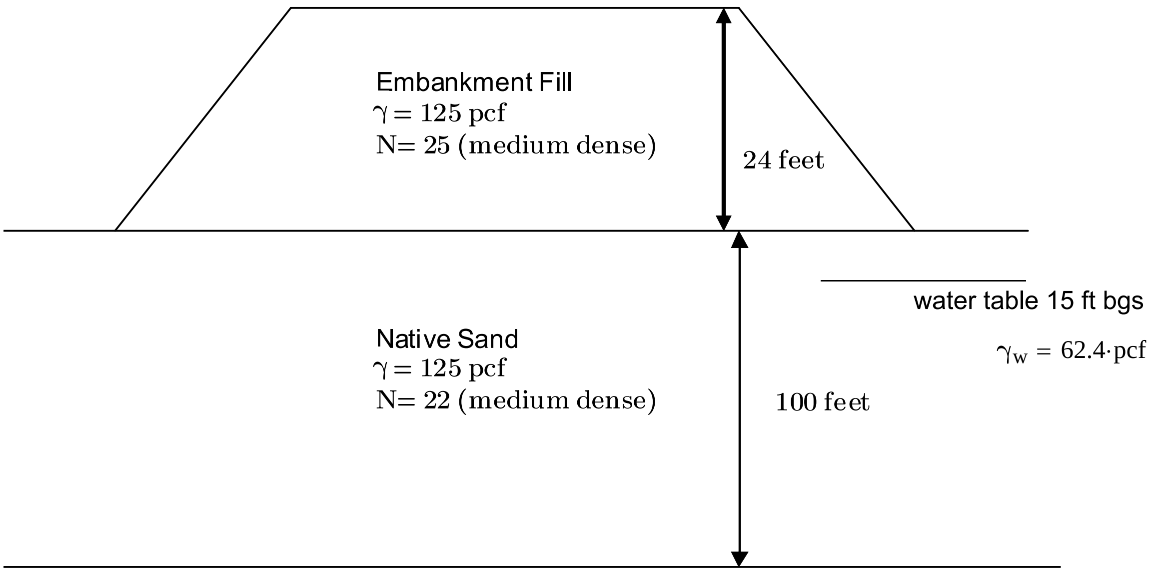
Bowles does not recommend the use of the Rankine Method for K_p when β>0.

Settlement Analysis:

Reference: FHWA Soils and Foundation Reference Manual - Volume 1 (FHWA NHI-06-088) 2006 pg 7-16

Look at fill for MSE Wall supported spread footing abutments
Maximum of ~11 feet of fill at Abutment No. 1
Maximum of ~24 feet of fill at Abutment No. 2

Worst case:
Boring BB-SSLX-101
with 24 feet of fill



Divide sand and gravel layer up into 10 ' layers:

Layer 1:	$H_1 := 10 \cdot ft$	$N_1 := 22$
Layer 2:	$H_2 := 10 \cdot ft$	$N_2 := 15$
Layer 3:	$H_3 := 10 \cdot ft$	$N_3 := 19$
Layer 4:	$H_4 := 10 \cdot ft$	$N_4 := 43$
Layer 5:	$H_5 := 10 \cdot ft$	$N_5 := 38$
Layer 6:	$H_6 := 10 \cdot ft$	$N_6 := 20$
Layer 7:	$H_7 := 10 \cdot ft$	$N_7 := 20$
Layer 8:	$H_8 := 10 \cdot ft$	$N_8 := 21$
Layer 9:	$H_9 := 10 \cdot ft$	$N_9 := 38$
Layer 10:	$H_{10} := 10 \cdot ft$	$N_{10} := 37$

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name: Sebago Lake Crossing | Client: Standish
Project Number: 15107.00 | Project Manager: LT
Date: 07/08/09 | Computed by: km

Embank. slope a = 24.00(ft)
Embank. width b = 57.00(ft)
p load/unit area = 3000.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 40.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	3000.00
5.00	2980.25
10.00	2875.28
15.00	2691.82
20.00	2476.50
25.00	2262.82
30.00	2065.83
35.00	1890.04
40.00	1735.27
45.00	1599.63
50.00	1480.72
55.00	1376.22
60.00	1284.04
65.00	1202.37
70.00	1129.69
75.00	1064.71
80.00	1006.36
85.00	953.72
90.00	906.05
95.00	862.72
100.00	823.17

at 5.0 ft $\Delta\sigma_{z1} := 2980.25 \cdot psf$

at 15.0 ft $\Delta\sigma_{z2} := 2691.82 \cdot psf$

at 25.0 ft $\Delta\sigma_{z3} := 2262.82 \cdot psf$

at 35.0 ft $\Delta\sigma_{z4} := 1890.04 \cdot psf$

at 45.0 ft $\Delta\sigma_{z5} := 1599.63 \cdot psf$

at 55.0 ft $\Delta\sigma_{z6} := 1376.22 \cdot psf$

at 65.0 ft $\Delta\sigma_{z7} := 1202.37 \cdot psf$

at 75.0 ft $\Delta\sigma_{z8} := 1064.71 \cdot psf$

at 85.0 ft $\Delta\sigma_{z9} := 953.72 \cdot psf$

at 95.0 ft $\Delta\sigma_{z10} := 862.72 \cdot psf$

Height of Layer 1: $H_1 := 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$ $tsf := \frac{tonf}{ft^2}$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress: $\sigma_{1o} := \frac{H_1}{2} \cdot \gamma_{sagr}$ $\sigma_{1o} = 625 \cdot psf$ $\sigma_{1o} = 0.313 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_1 = 22$

At P_o = 0.313 tsf $C_{N1} := 1.5$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60\_1} := C_{N1} \cdot N_1$
From Eq 3-3 pg 3-36

$N1_{60\_1} = 33$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C1 := 96$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1} = 2980.25 \cdot psf$

Height of Layer 2: $H_2 := 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress: $\sigma_{2o} := H_1 \cdot \gamma_{sagr} + \frac{H_2}{2} \cdot \gamma_{sagr}$ $\sigma_{2o} = 0.938 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_2 = 15$

At P_o = 0.938 tsf $C_{N2} := 1.0$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_2} := C_{N2} \cdot N_2 = 15$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C2 := 58$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2} = 2691.82 \cdot psf$

Height of Layer 3: $H_3 := 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{3o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + \frac{H_3}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$

$\sigma_{3o} = 1.251 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_3 = 19$

At P_o = 1.251 tsf $C_{N3} := 0.95$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_3} := C_{N3} \cdot N_3 = 18$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C3 := 65$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z3} = 2262.82 \cdot psf$

Height of Layer 4: $H_4 := 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress: $\sigma_{4o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + \frac{H_4}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$ $\sigma_{4o} = 1.564 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_4 = 43$

At P_o = 1.564 tsf $C_{N4} := 0.88$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_4} := C_{N4} \cdot N_4 = 38$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C4 := 107$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z4} = 1890.04 \cdot psf$

Height of Layer 5: $H_5 = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:
$$\sigma_{5o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + \frac{H_5}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$
$\sigma_{5o} = 1.877 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_5 = 38$

At P_o = 1.877 tsf $C_{N5} := 0.79$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_5} := C_{N5} \cdot N_5 = 30$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C5 := 78$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z5} = 1599.63 \cdot psf$

Height of Layer 6: $H_6 = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:
$$\sigma_{6o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + H_5 + \frac{H_6}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$
$\sigma_{6o} = 2.19 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_6 = 20$

At P_o = 2.19 tsf $C_{N6} := 0.75$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_6} := C_{N6} \cdot N_6 = 15$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C6 := 58$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z6} = 1376.22 \cdot psf$

Height of Layer 7: $H_7 = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:
$$\sigma_{7o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + H_5 + H_6 + \frac{H_7}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$
$\sigma_{7o} = 2.502 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_7 = 20$

At P_o = 2.502 tsf $C_{N7} := 0.50$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_7} := C_{N7} \cdot N_7 = 10$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C7 := 47$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z7} = 1202.37 \cdot psf$

Height of Layer 8: $H_8 = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{8o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + H_5 + H_6 + H_7 + \frac{H_8}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$

Corrected SPT N_{60}-value (bpf) $N_8 = 21$ $\sigma_{8o} = 2.816 \cdot tsf$ at mid-point

At P_o = 2.816 tsf $C_{N8} := 0.65$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_8} := C_{N8} \cdot N_8 = 14$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C8 := 57$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z8} = 1064.71 \cdot psf$

Height of Layer 9: $H_9 = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{9o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + H_5 + H_6 + H_7 + H_8 + \frac{H_9}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$

Corrected SPT N_{60}-value (bpf) $N_9 = 38$ $\sigma_{9o} = 3.129 \cdot tsf$

At P_o = 3.129 tsf $C_{N9} := 0.62$ From Figure 3-24 pg 3-57 at mid-point

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_9} := C_{N9} \cdot N_9 = 24$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C9 := 77$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z9} = 953.72 \cdot psf$

Height of Layer 10: $H_{10} = 10 \cdot ft$ Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress:

$$\sigma_{10o} := \left(H_1 + \frac{H_2}{2}\right) \cdot \gamma_{sagr} + \left(\frac{H_2}{2} + H_3 + H_4 + H_5 + H_6 + H_7 + H_8 + H_9 + \frac{H_{10}}{2}\right) \cdot \left(\gamma_{sagr} - \gamma_w\right)$$

Corrected SPT N_{60}-value (bpf) $N_{10} = 37$ $\sigma_{10o} = 3.442 \cdot tsf$ at mid-point

At P_o = 3.442 tsf $C_{N10} := 0.60$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36 $N1_{60\_10} := C_{N10} \cdot N_{10} = 22$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C10 := 73$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z10} = 862.72 \cdot psf$

Settlement at each layer of sand and gravel:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{z1}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.95 \cdot in$$

$$\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{z2}}{\sigma_{2o}}\right) \qquad \Delta H_2 = 0.8 \cdot in$$

$$\Delta H_3 := H_3 \cdot \frac{1}{C3} \cdot \log\left(\frac{\sigma_{3o} + \Delta\sigma_{z3}}{\sigma_{3o}}\right) \qquad \Delta H_3 = 0.52 \cdot in$$

$$\Delta H_4 := H_4 \cdot \frac{1}{C4} \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{z4}}{\sigma_{4o}}\right) \qquad \Delta H_4 = 0.23 \cdot in$$

$$\Delta H_5 := H_5 \cdot \frac{1}{C5} \cdot \log\left(\frac{\sigma_{5o} + \Delta\sigma_{z5}}{\sigma_{5o}}\right) \qquad \Delta H_5 = 0.24 \cdot in$$

$$\Delta H_6 := H_6 \cdot \frac{1}{C6} \cdot \log\left(\frac{\sigma_{6o} + \Delta\sigma_{z6}}{\sigma_{6o}}\right) \qquad \Delta H_6 = 0.25 \cdot in$$

$$\Delta H_7 := H_7 \cdot \frac{1}{C7} \cdot \log\left(\frac{\sigma_{7o} + \Delta\sigma_{z7}}{\sigma_{7o}}\right) \qquad \Delta H_7 = 0.24 \cdot in$$

$$\Delta H_8 := H_8 \cdot \frac{1}{C8} \cdot \log\left(\frac{\sigma_{8o} + \Delta\sigma_{z8}}{\sigma_{8o}}\right) \qquad \Delta H_8 = 0.16 \cdot in$$

$$\Delta H_9 := H_9 \cdot \frac{1}{C9} \cdot \log\left(\frac{\sigma_{9o} + \Delta\sigma_{z9}}{\sigma_{9o}}\right) \qquad \Delta H_9 = 0.1 \cdot in$$

$$\Delta H_{10} := H_{10} \cdot \frac{1}{C10} \cdot \log\left(\frac{\sigma_{10o} + \Delta\sigma_{z10}}{\sigma_{10o}}\right) \qquad \Delta H_{10} = 0.08 \cdot in$$

Total settlement =

$$\Delta H_{A1} := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 + \Delta H_7 + \Delta H_8 + \Delta H_9 + \Delta H_{10}$$

$\Delta H_{A1} = 3.558 \cdot in$ At Abutment No. 1

Frost Protection:

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

From the Design Freezing Index Map:
Standish, Maine
DFI = 1330 degree-days

Soils are coarse grained. Assume a water content = ~20%

From MaineDOT BDG Table 5-1:
Depth of frost penetration = 78.0 inches

$\text{Frost\_depth} := 78.0\text{in} \qquad \text{Frost\_depth} = 6.5\cdot\text{ft}$

Method 2 - Check Frost Depth using ModBerg Software

Closest Station is Portland

ModBerg Results

Project Location: Portland Wsfo Airport, Maine

Air Design Freezing Index = 1195 F-days
N-Factor = 0.80
Surface Design Freezing Index = 956 F-days
Mean Annual Temperature = 45.5 deg F
Design Length of Freezing Season = 118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	59.8	10.0	120.0	26	32	1.7	1.5	1,728

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 = 4.98 ft = 59.8 in.

Use Modberg Calculated Frost Depth = 5.0 feet for design

Seismic:

15107.00 Standish Sebago Lake Road Crossing Bridge
Date and Time: 6/25/2009 3:39:56 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04084
Zip Code Latitude = 43.787000
Zip Code Longitude = -070.547600
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.095	PGA - Site Class B
0.2	0.186	Ss - Site Class B
1.0	0.047	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04084
Zip Code Latitude = 43.787000
Zip Code Longitude = -070.547600
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 (sec)	Sa (g)	
0.0	0.152	As - Site Class D
0.2	0.298	SDs - Site Class D
1.0	0.112	SD1 - Site Class D

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

Purpose - The ground motion parameters obtained in this analysis are for use with the design procedures described in AASHTO Guidelines for the Seismic Design of Highway Bridges (2007) The user may calculate seismic design parameters and response spectra (both for period and displacement), for Site Class A through E.

Description - This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. In most cases the user may perform an analysis for a site by specifying location by either latitude-longitude (recommended) or zip code. However, locations in Puerto and the Virgin Islands may only be specified by latitude-longitude.

Ground motion maps are included in PDF format. These maps may be opened using a map viewer that is part of the software package.

Data - The 2007 AASHTO maps are based on 5% in 50 year probabilistic data from the U.S. Geological Survey data sets for the following regions: 48 conterminous states (2002), Alaska (2006), Hawaii (1998), Puerto Rico and the Virgin Islands (2003). These were the most recent data available at the time of preparation of the AASHTO maps. The AASHTO maps are labelled with a probability of exceedance of 7% in 75 years which is approximately equal to the 5% in 50 year data.

Disclaimer - Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.

Appendix D

Special Provision

SPECIAL PROVISION 636
MECHANICALLY STABILIZED EARTH RETAINING WALL

636.01 Description. The work under this item shall consist of design, fabrication, furnishing, transportation, and erection of Mechanically Stabilized Earth (MSE) retaining wall system of the required type, including miscellaneous items necessary for a complete installation.

The MSE retaining walls shall consist of reinforcing strips or reinforcing mesh earth wall systems utilizing architectural precast concrete facing panels supported on cast-in-place concrete leveling pads. All reinforcing strips or mesh material shall consist of galvanized steel. The wall structures shall be dimensioned to achieve the design criteria shown on the plans and specified herein.

The MSE retaining walls shall be constructed in accordance with these specifications and in conformity with the lines, grades, design criteria, and dimensions shown on the plans or established by the Geotechnical Engineer.

636.02 Quality Assurance. The MSE retaining wall system shall be one of the approved wall systems noted in the Contract Documents.

All necessary materials, except backfill and cast in-place concrete shall be obtained from the approved system designer.

Mechanically Stabilized Earth (MSE) retaining walls shall be designed and constructed as specified herein. The design shall be subject to review and acceptance by the Geotechnical Engineer. The acceptability of a MSE retaining wall design shall be at the sole discretion of the Geotechnical Engineer. Any additional design, construction or other costs arising as a result of rejection of a retaining wall design by the Geotechnical Engineer shall be borne by the Contractor.

Precast facing panels shall be manufactured in a concrete products plant with approved facilities. Before proceeding with production, precast sample units shall be provided for the Resident’s acceptance. These samples shall be kept at the plant to be used for comparison purposes during production.

All calculations and Shop Drawings shall be signed and sealed by a licensed Professional Engineer registered in accordance with the laws of the State of Maine and specializing in geotechnical construction.

The Contractor installing the MSE retaining walls shall have demonstrated experience constructing MSE walls and shall use personnel having demonstrated experience in the installation procedures recommended by the manufacturer and as specified herein.

All MSE walls shall be built in accordance with the plans and accepted shop drawings for the proposed wall systems.

A qualified representative from the wall design-supplier shall be present during construction of the MSE walls. The services of the qualified representative shall be at no additional cost to the project. The qualified experienced technical representative will advise the Contractor and the Resident concerning proper installation procedures.

The vendor's representative shall specify the required back-batter so that the final position of the wall is vertical. Furthermore, footing berms shall be placed in front of the first three (3) levels of panels erected, to maintain verticality.

636.03 Design Requirements. The MSE retaining walls shall be designed to provide the grade separation shown on the plans with a service life of not less than 100 years.

The MSE wall system shall be designed in accordance with:

1. The manufacturer's requirements
2. The Contract Plans
3. The requirements specified herein
4. AASHTO LRFD Bridge Design Specifications, current edition
5. AASHTO LRFD Bridge Construction Specifications, current edition
6. FHWA-NHI-00-043, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, March 2001
7. FHWA-NHI-00-044, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, September 2000

Where conflicting requirements occur, the more stringent requirements shall govern.

The MSE wall design shall follow the general dimensions of the wall envelope shown on the plans. Base of footing elevation shall be as shown on the plans, or may be lower. All wall elements shall be within the right-of-way limits shown on the plans. The panels shall be placed so as not to interfere with drainage or other utilities, or other potential obstructions.

All appurtenances behind in front of, under, mounted upon, or passing through the wall such as drainage structures, utilities, fences, concrete parapet wall or other appurtenances shown on the plans shall be accounted for in the stability design of the wall.

Facing panels shall have tongue and groove, ship lap or similar approved connections along all joints, both vertical and horizontal. Where foundation conditions indicate large differential settlements, vertical full-height slip joints shall be provided. The shape of the panels shall be such that adjacent panels will have continuous, vertical joints, or as noted on the plans.

MSE facing panels shall be installed on cast-in-place concrete leveling pads. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be 5.0 feet as measured to the top of the leveling pad, or as shown on the plans, whichever is greater. The top of the face panels shall be at or above the top of the panel elevation shown on the plans. Where coping or barrier is used, the wall face shall extend up into the coping or barrier a minimum of 2 inches.

The MSE walls shall be dimensioned so that the factored bearing pressure resistance of the foundation soils, as noted on the plans, is not exceeded. Requirements for over excavation of native foundation soils and replacement with compacted structural fill are detailed on the plans.

The design by the wall system supplier shall consider the stability of the wall as outlined below and in the Contract Documents:

(a) Failure Plane. The theoretical failure plane within the reinforced soil mass shall be determined per LRFD Section 11 and be analyzed so that the soil stabilizing components extend sufficiently beyond the failure plane within the reinforced soil mass to stabilize the material. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic, and seismic loads shall be accounted for in the design.

(b) External Stability - Load and Resistance Factors. Loads and load combinations selected for design shall be consistent with AASHTO LRFD. Application of load factors shall be taken as specified in AASHTO LRFD. Sliding resistance factors and bearing resistance factors shall be consistent with LRFD Section 10. Overturning provisions of LRFD Section 11 shall apply.

MSE walls shall be designed to resist failure by instability of temporary construction slope. Passive pressure in front of the wall mass shall be assumed to be zero for design purposes. The factored applied bearing pressures under the MSE mass for each reinforced length shall be clearly indicated on the design drawing.

(c) Internal Stability - Load and Resistance Factors. Evaluation of reinforcement pullout, reinforcement rupture and panel connection pullout or rupture shall be consistent with LRFD Section 11. Loads, load combinations and load factors shall be as specified in LRFD Article 11. Resistance factors for internal design shall be consistent with LRFD Article 11. Maximum reinforcement loads shall be calculated using the Simplified Method approach. Calculations for factored stresses and resistances shall be based upon assumed conditions at the end of the design life. The design life of steel soil reinforcements shall comply with LRFD Section 11.

(d) Backfill and Foundation Soils Parameters. The friction angle of the select backfill used in the reinforced fill zone for the internal stability design of the wall shall be assumed to be 34° unless noted otherwise. The friction angle of the foundation soils and random backfill shall be assumed to be 30° unless otherwise shown on the plans.

(e) Reinforcement Length. The soil reinforcement shall be the same length from the bottom to the top of each wall section. The reinforcement length defining the width of the entire reinforced soil mass may vary with wall height. The minimum length of the soil reinforcement shall be the greater of 22 feet or 0.6(H+d)+6.5 feet where H is the wall height as measured from the leveling pad and d id the height of soil above the wall, or 70 percent of H1 for walls with a sloped surcharge or walls supporting an abutment. The mechanical wall height, H or H1, shall be the vertical difference between the top of the leveling footing and the elevation at which the failure surface, as described above, intercepts the ground surface supported by the wall.

(f) Steel Reinforcement. For steel reinforcements, all structural connections, tie strips and loop inserts, the following galvanization and carbon steel loss rates shall be assumed:

	Mil./year/side
Zinc galvanizing (first 2 years)	0.58
Zinc galvanizing (subsequent years to depletion):	0.16
Carbon Steel (after zinc depletion to 100 years):	0.47

Calculations for factored stresses and resistances in steel reinforcements and connections, including tie-strips and loop inserts, shall be based upon assumed conditions at the end of the design life. (or: The nominal long-term design strength in steel reinforcements and connections, including tie-strips and loop inserts shall be determined at the end of the service life.) The applied factored reinforcement loads shall be calculated in accordance with LRFD Section, and shall be checked against the nominal tensile strength multiplied by a resistance factor per LRFD Table 11.5.6-1. Transverse and longitudinal grid members shall be sized in accordance with ASTM A 185.

When the expected differential settlement normal to the wall exceeds 3 in, the lower level reinforcement facing connections shall be designed to accommodate the increased tensile forces due to settlement.

(g) Facing Panel Requirements.

1. Facing panels shall be designed to resist compaction stresses that occur during wall erection.

2. The minimum thickness for concrete panels in the zone of embedded connections shall be 5.5 inches and 3.5 inches elsewhere. The minimum concrete cover shall be 1.5 inches. Facing panels shall meet the design requirements of LRFD 11.10.2.3.

3. The wall facing shall be designed to accommodate differential settlements of 1/100 foot.

4. The minimum spacing between adjacent panels shall be ¾ inches in order to accommodate differential settlements without impairing the appearance of the facing or compromising the structural integrity of the individual panels. Joints between panels shall be no more than 0.75 inch. Joint between panels shall have a ship lap configuration or tongue and groove connection. There shall be no openings through the wall facing, except for utilities to pass through the wall. Slip joints to accommodate differential settlement shall be included where shown on the plans.

5. Where wall or wall sections intersect with an angle of 130º or less, a special vertical corner element panel shall be used. The corner element panel shall cover the joint of the panels that abut the corner and allow for independent movement of the abutting panels. Corner elements shall not be formed by connecting standard facing panels that abut the acute corner.

636.04 Materials. The Contractor shall be responsible for the purchase or manufacture of the precast concrete facing panels, reinforcing mesh or strips, panel/reinforcement connections, bearing pads, joint filler, and all other necessary components. The Contractor shall furnish to the Resident the appropriate Certificates of Compliance certifying that the applicable wall materials meet the requirements of the project specifications. All materials used in the construction of the MSE retaining walls shall meet the requirements specified in the following subsections of the Maine Standard Specifications and as specified herein.

Materials not conforming to this section of the specifications, or from sources not listed in the contract documents, shall not be used without written consent from the Resident.

636.041 Reinforced Concrete Facing Panels. Reinforced concrete facing panels shall meet the requirements specified in the following subsections:

Structural Precast Concrete Units	712.061
Drainage Geotextile	722.02

636.042 Precast Panel Tolerances and Surface Finish. Concrete surface for the front face shall have a smooth steel formed finish, or as noted on the plans. The rear face shall have an unformed surface finish. The rear face of the panel shall be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of ¼ inch. All uncoated steel projecting from the panel unit shall be galvanized in accordance with ASTM A 123/A 123M (AASHTO M 111) with a minimum coating thickness of 2 oz/ft^2.

Precast panel tolerances shall comply with the following; units that do not meet the listed tolerances will be rejected.

1. Panel dimensions (edge to edge of concrete) within ±3/16 inch.
2. Panel thickness: ± ¼ inch.
3. Squareness. The length difference between the two diagonals shall not exceed ½ inch.
4. Distance between the centerline of dowel and dowel sleeve, and to centerline of reinforcing steel shall be ± 1/8 inch.
5. Face of panel to centerline of dowel and dowel sleeve, and to centerline of reinforcing steel shall be ± 1/8 inch.
6. Position of panel connection devices (Tie Strip) shall be ± 1 inch.
7. Location of Coil and loop Imbeds shall be ± 1/8 inch.
8. Warping of the exposed panel face shall not exceed 1/4 inch in 5 feet.
9. Surface defects on smooth-formed surfaces measured over a length of 5 feet shall not exceed 1/8 inch. Surface defects on textured-finished surfaces measured over a length of 5 feet shall not exceed 5/16 inch.

636.043 Reinforcing. All reinforcing, tie strips, and attachment devices shall be carefully inspected to insure they are true to size and free from defects that may impair their strength and durability.

A. Reinforcing Mesh shall be shop fabricated from cold drawn steel wire conforming to the requirements of AASHTO M 32 (ASTM A 82-97) yield strength minimum of 65 ksi and shall be welded into the finished mesh fabric in accordance with AASHTO M 55 (ASTM A 185). Galvanizing shall be in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum coating thickness shall be 2 oz/ft^2. Any damage done to the mesh galvanization prior to the installation shall be repaired in an acceptable manner and provide a minimum galvanized coating of 2 oz/ft^2.

B. Reinforcing Strips shall be fabricated from hot rolled bars to the required shape and dimensions. Their physical and mechanical properties shall conform to AASHTO M 223 (ASTM A 572/A572M) Grade 65, or approved equal. Reinforcing strips shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum galvanization coating thickness shall be 2 oz/ft^2. Any damage done to the mesh galvanization prior to the installation shall be repaired 2 oz/ft^2.

C. Tie strips shall be fabricated of hot rolled steel conforming to ASTM A 1011/A1011M, Grade 50 or equivalent. Tie strips shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum coating thickness shall be 2 oz/ft^2.

D. The tie strips and reinforcing strips shall be cut to lengths and tolerances shown on the submitted plans. Holes for bolts shall be punched in the locations shown.

636.044 Attachment Devices.

A. Steel clevis loop embeds shall be fabricated of cold drawn steel wire conforming to ASTM A 510, UNS G 10350 or AASHTO M 32 (ASTM A 82). Loop embeds shall be welded in accordance with AASHTO M 55 (ASTM A 185). Both shall have electrodeposited coatings of zinc applied in accordance with ASTM B 633.

B. Fasteners shall consist of hexagonal cap screw bolts and nuts, which are galvanized and conform to the requirements of AASHTO M 164 (ASTM A 325) or equivalent.

C. Connector pins and mat bars shall be fabricated from AASHTO M 183 (ASTM A 36/A36M) steel and welded to the soil reinforcement mats as shown on the plans. Galvanization shall conform to AASHTO M111 (ASTM A 123/A123M) with a minimum coating thickness of 2 oz/ft^2. Connector bars shall be fabricated of cold drawn steel wire conforming to the requirements of ASTM A 82 (AASHTO M 32) and galvanized in accordance with ASTM A 123/A123M.

D. Structural plate connectors and fasteners used for yokes to connect reinforcements to wall panels around pile or utility conflicts shall conform to the material requirements for reinforcing strips and fasteners in 636.042 (c).

636.045 Joint Materials. Joint material shall be installed to the dimensions and thicknesses specified below, or in accordance with the plans or approved shop drawings.

A. Provide flexible foam strips for filler for vertical joints between panels, and in horizontal joints where pads are used.

B. Provide in horizontal joints between panels either preformed EPDM rubber pads conforming to ASTM D 2000 for 4AA, or 812 rubbers or neoprene elastomeric pads having a Durometer Hardness of 55±5, or high density polyethylene pads with a minimum density of 0.946 g/cm3 in accordance with ASTM D 1505

636.046 Nonwoven Drainage Geotextile. Cover all joints between panels on the back side of the wall with a geotextile fabric meeting the minimum requirements of 722.02 Class 2. Slit film and multifilament woven and resin bonded woven geotextile fabrics are not allowed for this application. The minimum width of the fabric shall be 12 inches. Lap fabric at least 12 inches. where splices are required. Nonwoven Drainage Geotextile shall be bonded with an approved adhesive compound to the back face covering all joints between panels. Adhesives used to hold the geotextile filter fabric material to the rear of the facing panels prior to backfill placement shall be supplied by the wall supplier and approved by the Resident.

636.047 Concrete Leveling Pad. The cast-in-place leveling pad shall be constructed of Class B concrete conforming to the requirements of Section 502 - Structural Concrete. Leveling pad shall have minimum dimensions of 6 inches thickness and 12 inches width and be placed at the design elevation shown on the shop drawings within a 1/8 inch tolerance.

636.048 Backfill Materials. All backfill materials used in the MSE Walls volume shall conform to Gravel Borrow conforming to the requirements of Section 703.20, with and the following additional requirements:

A. The maximum aggregate size is limited to 4 inch (U.S Sieve Size - 102 mm).

B. Soundness. The material shall be substantially free of shale or other soft, poor durability particles. The materials shall have a magnesium sulfate soundness loss, as determined by AASHTO T104 (ASTM C 88), of less than 30 percent after four cycles.

C. Electrochemical Requirements. The backfill materials shall meet the following criteria:

Requirements		Test Methods
Resistivity	>3,000 ohm-centimeters	AASHTO T 288
pH between	Between 5 and 10, inclusive	AASHTO T 289
Chlorides	<100 parts per million	AASHTO T 291
Sulfates	<200 parts per million	AASHTO T 290
Organic Content	<1%	AASHTO T 267-86

D. The plasticity index (PI) as determined by AASHTO T90 shall not exceed 6.

E. The select backfill material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T236 (ASTM D3080-72), on the portion finer than the 2 mm [#10 sieve], compacted to 95 percent of AASHTO T99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than 3/4 inch. Before construction begins, the borrow material selected shall be subject to show conformance with this frictional requirement. Compliance with the test requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

636.05 Crushed Stone for Abutment Foundation. Aggregate for use in the foundation layer below the abutment shall be crushed stone conforming to the following gradation requirements:

Sieve Designation	Percent of Weight Passing Square Mesh Sieves
1 inch	100
¾ inch	90-100
½ inch	20-55
3/8 inch	0-15
No. 4	0-5

636.051 Impervious Membrane. An impervious geomembrane shall be installed near the top of the reinforced backfill to reduce the chance of water infiltrating into the reinforced backfill. The geomembrane shall be bonded to the inside face of the wall panels and extend perpendicularly from the wall face into the fill, while being parallel to the top of the wall. The membrane should be sloped to drain away from the facing and outlet beyond the reinforcing zone. The impervious geomembrane shall extend into the fill a distance of 1 foot beyond the MSE reinforcement. The geomembrane shall have a minimum thickness of 0.76 mm, 30 mil (0.03 inch, 1/32 inch)

The geomembrane shall have both sides textured with a rough finish to improve resistance against sliding. The texture shall be approved by the Resident before installation. The geomembrane shall be shown on the design drawings of the MSE submittal of the Contractor.

636.052 Acceptance of Material. The Contractor shall furnish to the Resident a Certificate of Compliance certifying that the above materials comply with the applicable contract specifications including the backfill material, in accordance with Section 700. A copy of all test results performed by the Contractor necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based on the Certificate of Compliance, accompanying test reports, and visual inspection by the Resident.

636.06 Submittals.

A. Design computations demonstrating compliance with the criteria specified herein and shown on the plans, shall be prepared, signed and stamped by a licensed Professional Engineer licensed in the State of Maine and specializing in geotechnical engineering. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties.

The design calculations shall include:

1. Statement of all assumptions made and copies of all references used in the calculations.

2. Analyses demonstrating compliance with all applicable earth, water, surcharges, seismic, or other loads, as specified herein and required by AASHTO LRFD.

3. Analyses or studies demonstrating durability and corrosion resistance of retaining wall systems for the proposed location and environment. The designer shall provide all corrosion protection devices necessary for the retaining wall to have a minimum service life of 100 years in the proposed location and environment.

B. A detailed resume of the wall designer listing similar projects with references, and demonstrating necessary experience to perform the MSE retaining wall design, including a brief description of each project that is similar in scope.

C. A detailed listing of MSE walls that the Contractor has constructed including a brief description of each project and a listing of personnel who will construct the walls demonstrating their experience in construction of MSE retaining walls. A reference shall be included for each project listed. As a minimum, the reference shall include an individual’s name, address and current phone number.

D. Manufacturer's product data for the MSE wall system, including material, manufacture and erection specifications, all specified erection equipment necessary, details of buried MSE wall elements, special details required of reinforcing layout around drainage structures and sign foundations, structures design properties, type of backfill and details for connections between facing panels.

E. Details of precast yard and concrete mix design.

F. Shop drawing showing the configuration and all details, dimensions, quantities and cross sections necessary to construct the MSE wall, including but not limited to the following:

1. A plan view of the wall, which shall include Contract limits, stations and offsets, and the face of wall line shown on the plans.
2. An elevation view of the wall which shall include the elevation at the top of the wall at all horizontal and vertical break points and at least every 50 feet along the face of the wall, all steps in the leveling pads, the designation as to the type of retaining wall system(s), and an indication of the final ground line and calculated factored bearing pressures. The face of wall shown on the plans shall be indicated.
3. A typical cross section or cross sections showing the elevation relationship between existing ground conditions and proposed grades, and the proposed wall configuration, including details for the proposed methods for connecting to existing conditions. The sections shall also indicate the location of the face of wall shown on the plans.
4. General notes pertaining to design criteria and wall construction.
5. A listing of material quantities for each wall.
6. Details of sleeves and pipes and other embedded items to be installed through the walls.
7. Clearly indicated details for construction of walls or reinforcing elements around drainage, foundations, utilities or any other potential obstructions.
8. Details of the architectural treatment of facing panels.
9. Drainage design detail and design scheme.
10. Location of utilities.
11. Sequence and schedule of construction, including overall construction schedule.
12. Methods of excavation and backfill.
13. Method of maintaining stability of excavated trenches.
14. Method of monitoring plumbness and deviation of wall.
15. Excavation support system, if any.
16. Any acceptance testing and frequency.
17. Details and location of all necessary construction and expansion joints along the wall.
18. Connection details at the interface of the wall and any adjacent proposed cast in place retaining wall or abutment structure.
19. Details of impermeable membrane connection to abutment in roadway runoff collection system.

636.07 Delivery, Storage and Handling.

A. Contractor shall check the material upon delivery to assure that the proper material has been received. A product certification should be provided with each shipment.

B. Material shall be stored above -20º F

C. Contractor shall prevent excessive mud, wet cement, epoxy and like substances which may affix themselves to the material from coming in contact with the material.

D. Material may be laid flat and stored outside for 30 days. For extended storage, material shall be stored in or beneath a trailer or covered with a colored tarpaulin to prevent long-term exposure.

636.08 Wall Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 - Excavation and Embankment, except as modified herein. Temporary excavation support as required shall be the responsibility of the contractor.

636.09 Foundation Preparation. The foundation for the structure shall be graded level for a width equal to the length of reinforcement elements plus 5 feet, or as shown on the plans. Prior to wall construction the foundation shall be compacted with at least 10 passes of a smooth wheel vibratory roller weighing at least 10,000 lbs. Any foundation soils found to be unsuitable or incapable of sustaining the required compaction shall be removed and replaced with Special Borrow Material. The foundation for the structure shall be approved by the Resident before erection is started.

A concrete leveling pad shall be constructed as indicated on the submitted plans. The leveling pad shall be cast to the design elevations as shown on the plans. Allowable elevation tolerances are +0.01 foot and -0.02 foot from the design elevations. Placement of wall panels may begin after 24 hours curing time of the concrete leveling pad.

636.10 Wall Erection. A field representative from the proprietary wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the project.

Precast concrete panels shall be placed so that their final position is vertical or battered as shown on the plans. The vendor representative shall specify the required back-batter so that the final position of the wall is vertical. Earth berms at the footing shall be placed to maintain the desired position of panels. For erection, panels are handled by means of lifting devices connected to the upper edge of the panel. Panels should be placed in successive horizontal lifts in the sequence shown on the approved shop drawings as backfill placement proceeds. As backfill material is placed behind the panels, the panels shall be maintained in position by means of temporary wedges or bracing according to the wall supplier’s recommendations.

Concrete facing vertical tolerances and horizontal alignment tolerances shall not exceed ¾ inch when measured with a 10 foot straightedge (¼ inch per yard). During construction, the maximum allowable offset in any panel joint shall be ¾ inch. The overall vertical tolerance of the wall (from top to bottom) shall not exceed ½ inch per 10 feet of wall height.

636.11 Backfill Placement. Backfill shall not be placed between November 1st and April 1st. Backfill placement shall closely follow erection of each course of panels. Backfill shall be placed and compacted in such a manner as to avoid any damage or disturbance of the wall materials or misalignment of the facing panels or reinforcing elements. Any wall materials which become damaged during backfill placement shall be removed and replaced at the Contractor's expense. Any misalignment or distortion of the wall facing panels due to placement of backfill outside the limits of this specification shall be corrected by the Contractor at his expense. Prior to the placement of the soil reinforcement, the backfill elevation after compaction shall be at the required elevation of the reinforcements. At each reinforcement level, the backfill shall be placed to the level of the connection. Backfill placement methods near the panels shall assure that no voids exist directly beneath the reinforcing element.

Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180, Method C or D (with oversize correction, as outlined in Note 7 of that test). If 30 percent or more of the backfill material is greater than 3/4 inch in size, AASHTO T180 is not applicable, and the acceptance criterion for control of compaction shall be either a minimum of 70 percent of the relative density of the material as determined by ASTM D4253 and D4254, or a method of compaction consisting of at least 4 (four) passes by a heavy roller.

Where spread footings support bridge or other structural loads, the top 5 feet below the bottom of footing elevation shall be compacted to 98 percent of the maximum density as determined by AASHTO T180, Method C or D (with oversize correction, as outlined in Note 7 of that test).

The moisture content (determined in accordance with AASHTO T180, Method C or D) of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer. Backfill materials shall have be placed at a moisture content not more than 2 percentage points less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniformly acceptable throughout the entire lift.

At each reinforcing level, backfill shall be leveled before placing and bolting the reinforcing. The maximum lift thickness after compaction shall not exceed 12 inches. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

Heavy compaction equipment shall not be used to compact backfill within 3 feet of the wall face. Compaction within 3 feet of the back face of the wall shall be achieved by at least three (3) passes of lightweight mechanical tamper, lightweight roller, or vibratory system. The specified lift thickness shall be adjusted as warranted by the type of compaction equipment actually used. No vehicular equipment shall be operated within 3 feet of the panels.

The frequency of sampling of the backfill material necessary to assure gradation control throughout construction shall be as directed by the Resident.

At the end of each day's operation, the Contractor shall slope the last level of the backfill away from the wall facing to rapidly direct runoff away from the wall face. In addition, the Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

636.12 Reinforcement Placement. Prior to placing the first layer of reinforcements (strips, mats or grids), backfill shall be placed and compacted in accordance with Subsection 636.11, Backfill Placement.

Bending of reinforcements in the horizontal plane resulting in a permanent deformation in their alignment shall not be allowed. Gradual bending in the vertical direction that does not result in permanent deformations is allowable.

Cutting of longitudinal or transverse reinforcement bars to avoid conflicts with utility obstructions or piles will not be allowed. A structural connection (yokes) from the wall panel to the reinforcement shall be used whenever it is necessary to avoid cutting or excessive skewing of reinforcement due to pile or utility conflicts.

Soil reinforcements shall be placed normal to the face of the wall, unless otherwise shown on the plans or directed by the Resident. If skewing of the soil reinforcements is required due to obstructions in the reinforced fill, rotatable bolted connections shall be used and the maximum skew angle shall not exceed 15º from the normal position except in the case of acute corner where redundant reinforcements are used. The tensile capacity of splayed reinforcement shall be reduced by the cosine of the splay angle.

636.13 Method of Measurement. Mechanically Stabilized Earth Retaining Wall will be measured by the square foot of face area computed using the plan dimensions. No adjustment in the pay quantity will be made if the computed quantity, based on the working drawings, varies from the plan quantity.

Vertical dimension limits will be from the top of leveling pad to the top of the wall facing units, as shown on the plans. The horizontal dimension limits will be from the edges of the facing units at each end of a wall, as shown on the plans. No field measurements will be made unless the Resident specifies, in writing, a change to the limits indicated on the plans.

The wall surface area, as shown on the plans, includes the surface area of nominal panel joint openings and wall penetrations such as pipes and other utilities.

636.14 Basis of Payment. The accepted quantity of Mechanically Stabilized Earth Retaining Wall will be paid for at the contract unit price per square foot. Payment shall be full compensation for design, fabrication and erection of MSE retaining walls, furnishing all labor, equipment and materials including concrete face panels, fasteners, reinforcing mesh, reinforcing strips, tie strips, hardware, joint fillers, coping, woven drainage geotextile, impervious membrane, select granular backfill and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately but will be considered incidental to the Mechanically Stabilized Earth Retaining Wall.

Excavation, including extra excavation due to unsuitable foundation material, will be measured and paid for under Item 203.20 - Common Excavation. Foundation material and select backfill material will be considered incidental to the Mechanically Stabilized Earth Retaining Walls.

The unit price for Mechanically Stabilized Earth Wall shall include costs for:

1. All design work, preparation of written submittals and plans, revision of submittals, sample submittals and any other necessary preliminary work prior to and after acceptance of the retaining wall by the Resident.

2. All materials, including transportation, for the MSE walls, including facing panels, MSE reinforcing elements, attachment devices, fasteners, bearing blocks and shims, joint materials, copings, vertical corner elements, concrete masonry, reinforcing steel, crushed stone, select backfill and incidentals.

3. All labor and equipment required to excavate and prepare the wall foundation, form and cast the leveling pad, erect the MSE wall to the lines and grades shown on the plans, place and compact backfill, place and compact the drainage layer, and construct any other items necessary to complete the MSE wall.

4. All temporary sheeting, temporary excavation, and temporary dewatering necessary to perform the other work in this section.

There will be no allowance for excavating and backfilling for the Mechanically Stabilized Earth Retaining Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation.

Payment will be made under:

Pay Item		Pay Unit
636.30	Mechanically Stabilized Earth Retaining Wall	Square foot