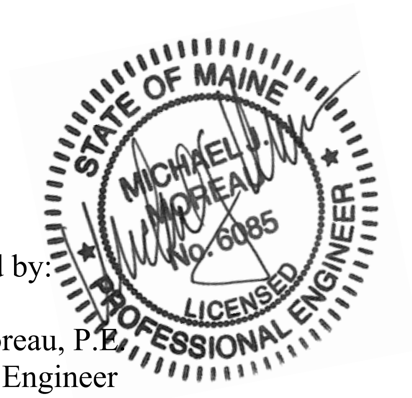
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

B&A OVERHEAD BRIDGE
LAGRANGE, MAINE



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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for replacement of the B&A Overhead Bridge which carries Route 16 over an abandoned railroad bed converted to a recreational trail in Lagrange, Maine. The replacement bridge will be a buried structure consisting of a 32-foot span Fiber Reinforced Polymer (FRP) arch on spread footings cast on compacted fill or glacial stream soils. The arch headwalls shall consist of mechanically stabilized earth (MSE) wall structures. The replacement bridge design will conform to the requirements of the Maine Department of Transportation (MaineDOT) Bridge Design Guide (BDG) and the AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012, (herein referred to as LRFD). The design and construction recommendations below are discussed in greater detail in Section 7.0 Geotechnical Design Recommendations.

Arch Stem Wall and Foundation Design – Arch stem wall and stem wall foundations shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure as appropriate. Stem walls and stem wall foundations shall be designed for all relevant service and strength limit states in accordance with LRFD.

The design of project stem walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of spread footings on soil. A maximum frictional coefficient of 0.55 at the soil-concrete interface should be assumed. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed the middle two-thirds (2/3) of the footing base width.

Service limit load conditions may control arch foundation design of the B&A Overhead Bridge. A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and overall stability. Global stability conditions are satisfied since the factored Service I bearing resistances are greater than the proposed factored bearing pressures provided by TY Lin and the proposed final build-out embankment heights will be less than the original height of the esker. The foundations shall also be constructed a minimum of 6.5 feet below exterior finished grade providing additional passive resistance.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31 calculated using Rankine Theory for stem wall design. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Calculation of passive earth pressures for resisting lateral thrust forces from the arch should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating small lateral arch stem wall foundation movements. If the ratio of lateral foundation movement to stem wall height, (y/H), exceeds 0.005, then a Coulomb passive earth pressure coefficient, K_p, of 6.89 should be used. The designer shall use a resistance factor for passive earth pressures (φ_{ep}) of 0.50 for earth pressure mobilized to resist lateral sliding forces. For designing the arch foundation reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

The strength and service limit state factored bearing resistances for arch foundation spread footings on compacted granular fill or native glacial stream deposits shall depend on the footing size as presented in the graph in Section 7.2 of this report. The designer shall use the service limit factored bearing resistance for preliminary footing sizing to control settlement. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

The arch stem wall design shall include a drainage system to intercept any groundwater in the wall backfill. The contractor shall construct weep holes 10 feet center-to-center approximately 6 inches above finish grade through the arch stem walls to facilitate drainage. Arch stem wall foundations shall not be constructed directly on old bridge pier foundations. The contractor shall remove a sufficient depth of the old foundation to allow placement of at least 18 inches of compacted soil between the former intermediate pier foundations and the new arch stem wall footing base.

Mechanically Stabilized Earth (MSE) Arch Headwalls – Arch headwalls will consist of mechanically stabilized earth walls. Design and construction of the walls shall be in accordance with LRFD and MaineDOT Special Provision 636, Mechanically Stabilized Earth Retaining Wall (See Appendix D, Special Provisions). These walls shall be a design-build item designed by a Professional Engineer licensed in Maine and retained by the contractor.

MSE walls shall be investigated at the strength limit state for bearing capacity failure, lateral sliding, excessive loss of base contact, pullout of soil reinforcements and structural failure. A sliding resistance factor, φ_τ, of 1.0 shall be applied to the nominal sliding resistance of soil-on-soil beneath the MSE mass. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.58 (tan 30 degrees) at the foundation soil to soil in-fill interfaces. For the lowest MSE level, the eccentricity of factored loads at the strength limit state shall not exceed the middle two-thirds (2/3) of the reinforced soil base width. The MSE wall designer may assume Soil Type 4 backfill soil material for the MSE wall volume backfill with the following properties: ϕ = 32 degrees, γ = 125 pcf. Factored bearing pressures should be computed using a uniform base distribution over the effective width. Calculated factored bearing resistance values for MSE reinforced soil volumes founded on compacted granular soils are provided in Section 7.3 of this report.

The wall designer shall use a factored bearing resistance of 6 ksf to limit settlement to 1-inch when analyzing the service limit state, and for preliminary reinforcement length sizing. The MSE mass shall be assessed at the service limit state using a resistance factor of ϕ = 1.0 for settlement and horizontal movement, and the overall stability shall be assessed at the Service I Load Combination with a resistance factor, ϕ, of 0.65. We recommend that the designer use a Coulomb active earth pressure coefficient, K_a, of 0.31 to evaluate the external stability of the wall. The wall designer shall estimate the traffic surcharge as a uniform horizontal earth pressure due to 2.0 feet of soil.

The reinforcing length shall be uniform throughout the entire height of the wall. A concrete leveling pad with a width no less than 2.0 feet shall be provided to support the MSE wall face elements. The leveling pad for the wall panels shall be founded a minimum of 6.5 feet below finished exterior grade for frost protection. An impervious geomembrane consisting of low-permeability, 2-sided, texture HDPE with a minimum thickness of 30 mils shall be installed near the top of the reinforced soil zone to minimize water infiltration. MSE wall design must also consider guardrail placement along the highway to avoid interference with reinforcement and membrane configuration.

Settlement – We have estimated that the total settlement of a prepared subgrade consisting of compacted fill or native glacial stream deposits will be on the order of ½-inch for conventional arch spread footings where service limit state loads correspond to those in Section 7.2 of this report. We estimate that differential settlement will be on the order of ½-inch or less. In all cases above, this settlement is acceptable and will occur during construction. We anticipate that post-construction settlement will be negligible.

We estimate that settlement as a result of embankment construction comprising approximately 23 feet of granular fill over natural soils will be on the order of 1-inch or less and will occur during construction. We anticipate that post-construction settlement will be negligible.

We estimate settlement beneath MSE wall bases will be on the order of 1-inch or less. This settlement is acceptable and will occur during construction. We anticipate that post-construction settlement will be negligible.

Frost Protection – Foundations placed on granular soils shall be founded a minimum of 6.5 feet below finish exterior grade for frost protection. The contractor must remove existing concrete abutment structures 6.5 feet below the proposed highway finish grade to help minimize frost heaves in the completed new highway at the former abutment locations.

Seismic Design Considerations – Seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine.

Construction Considerations –

<u>Excavation</u>

- Remove the old intermediate pier columns and their foundations a minimum of 18 inches below the proposed arch foundation elevation. **Construction of new arch spread footing foundations directly on old concrete footings will not be allowed**. This may require staged construction methods, earth support systems, or shoring and bracing.
- Construction of new arch headwall foundations will require soil excavation. Earth support systems may be required.
- The contractor must remove the existing bridge abutments down to a level at least 6.5 feet below the proposed highway finished grade to prevent frost heaves in the completed highway at the old abutment locations.

<u>Foundation Subgrade Preparation</u>

- Sample, test and compact the arch footing subgrade soil to 95% of the Modified Proctor maximum dry density. Place at least 6 inches of ¾-inch crushed stone over prepared subgrade and compact this material with at least 4 passes of a walk-behind vibratory compactor with minimum 200 lb static weight.

<u>Dewatering</u>

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Cofferdams, temporary ditches, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert surface water or groundwater if significant seepage is encountered during excavation.
- Protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and/or rutting occur, remove the disturbed soil materials and replace it with compacted granular borrow

<u>Reuse of Excavated Soil</u>

- <u>Do not</u> use excavated existing subbase aggregate for pavement structure construction or to re-base shoulders or for arch and headwall backfill soil. Excavated subbase sand and gravel may be used as common borrow below subgrade elevation in fill embankment areas in accordance with MaineDOT Standard Specification Sections 203 and 703.

<u>Embankment Fill Areas</u>

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

<u>Erosion Control</u>

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

1.0 INTRODUCTION

MaineDOT plans to replace the B&A Overhead Bridge which carries Route 16 over an abandoned Montreal, Maine and Atlantic Railway rail bed converted to a recreational trail in the Town of Lagrange, Penobscot County, Maine. We show the project location on Sheet 1, Location Map found at the end of this report. We conducted subsurface investigations at the bridge site to develop geotechnical recommendations for the structure replacement. This report summarizes our findings, discusses our evaluation of the subsurface conditions and presents our geotechnical recommendations for design and construction of the proposed bridge foundations.

The existing simply supported, four-span, steel I-beam bridge was originally built in 1944. The abutments and piers were founded on spread footings formed and cast over native glacial stream sediments. The recreational trail is skewed to the bridge at approximately 32.5 degrees. The bridge had a partial deck replacement in 1994, but is currently in poor condition and in need of complete replacement. The bridge had a sufficiency rating of 49.7 in 2011.

The MaineDOT Bridge Program has selected the B&A Overhead Bridge site to construct a concrete-filled tubular FRP arch structure. The composite tubular arch structure was developed by the University of Maine’s Advanced Engineered Wood Composites (AEWC) Center in Orono, Maine. The carbon fiber arch tubes are inflated and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete.

The proposed structure will be a single-span arch with a span length of approximately 32 feet and will be founded on reinforced concrete stem walls supported on spread footings on soil. The horizontal alignment will be shifted to the west approximately 10 feet at the bridge location and brought back to existing alignment approximately 350 feet north and south of the bridge. The vertical alignment will be lowered one to two feet. Finish grade of the proposed new alignment of Route 16 near Station 35+00 will require approximately 23 feet of fill at the proposed bridge and lesser amounts up and down station.

2.0 GEOLOGIC SETTING

Regional surficial geology maps show that the bridge site is situated on an esker where glacial-stream sediments predominate with nearby glacial-marine soil unit contacts. The existing B&A Overhead Bridge foundations and approaches are constructed on the backslopes and base of a large open cut through the native glaciated sediments. The highway is constructed on the top of the large esker formed by these glacial-stream deposits. The open cut was excavated through the esker for the construction of a now abandoned railroad grade. The former railroad bed is now used as a recreational trail.

The Maine Geologic Survey (MGS) “Surficial Geology of Boyd Lake Quadrangle, Maine, Open-File No. 81-5” (1981) indicates that surficial soils in the vicinity of the B&A

Overhead Bridge are predominantly glacial-stream deposits with nearby glacial-marine soil unit contacts. The glacial-stream deposits are typically comprised of sand and gravel, but may include minor till. Glacial-marine deposits generally consist of silt, clay, and sand, commonly a clayey silt, but sand is abundant at the surface in some places.

According to the Maine Geologic Survey "Bedrock Geologic Map of Maine" (1985), the bedrock at the B&A Overhead Bridge site consists of Silurian-Ordovician, calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions in the vicinity of the existing bridge by drilling six test borings, BB-LBAR-101 through BB-LBAR-106. The approximate boring locations are shown on Sheet 2, Boring Location Plan, and the generalized subsurface conditions are shown on Sheet 3, Interpretive Subsurface Profile, found at the end of this report. We terminated BB-LBAR-103 and 104 with bedrock cores. Maine Test Borings of Brewer, Maine, conducted the borings on January 2 through January 12, 2012. We present the details and sampling methods used, field data obtained, and soil and groundwater conditions encountered in the boring logs on Sheets 4 and 5, Boring Logs, and in Appendix A, Boring Logs, at the end of this report.

The borings were drilled using solid stem and hollow stem augers and cased wash boring techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report were gathered using rope and cathead drilling methods except at BB-LBAR-104 where the drilling contractor used an automatic hammer. Thus, for all of the borings except BB-LBAR-104 the SPT N-values correspond to SPT N_{60}-values. At BB-LBAR 104, we compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.783 to the raw field N-values obtained with the Maine Test Boring drill rig. Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core at BB-LBAR-103. Drill casing was out of plumb near the bottom of the boring at BB-LBAR-104 which prevented advancement of the NQ-2 core barrel. Consequently, bedrock at this boring was cored using a BX core barrel producing a 1 5/8-inch diameter rock core.

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

4.0 LABORATORY TESTING

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

5.0 SUBSURFACE CONDITIONS

The bridge is situated atop an esker over a deep soil cut constructed to accommodate a railroad grade now abandoned and converted to a recreational trail. Other than the pavement structure gravel, the approach embankment soils and the soils we encountered in the borings in the vicinity of the existing bridge predominantly consist of glacial-stream deposits. We encountered minor, inter-fingered, units of glacial till in borings BB-LBAR-103 and BB-LBAR-104, and a minor unit of glacial-marine soil at BB-LBAR-104. The following sections discuss the subsurface conditions we encountered.

5.1 Granular Fill

We encountered granular fill to a depth of approximately 9.0 feet below ground surface (bgs) in BB-LBAR-106. The fill is generally comprised of fine to coarse sand, with little gravel and silt. The one SPT N_{60}-value we measured in the granular fill was 43 blows per foot (bpf) indicating that the unit is dense in consistency.

The tested granular fill sample had a water content of approximately 7 percent. Grain size analyses conducted on the selected sample of fill indicates that this soil is classified as A-1-b by the AASHTO Classification System and SW-SM under the Unified Soil Classification System.

5.2 Glacial-Stream Deposits

We encountered glacial stream soils to depths ranging between approximately 22 feet and 100 feet bgs at the boring locations. Note that the glacial-stream soils were not fully penetrated in some of the borings. The glacial stream sediments had wide ranging grain size distributions consisting of any one of the following:

- fine to coarse sand, trace silt;
- fine to coarse sandy gravel, trace silt;
- gravel with little to some fine to coarse sand, trace silt;
- fine to medium sand, trace silt;
- fine to medium sand, trace gravel, trace silt;

- fine to coarse sand, trace to some gravel, trace to little silt;
- gravelly, fine to coarse sand, trace silt.

We also encountered cobbles at numerous depths in the glacial-stream soils and occasional boulders. The SPT N_{60}-values in the glacial stream deposits ranged from 8 to 99 bpf indicating that the unit is loose to very dense in consistency. The glacial stream sediment samples selected for laboratory testing had water contents ranging between approximately 1 and 24 percent. Grain size analyses conducted on selected samples of the glacial-stream soils indicate that the soils are classified as A-1-a, A-2-4 and A-3 by the AASHTO Classification System and SM, SP, SW, SP-SM and GW-GM under the Unified Soil Classification System.

5.3 Glacial Till

We observed minor units, approximately 15.0 feet thick and 17.4 feet thick, of glacial till soils at approximately 50.0 feet bgs in boring BB-LBAR-103, and 84.0 feet bgs in boring BB-LBAR-104, respectively. The glacial till soils had a typical grain size distribution consisting of:

- gravel with little to some fine to coarse sand, trace to little silt;

We also encountered numerous cobbles in the glacial till soils. The SPT N_{60}-values in the till deposits ranged from 31 to 117 bpf indicating that the unit is dense to very dense in consistency. The single till sample selected for laboratory testing had a water content of 6 percent. Grain size analyses conducted on the selected sample of the glacial till indicates that the soils are classified as A-1-a by the AASHTO Classification System and GW-GM under the Unified Soil Classification System.

5.4 Glacial-Marine Soils

We observed a minor, approximately 3.5 foot thick layer of glacial-marine soil at approximately 80.5 feet bgs in boring BB-LBAR-104. Based on visual observations, the glacial-marine soil had a typical grain size distribution consisting of:

- fine to medium sand, trace coarse sand, trace gravel, trace to little silt;

We measured one SPT N_{60}-value in the glacial-marine deposit of 59 bpf indicating that the unit is very dense in consistency. We did not perform laboratory tests on these soils.

5.5 Bedrock

We encountered bedrock at approximate depths ranging from 99.8 to 101.4 feet bgs at the BB-LBAR-103 and BB-LBAR-104, respectively. We visually identified the bedrock cores at both boring locations as a grey to black, fine-grained, metamorphic slate or metasiltstone that is complexly bedded and folded, soft, slightly weathered with very close to close, open joints. The bedrock contains fractures that are oriented from horizontal to near vertical with some slickensides evident. Silt in-filling and iron-staining are absent. The bedrock also contains numerous calcite seams.

We determined that the rock quality designation (RQD) of the bedrock ranged from 33 to 47 percent in BB-LBAR-103 which correlates to a poor rock mass quality. Due to drilling difficulties, we used a small diameter BX core barrel in BB-LBAR-104. The BX rock core has insufficient diameter to produce valid RQD values, but verified bedrock type and depth. Table 5-1 below summarizes the top of bedrock elevations at the boring locations:

Boring	Station (feet)	Depth to Bedrock (feet bgs)	Elev. of Apparent Bedrock Surface (feet)
BB-LBAR-103	35+11.1, 33.3 LT	99.8	90.3
BB-LBAR-104	34+60.0, 16.6 RT	101.4	96.5

Table 5-1. Bedrock Depth and Elevation at the Boring Locations

5.6 Groundwater

We observed the groundwater level at the approximate depths of 18.9 feet in BB-LBAR-102, 37.0 feet in BB-LBAR-103 and 18.5 feet bgs in boring BB-LBAR-105. We did not observe groundwater in the other borings. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

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

6.0 FOUNDATION ALTERNATIVES

Both rehabilitation of the existing substructures and superstructure and full replacement options were initially being considered by the MaineDOT bridge design team. However, MaineDOT has selected the B&A Overhead Bridge replacement project to construct a composite tubular arch bridge structure. The fiber-reinforced polymer (FRP) arches were developed by the University of Maine's AEWC Advanced Structures and Composites Center in Orono, Maine. The arch headwalls will consist of mechanically stabilized earth walls.

The following practical foundation alternatives may be considered to support the arch structure:

- Spread footings,
- Driven H-piles

We recommend the use of spread footings to support the arch structure to take advantage of the good bearing soils present at the site. Section 7.0, Geotechnical Design Recommendations, of this report provides recommendations for spread footings and mechanically stabilized earth headwalls proposed to support the arch structure.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

AEWC's carbon fiber tubes are inflated off-site and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the tubular structure.

The arch structure will be supported by reinforced concrete stem walls constructed on spread footings. The arch headwalls will consist of mechanically stabilized earth walls. The FRP tubular arches will be designed by AEWC and supplied to the project designer, TY Lin. TY Lin will design the arch stem wall and spread footing foundations. The MSE walls used for arch headwalls will be a design-build item provided by a proprietary wall designer retained by the contractor.

7.1 Arch Stem Wall and Foundation Design

Arch stem walls and supporting spread footings shall be designed for all relevant strength and service limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 6th Edition, 2012 Articles 3.4.1, 11.5.5., and 12.5. The stem walls and footings shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and any lateral thrust forces transferred through the bridge arches.

The design of project stem walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of spread footings on soil. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.55 at the concrete footing to soil interface. For footings on soil, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed the middle two-thirds (2/3) of the footing base width.

Service limit load conditions may control arch foundation design of the B&A Overhead Bridge. A resistance factor of $\phi = 1.0$ shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and overall stability. The overall global stability of a foundation is typically investigated during final design by the geotechnical engineer at the Service I Load Combination and a resistance factor, φ, of 0.65. In this case, global stability conditions are satisfied since the factored Service I bearing resistances are greater than the proposed factored bearing pressures provided by TY Lin (See Appendix C, Calculations) and the proposed final build-out embankment heights will be less than the original height of the esker. The foundations shall also be constructed a minimum of 6.5 feet below exterior finished grade providing additional passive resistance.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31 calculated using Rankine Theory for stem wall design. The designer may assume Soil Type 4 [Bridge Design Guide (BDG) Section 3.6.1] for backfill soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pounds per cubic foot (pcf). We provide supporting calculations in Appendix C, Calculations.

Early design results indicated that lateral arch forces are insignificant for the proposed arch. However, if calculation of passive earth pressures for resisting lateral thrust forces from the arch are needed, the designer should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating small lateral arch stem wall foundation movements. If the ratio of lateral foundation movement to stem wall height, (y/H), exceeds 0.005, then a Coulomb passive earth pressure coefficient, K_p, of 6.89 should be used. See Appendix C, Calculations, for supporting documentation. The designer shall use a resistance factor for passive earth pressures (φ_{ep}) of 0.50 for earth pressure mobilized to resist lateral sliding forces. For designing the arch foundation reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG when an approach slab is not specified. The live load surcharge on arch stem walls and footings may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of no less than 2.0 feet, per LRFD Table 3.11.6.4-1. The live load surcharge on abutments (stem walls) may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 7-1 below:

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

Table 7-1. Equivalent Height of Soil for Estimating Live Load Surcharge

Arch stem walls shall include a drainage system behind the wall to intercept any groundwater. The contractor shall construct weep holes 10 feet center-to-center approximately 6 inches above finish grade through the arch stem walls. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

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

To help reduce stress concentrations and premature foundation distress, the contractor shall remove the existing intermediate pier columns and their spread footings at the arch foundation locations down to a level at least 1.5 feet below the proposed arch spread footing base elevation. The placement of 1.5 feet of compacted granular borrow beneath the proposed arch footing will invoke a more uniform subgrade reaction.

7.2 Arch Foundation Bearing Resistance

The strength and service limit state factored bearing resistances for spread footings on compacted granular fill or native glacial stream deposits shall depend on the footing size as presented in Figure 7-1 below. The designer shall use the service limit factored bearing resistance for preliminary footing sizing to control settlement. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 3.4.1 and 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-1. The factored bearing resistance for any structure founded on compacted fill or native glacial stream deposits shall be investigated at the strength and service limit states using factored loads and a factored bearing resistance dependent on the proposed footing width.

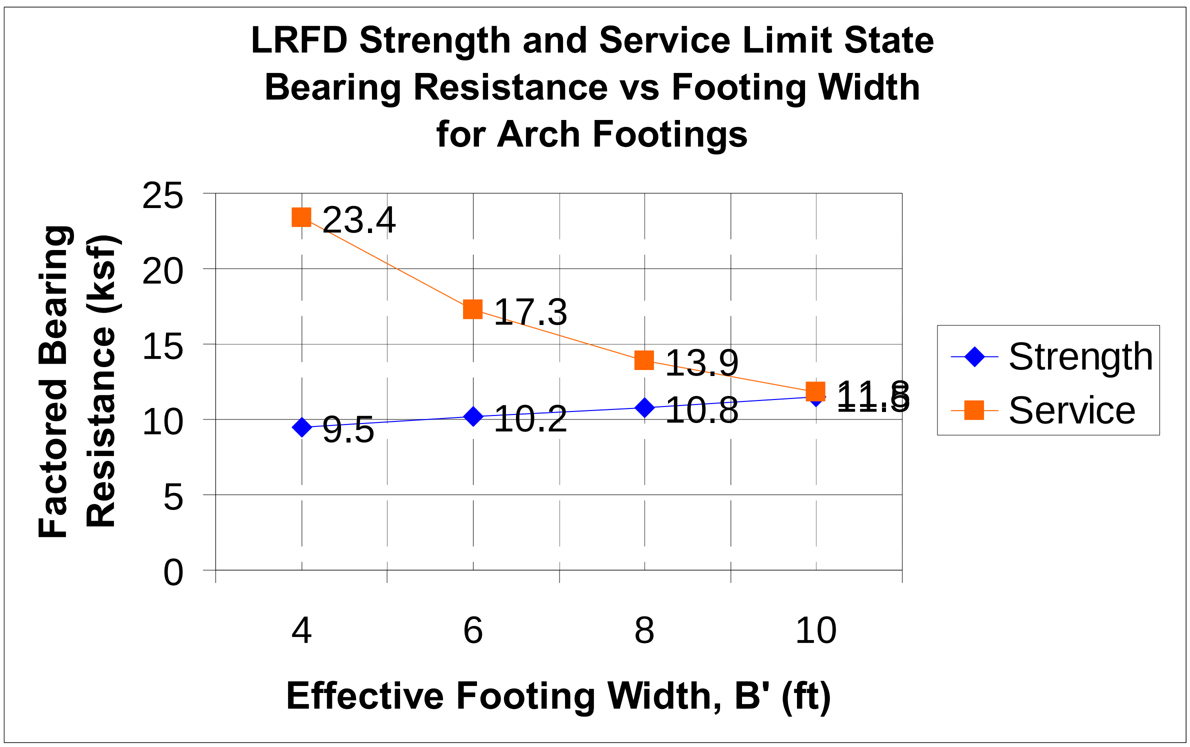


Figure 7-1. Strength and Service Limit State Factored Bearing Resistance for Spread Footings on Glacial Stream Deposits or Compacted Fill

Figure 7-1 above shows the appropriate factored bearing resistance assuming ½-inch settlement for comparable effective footing widths, B' for the strength and service limit states. The strength limit state bearing resistances assume a bearing resistance factor, φ_b, for spread footings on soil of 0.45, based on bearing resistance evaluation using semi-empirical methods. The service limit state bearing resistances assume a bearing resistance factor of $\phi = 1.0$. Both the strength and service limit bearing resistances are adequate for the proposed footing size of 8 feet based on maximum bearing pressures provided by TY Lin. See Appendix C, Calculations, for supporting documentation.

7.3 Mechanically Stabilized Earth Arch Headwall Design

The arch headwalls will consist of mechanically stabilized earth walls. Design and construction of the walls shall be in accordance with LRFD and MaineDOT Special Provision 636, Mechanically Stabilized Earth Retaining Wall (See Appendix D, Special Provisions). These walls shall be a design-build item designed by a Professional Engineer licensed in Maine and retained by the contractor.

MSE walls shall be investigated at the strength limit state for bearing capacity failure, lateral sliding, excessive loss of base contact, pullout of soil reinforcements and structural failure. A sliding resistance factor, φ_τ, of 1.0 shall be applied to the nominal sliding resistance of soil-on-soil beneath the MSE mass. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.58 (tan 30 degrees) at the foundation soil to soil in-fill interfaces. For the lowest MSE level, the eccentricity of factored loads at the strength limit state shall not exceed the middle two-thirds (2/3) of the reinforced soil base width. The MSE wall designer may assume Soil Type 4 backfill soil material for the MSE wall volume backfill with the following properties: ϕ = 32 degrees, γ = 125 pcf. Factored bearing pressures should be computed using a uniform base distribution over the effective width.

The wall designer shall use a bearing resistance of 6 ksf to limit settlement to 1-inch when analyzing the service limit state, and for preliminary reinforcement length sizing. The MSE mass shall be assessed at the service limit state using a resistance factor of ϕ = 1.0 for settlement and horizontal movement, and the overall stability shall be assessed at the Service I Load Combination with a resistance factor, ϕ, of 0.65. We recommend that the designer use a Coulomb active earth pressure coefficient, K_a, of 0.31 to evaluate the external stability of the wall. The wall designer shall estimate the traffic surcharge as a uniform horizontal earth pressure due to 2.0 feet of soil. The strength and service limit factored bearing resistance values we calculated for MSE reinforced soil volumes founded on compacted granular soils are provided in Figure 7-2 below. See Appendix C, Calculations, for supporting documentation.

The reinforcing length shall be uniform throughout the entire height of the wall. A concrete leveling pad with a width no less than 2.0 feet shall be provided to support the MSE wall face elements. The leveling pad for the wall panels shall be founded a minimum of 6.5 feet below finished exterior grade for frost protection. An impervious geomembrane consisting of low-permeability, 2-sided, texture HDPE with a minimum thickness of 30 mils shall be installed near the top of the reinforced soil zone to minimize water infiltration. The MSE wall designer must also consider the placement of guardrail along the highway when designing reinforcement and membrane configuration.

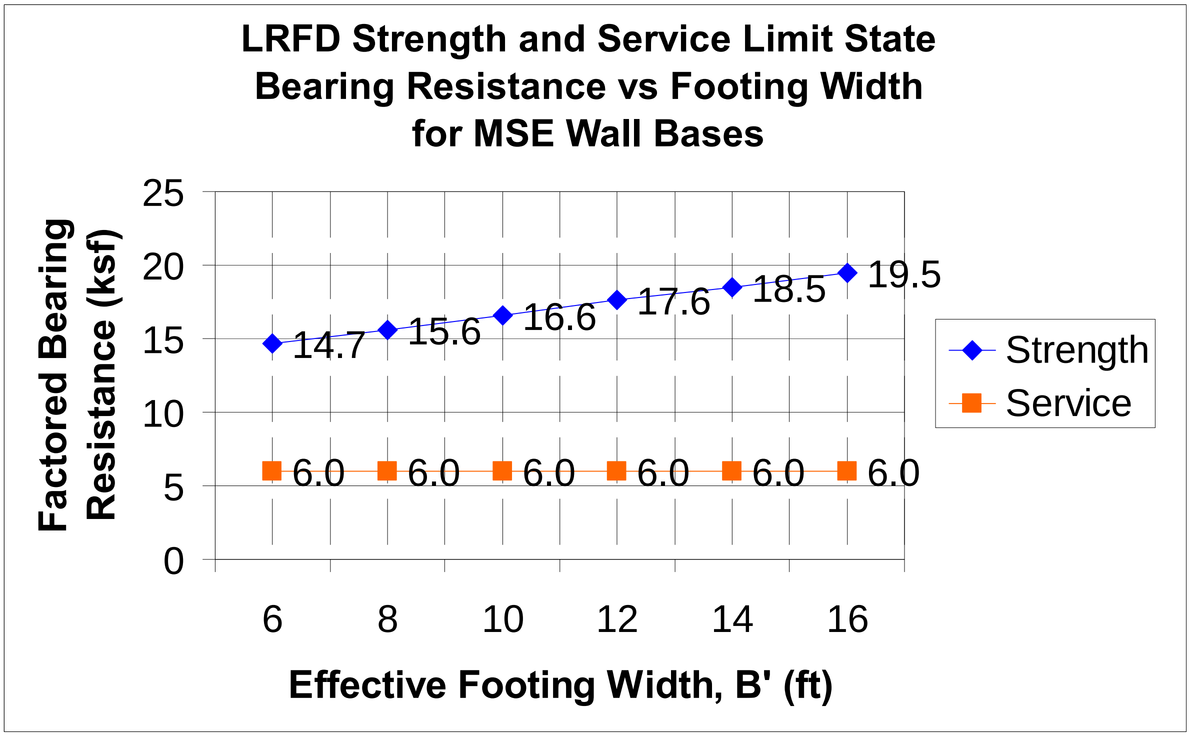


Figure 7-2. **Strength and Service Limit State Factored Bearing Resistance for MSE Wall Bases on Glacial Stream Deposits or Compacted Fill**

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

7.4 Settlement

Where service limit state bearing pressures are equal or less than the service limit bearing resistances presented in Section 7.2 of this report, we have estimated that total settlement of conventional arch spread footings constructed over a prepared subgrade consisting of compacted fill or native glacial stream deposits will be on the order of ½-inch. We estimate that differential settlement will be on the order of ½-inch or less. In all cases above, this settlement is acceptable and, due to the granular nature of the site soils, this settlement will occur during construction. We anticipate that post-construction settlement will be negligible.

We estimate that settlement as a result of embankment construction comprising approximately 23 feet of granular fill over natural soils will be on the order of 1-inch or less and will occur during construction. We anticipate that post-construction settlement will be negligible.

We estimate settlement beneath MSE wall bases will be on the order of 1-inch or less. This settlement is acceptable and will occur during construction. We anticipate that post-construction settlement will be negligible. Supporting settlement calculations are provided in Appendix C, Calculations.

7.5 Frost Protection

We have evaluated the potential frost depth at the site for footings placed on soil. Based on State of Maine frost depth maps, BDG Figure 5-1, the site has a design-freezing index of approximately 1910 F-degree days. Considering wet periods of the year, we assumed a water content of 20 percent, and this correlates to a frost depth of 6.4 feet at this site. We also considered frost depth projections computed by Modberg software developed by the US Army Cold Regions Research and Engineering Laboratory. The results of the Modberg frost depth model indicate a potential frost depth of 6.8 feet. Consequently, we recommend that any foundations or leveling pads constructed on soil at this site be founded a minimum of 6.5 feet below finished exterior grade. We present supporting calculations in Appendix C, Calculations.

The contractor shall remove the existing bridge abutments down to a level at least 6.5 feet below the proposed highway finished grade at the former abutment locations. This is necessary to help minimize frost heaves in the newly completed highway at the old abutment locations.

7.6 Seismic Design Considerations

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

7.7 Construction Considerations

The following construction conditions shall be considered by the contractor.

7.7.1 Excavation

The contractor shall remove the old intermediate pier columns and their foundations a minimum of 1.5 feet below the proposed bottom of the arch spread footing foundation elevation. **Construction of new arch spread footing foundations directly on old concrete footings will not be allowed**. This work may require staged construction methods, earth support systems, or shoring and bracing.

The contractor must remove the existing bridge abutments down to a level at least 6.5 feet below the highway finished grade to minimize frost heaves in the highway at the old abutment locations.

Construction of proposed arch headwall foundations will require soil excavation. Earth support systems may be required.

7.7.2 Foundation Subgrade Preparation

The contractor must sample and conduct Proctor tests on the footing subgrade soil before any foundation construction. Based on the subgrade soil Proctor test results, the contractor shall compact the subgrade soil to 95% of the Modified Proctor (AASHTO T-180) maximum dry density. After subgrade preparation, the contractor shall place at least 6 inches of ¾-inch crushed stone (See Appendix D, Special Provisions) over prepared subgrade and compact this material with at least 4 passes of a walk-behind vibratory compactor with minimum static weight of 200 lb.

7.7.3 Dewatering

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

The contractor must protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and/or rutting occur, we recommend that the contractor remove the disturbed soil materials and replace it with compacted granular borrow

7.7.4 Reuse of Excavated Soil

The project plans call for excavation of soil in the existing approach areas to achieve planned grades and to remove existing pier foundations and bridge abutments. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and any granular fill or native soil excavation may be used as common borrow below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met. Contractors should expect that prior to placement and compaction it may be necessary to spread out and dry portions of these soils that are excessively moist.

7.7.5 Embankment Area Construction

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

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

7.7.6 Erosion Control Recommendations

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

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement of the B&A Overhead Bridge over a recreational trail in Lagrange, Maine. We have prepared the report in accordance with generally accepted soil and foundation engineering practices. No other intended use or warranty is expressed or implied.

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations completed at discrete locations on the project site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

REFERENCES

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

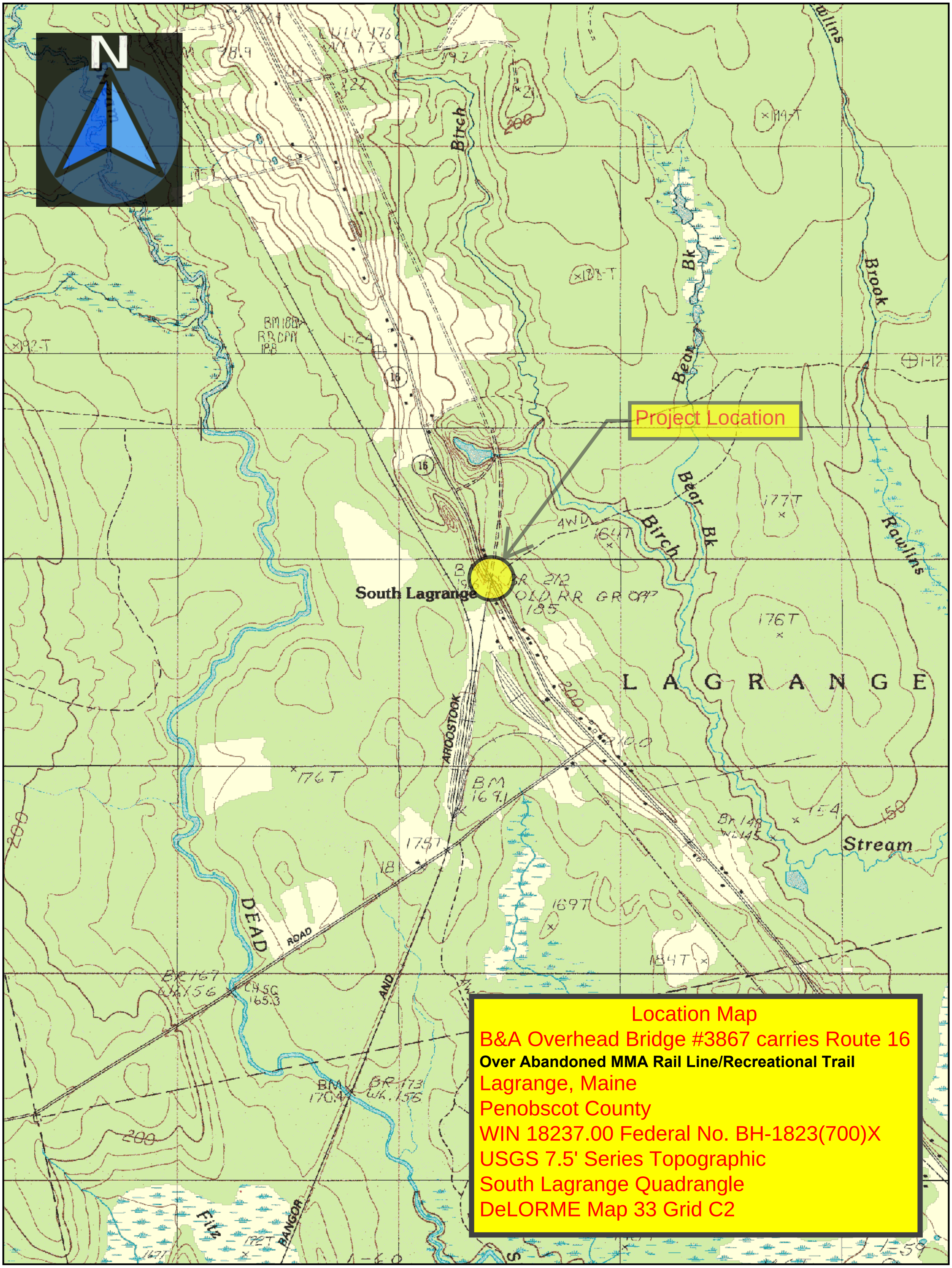
Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York, NY.

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

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

Terzaghi, K., Peck, R., and Mesri, G. (1996), Soil Mechanics in Engineering Practice, Third Edition, John Wiley and Sons, New York, NY.

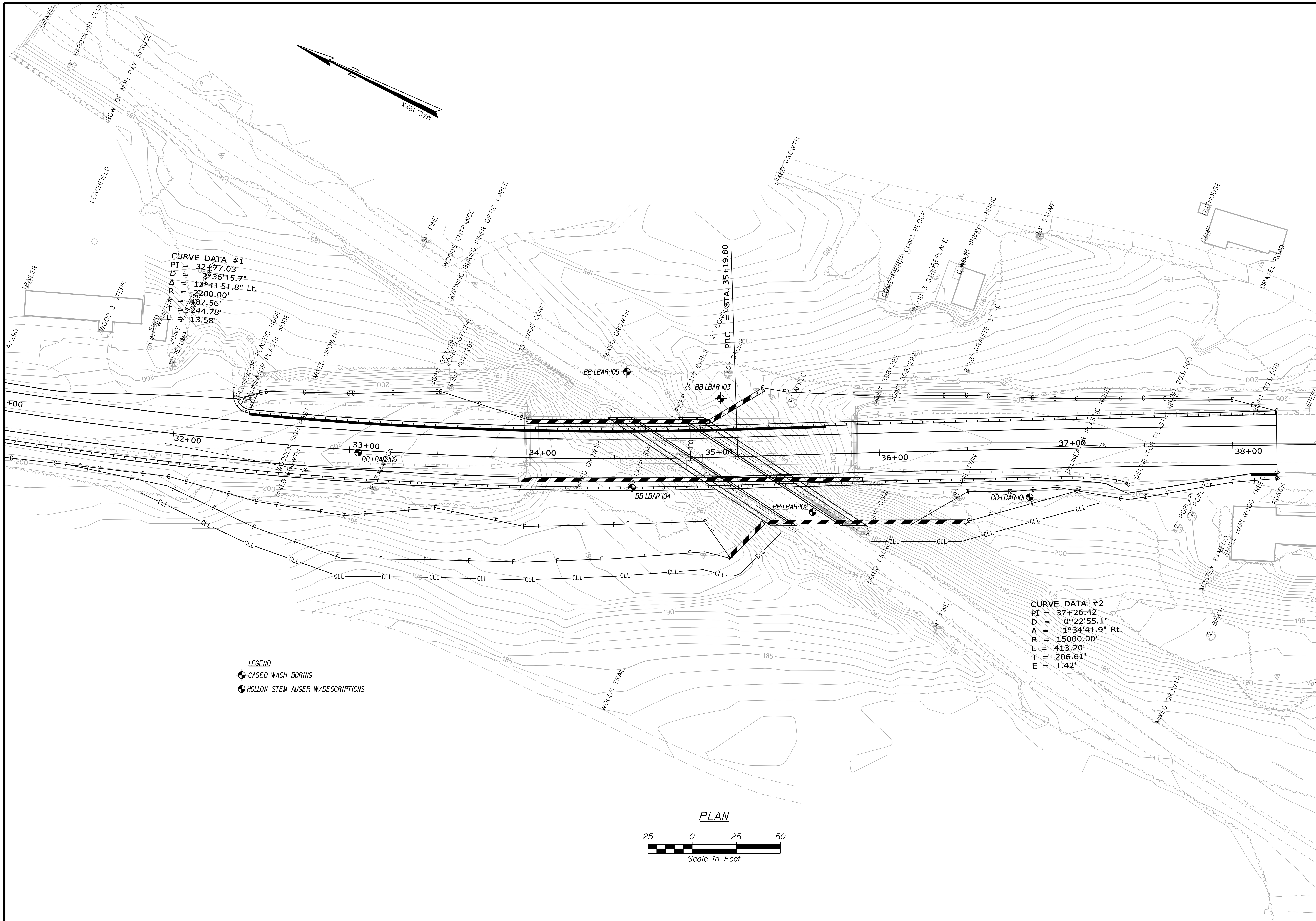
Sheets



Map Scale 1:24000

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

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:4/23/2012



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(700)X

BRIDGE NO. 3867 | WIN 18237.00 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M.MOREAU	FEB 2012
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

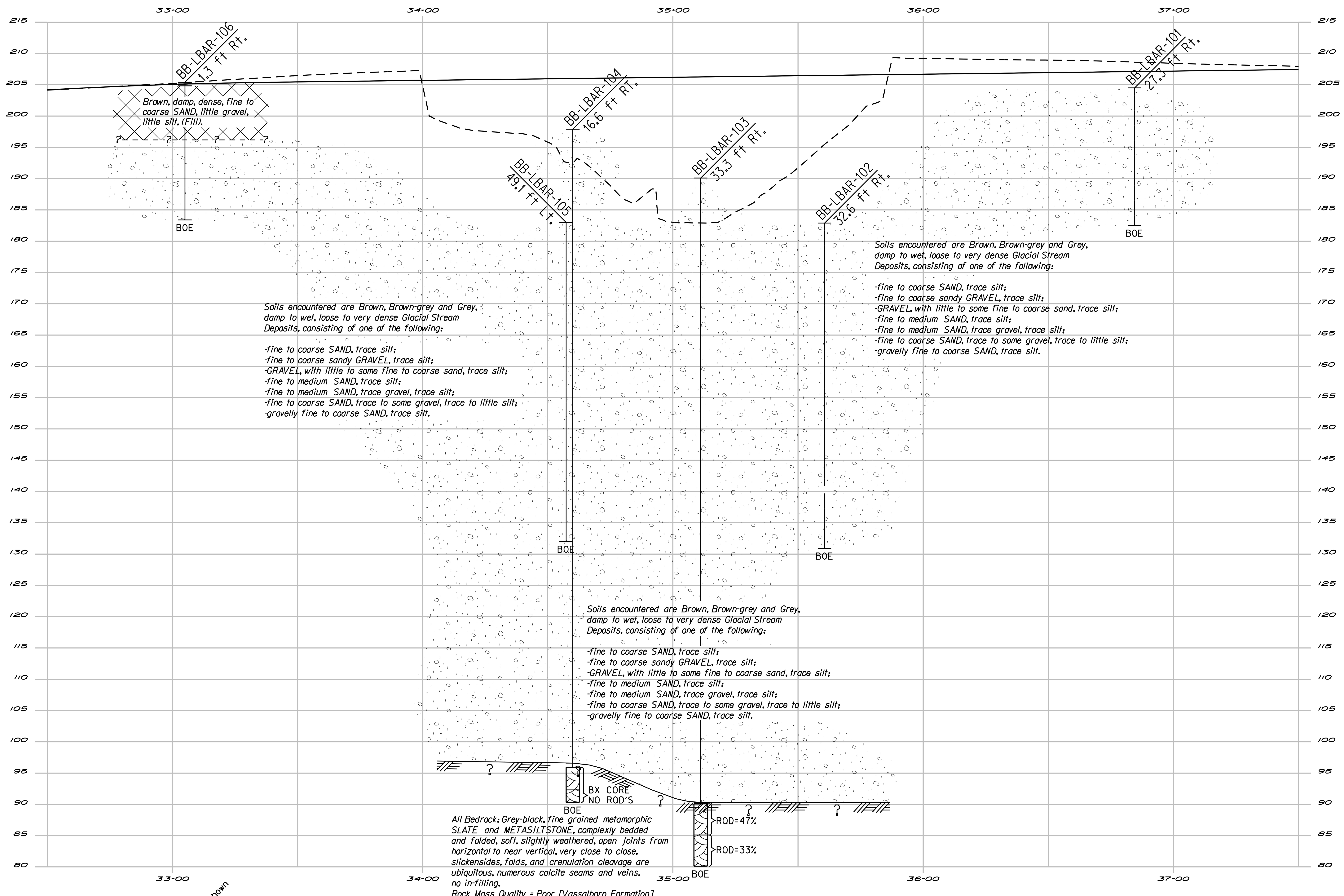
DATE

B&A OVERHEAD BRIDGE
ROUTE 16 OVER FORMER B&A RAILROAD
LAGRANGE PENOBSCOT COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 5

Filename: ...\00\geotech\msta\007_ISP1.dgn   Division: GEOTECH   Username: terry.white   Date:4/23/2012



PROFILE

HORIZ 25 0 25 50

VERT 10 0 10 20

SCALE

LEGEND

Boring No.
Offset, if shown

Pavement Thickness if applicable

Strata Interface

Boring

RQD= Rock Quality Designation for Rock Core Sample

BOE= Bottom Of Exploration

BOE No Refusal

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-1823(700)X

BRIDGE NO. 3867   WIN 18237.00   BRIDGE PLANS

PROJ. MANAGER	BY	DATE
DESIGN-DETAILED	M.MOREAU / T.WHITE	FEB 2012
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

B&A OVERHEAD BRIDGE
ROUTE 16 OVER FORMER B&A RAILROAD
LAGRANGE PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:4/23/2012

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-106 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	205.4	Auger ID/OD:	2.5"/6"
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/11/2012-1/11/2012	Drilling Method:	Hollow Stem Auger	Core Barrel:	N/A
Boring Location:	33+05, 1.3 ft Rt.	Casing ID/OD:	N/A	Water Level*:	
Hammer Efficiency Factor: 0.6		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							HSA	204.82	7" PAVEMENT. 0.58	
	1D	1/1	2.00 - 2.08	50(1")	---				Brown, damp, dense, fine to coarse SAND, little gravel and silt, (Fill).	
5	2D	24/18	5.00 - 7.00	17/26/17/33	43	43			Similar to above.	G#244775 A-1-b, SW-SM WC=6.8%
								196.40	9.00	
10	3D	24/15	10.00 - 12.00	21/24/12/11	36	36			Brown, damp, dense, fine to coarse SAND, trace fine gravel, trace silt and cobbles, (Glacial Stream Deposit).	
15	4D	24/12	15.00 - 17.00	17/23/21/27	44	44			Similar to above.	
								189.40	16.00 Brown, damp, dense to very dense, fine to coarse SAND, some fine gravel, little silt, (Glacial Stream Deposit).	G#261974 A-2-4, SM WC=8.3%
20	5D	24/8	20.00 - 22.00	26/31/25/33	56	56			Similar to above.	
								183.40	22.00 Bottom of Exploration at 22.00 feet below ground surface. NO REFUSAL	
25										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 Boring No.: BB-LBAR-106

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-105 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	183.0	Auger ID/OD:	2.5"/6"
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	M. Moreau/E. Giguere	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/10/2012-1/10/2012	Drilling Method:	HSA/CWB	Core Barrel:	N/A
Boring Location:	34+57.3, 49.1 ft Lt.	Casing ID/OD:	NW	Water Level*:	18.5 ft bgs.
Hammer Efficiency Factor: 0.6		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	18/18	0.00 - 1.50	24/34/34	68	68	HSA		Brown, damp, very dense, fine to coarse SAND, trace to little gravel, little silt, cobbles, (Glacial Stream Deposit). Auger cuttings indicate former gravel Railroad ballast near surface embedded into the Glacial Stream Deposits. Ground frozen to 6" bgs. Cobble at 1.5 ft bgs. Throwing spoon out of alignment.	
								179.50	3.50	
5	2D	24/19	5.00 - 7.00	4/5/6/7	11	11			Brown, damp, medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	G#244770 A-3, SP WC=3.0%
10	3D	24/16	10.00 - 12.00	7/7/9/10	16	16			Similar to above.	G#244771 A-3, SP WC=3.8%
								170.00	13.00	
15	4D	24/11	15.00 - 17.00	16/15/15/18	30	30			Grey-brown, medium dense, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit). Cobble at 19.0 ft bgs.	
20	5D	24/8	20.00 - 22.00	11/12/9/10	21	21			Similar to above, except wet.	G#244772 A-1-a, SW WC=8.0%
								159.50	23.50	
25	6D	24/8	25.00 - 27.00	16/15/14/19	33	33			Grey-brown, damp, dense, fine to coarse SAND, little gravel, trace silt, (Glacial Stream Deposit). Some running sand coming up in auger. Cobbles from 27.0-29.0 ft bgs.	
								154.50	28.50	
30	7D	24/14	30.00 - 32.00	7/6/9/8	15	15			Grey-brown, medium dense, damp, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit).	G#244773 A-1-a, SW WC=10.2%
35	8D	24/3	35.00 - 37.00	10/9/10/13	19	19	23, 24, 28, 39, 44		Running sand in auger prevents getting sample from hollow stem, switched to NW Casing and continue boring.	
								144.50	38.50	
40	9D	24/13	40.00 - 42.00	9/9/10/13	19	19	31, 39, 44, 60, 61		Grey, moist, medium dense to dense, fine to medium SAND, trace coarse sand, trace gravel, trace silt, (Glacial Stream Deposit).	G#244774 A-3, SP-SM WC=21.3%
45	10D	24/12	45.00 - 47.00	9/10/12/17	22	22	37, 52, a63		Similar to above, but sand is coarser. a63 blows for 0.6 ft. Washed ahead from 47.6-49.0 ft bgs. Cobble from 47.6-47.9 ft bgs.	
	11D	24/12	49.00 - 51.00	23/21/25/26	46	46			Similar to above.	
								132.00	51.00 Bottom of Exploration at 51.00 feet below ground surface. NO REFUSAL	
75										

Remarks:

CWB = Cased Wash Boring

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-LBAR-105

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-104 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	197.9	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/2/12-1/5/12	Drilling Method:	Cased Wash Boring	Core Barrel:	BX Core
Boring Location:	34+60, 16.6 ft Rt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.783		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	24/21	0.00 - 2.00	12/48/28/17	76	99	SSA	197.73	2" TOPSOIL. 0.17 Brown, moist, very dense, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit).	
5	2D	24/8	5.00 - 7.00	11/15/24/24	39	51	20, 22, 29, 33, 31		Similar to above, except wet.	
10	3D	24/8	10.00 - 12.00	29/37/25/8	62	81	11, 25, 21, 32, 34		Similar to above.	
15	4D	24/9	15.00 - 17.00	20/17/22/19	39	51	20, 33, 36, 39, 55		Similar to above.	
20	5D	9.6/9.6	20.00 - 20.80	22/50(3.6")	---		11, 23, 29, 37, 50	177.90	20.00 Brown, moist, very dense, fine to coarse SAND, trace fine gravel, occasional cobbles, (Glacial Stream Deposit). Cobble from 20.8-21.2 ft bgs. Roller Coned ahead from 21.2-25.0 ft bgs.	
25	6D	24/12	25.00 - 27.00	16/13/15/15	28	37	8, 13, 32, 28, 47	172.90	25.00 Grey, wet, dense, fine to coarse SAND, trace gravel and silt, (Glacial Stream Deposit).	G#244764 A-3, SP-SM WC=14.9%
30	7D	24/12	30.00 - 32.00	14/13/17/16	30	39	14, 19, 35, 32, 50		Similar to above.	G#244765 A-2-4, SP-SM WC=13.9%
35	8D	24/15	35.00 - 37.00	18/14/17/16	31	40	24, 25, 34, 44, 38		Similar to above.	
40	9D	24/15	40.00 - 42.00	12/15/17/18	32	42	16, 24, 42, 44, 51		Similar to above.	G#244766 A-3, SP-SM WC=17.1%
45	10D	24/18	45.00 - 47.00	16/16/18/19	34	44	32, 29, 39, 47, 57		Similar to above.	
50	11D	24/15	50.00 - 52.00	21/23/34/38	57	74	43, 57, 116, 98, 87	147.90	50.00 Grey, wet, medium dense to very dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	G#244767 A-3, SP-SM WC=20.3%
55	12D	24/2	55.00 - 57.00	24/15/6/6	21	27	39, 37, 37, 41, 57		Similar to above. Cobble from 59.0-60.0 ft bgs.	
60	13D	24/5	60.00 - 62.00	30/16/20/21	36	47	53, 64, 45, 35, 30	137.90	60.00 Brown and grey, wet, dense to very dense, GRAVEL, little to some fine to coarse sand, trace silt, (Glacial Stream Deposit).	G#244768 A-1-a, GW WC=3.7%
65	14D	24/1	65.00 - 67.00	11/20/31/37	51	67	32, 54, 149, 137, 113		Similar to above. very gravelly or cobbly	
70	15D	24/7	70.00 - 72.00	25/23/22/19	45	59	70, 70, 68, 64, 70		Similar to above.	G#244769 A-1-a, GW-GM WC=6.7%
75										

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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-LBAR-104

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-104 WIN: 18237.00

Driller: Maine Test Boring; Elevation (ft.) 197.9; Auger ID/OD: 4" Solid Stem; Operator: Porter/Nason; Datum: NAVD88; Sampler: Standard Split Spoon; Logged By: E. Giguere/M. Moreau; Rig Type: Mobile B-53; Hammer Wt./Fall: 140#/30"; Date Start/Finish: 1/2/12-1/5/12; Drilling Method: Cased Wash Boring; Core Barrel: BX Core; Boring Location: 34+60, 16.6 ft Rt.; Casing ID/OD: NW; Water Level*: None Observed; Hammer Efficiency Factor: 0.783; 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	16D	24/7	75.00 - 77.00	19/31/16/27	47	61	60, 73, 98, 132, 178		Grey-brown, wet, very dense, fine to coarse sandy GRAVEL, trace silt, occasional cobbles, (Glacial Stream Deposits). Cobble from 79.7-80.5 ft bgs.	
80	17D	24/12	80.50 - 82.50	17/20/25/25	45	59	58, 61, 105, 125, 162	117.40	80.50 Grey-brown, wet, very dense, fine to medium SAND, trace coarse sand, trace gravel, trace to little silt, (Glaciomarine).	
								113.90	84.00	
85	18D	24/13	85.00 - 87.00	31/28/22/16	50	65	54, 81, 97, 95, 103		Grey brown, wet, very dense, GRAVEL, some fine to coarse sand, little silt, numerous cobbles, (Till). Washed ahead to 90.0 ft bgs.	
90	19D	24/13	90.00 - 92.00	38/56/34/17	90	117	51, 62, 78, 84, 92		Similar to above. Washed ahead to 95.0 ft bgs. Switched to Rope and Cathead with 140# safety hammer after taking 90.0 ft sample.	
95	20D	24/?	95.00 - 97.00				67, 83, 104, 121, 136		Similar to above. Blow counts at 95-97 ft bgs taken with rope and cathead: 24/31/35/27 So N60 = 66.	
100	MD	4/0	100.00 - 100.33	50	---		93, a173		Roller Coned ahead from 100.0-102.0 ft bgs. a173 blows for 0.1 ft.	
								96.50	101.40 Top of Bedrock at Elev. 96.5 ft.	
	R1b	43.2/43.2	102.00 - 105.60	RQD = N/A			BX CORE		Could not get NQ-2 Core Barrel down to bottom of casing, switched to BX Core Barrel. b No RQD values are provided because the BX core diameter is insufficient for estimating competent RQD values.	
105	R2b	24/24	105.60 - 107.60	RQD = N/A				90.30	R1 and R2 Bedrock: Grey-black, fine grained, metamorphic SLATE and METASILTSTONE, complexly bedded and folded, soft, slightly weathered, with very close to close, open joints. The bedrock contains fractures that are oriented from horizontal to near vertical, with some slickensides evident, numerous calcite seams and veins, no in-filling or iron-staining. [Vassalboro Formation] R1:Core Times (min:sec) 102.0-103.0 ft (5:35) 103.0-104.0 ft (4:30) 104.0-105.0 ft (5:05) 105.0-105.6 ft (2:42) 100% Recovery Core Blocked at 105.6 ft bgs. R2:Core Times (min:sec) 105.6-106.6 ft (5:35) 106.6-107.6 ft (2:45) 100% Recovery Core Blocked at 107.6 ft bgs. 107.60 Bottom of Exploration at 107.60 feet below ground surface.	
110										
115										
120										
125										

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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-LBAR-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1823(700)X
BRIDGE NO. 3867 WIN 18237.00 BRIDGE PLANS

		BY	DATE	
PROJ. MANAGER				SIGNATURE
DESIGN-DETAILED	M.MOREAU	T.WHITE	FEB 2012	
CHECKED-REVIEWED				P.E. NUMBER
DESIGN2-DETAILED2				
DESIGN3-DETAILED3				DATE
REVISIONS 1				
REVISIONS 2				
REVISIONS 3				
REVISIONS 4				
FIELD CHANGES				

B&A OVERHEAD BRIDGE
ROUTE 16 OVER FORMER B&A RAILROAD
LAGRANGE PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER 4 OF 5

Filename: ...\msta\009_BORING LOGS2.dgn   Division: GEOTECH   Username: terry.white   Date:4/23/2012

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-103
WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	190.1	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	Casing ID/OD:	NW & HW	Water Level*:	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor: 0.6		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	24/24	0.00 - 2.00	10/4/4/5	8	8	SSA	189.85	3" TOPSOIL. 0.25 Brown, damp, loose, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
5	2D	24/18	5.00 - 7.00	2/3/5/5	8	8	17		Similar to above.	
							14			
							24			
							26			
							29	181.10	9.00	
10	3D	24/12	10.00 - 12.00	6/8/8/12	16	16	8		Brown, moist, medium dense, fine to medium SAND, trace fine gravel, trace to little silt, (Glacial Stream Deposit).	
							20			
							27			
							43			
							42			
15	4D	24/15	15.00 - 17.00	4/8/8/9	16	16	20		Similar to above.	G#244759 A-2-4, SP WC=17.9%
							33			
							36			
							36			
							35			
20	5D	24/12	20.00 - 22.00	5/6/7/14	13	13	21		Similar to above.	G#244760 A-2-4, SM WC=9.1%
							40			
							41			
							53			
							56			
25	6D	15.6/2	25.00 - 26.30	8/10/50(3.6")	---		19			
							23		Cobble from 26.3-26.8 ft bgs. Roller Coned ahead to 30.0 ft bgs.	
							30			
							18	161.60	28.50	
							23			
30	7D	24/15	30.00 - 32.00	7/8/8/11	16	16	37		Brown, wet, medium dense, fine to medium SAND, trace silt, (Glacial Stream Deposit).	G#244761 A-2-4, SP WC=24.1%
							41			
							47			
							52			
							67			
35	8D	24/13	35.00 - 37.00	4/9/10/10	19	19	41		Similar to above.	
							62			
							71			
							73	151.10	39.00	
							81			
40	9D	24/12	40.00 - 42.00	8/9/12/14	21	21	48		Brown, wet, medium dense to dense, fine to coarse SAND, trace gravel, little silt, (Glacial Stream Deposit).	G#244762 A-2-4, SM WC=22.6%
							61			
							139			
							142			
							153			
45	10D	24/7	45.00 - 47.00	12/13/18/23	31	31	89		Similar to above.	
							141			
							138			
							140			
							139	140.10	50.00	
50	11D	24/13	50.00 - 52.00	16/21/11/13	32	32	91		Brown-grey, wet, dense, GRAVEL, some fine to coarse sand, trace silt, (Till).	G#244763 A-1-a, GW-GM WC=5.8%
							126			
							143			
							157			
							139			
55	12D	24/4	55.00 - 57.00	8/14/17/26	31	31	97		Similar to above.	
							127			
							131	132.10	58.00	
							128			
							131			
60	13D	24/8	60.00 - 62.00	24/48/40/29	88	88	13		Brown-grey, wet, very dense, GRAVEL, little fine to coarse sand, trace silt and cobbles, (Till). Changed to NW Casing at 60.0 ft bgs.	
							47			
							84			
							73			
							67	125.10	65.00	
65	14D	24/3	65.00 - 67.00	27/22/30/21	52	52	27		Grey, wet, very dense, GRAVEL, little fine to coarse sand, trace silt, cobbles, (Glacial Stream Deposit).	
							52			
							51	122.10	68.00	
							32			
							51			
70	15D	24/15	70.00 - 72.00	17/26/15/23	41	41	23		Grey-brown, wet, dense, fine to medium SAND, trace silt, (Glacial Stream Deposit).	
							34			
							65			
							71			
75							79			

Remarks:
Hole caved at 28.8 ft bgs, no H2O, 1/10/2012

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-LBAR-103

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-103
WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	190.1	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	Casing ID/OD:	NW & HW	Water Level*:	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor: 0.6		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
75	16D	20/12	75.00 - 76.67	14/19/42/50(.2')	61	61	20		Similar to above, but coarser.	
							75	114.10	76.00	
							104			
							114			
							108			
80	17D	24/8	80.00 - 82.00	21/22/18/18	40	40	39		Brown-grey, wet, dense to very dense, GRAVEL, some fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	
							67			
							83			
							94			
							129			
85	MD	4/0	85.00 - 85.33	50 (4.0")	---		63		Drill attitude indicates similar to above. Washed ahead from 85.0-90.0 ft bgs.	
							97			
							127			
							119			
							187			
90	18D	6/3.6	90.00 - 90.50	75 (6.0")	---		137		Similar to above.	
							141			
							101			
							121			
							102	96.10	94.00	
95	19D	24/15	95.00 - 97.00	26/38/58/42	96	96	69		Brown-grey, wet, very dense, gravelly fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	
							110			
							115			
							a158		a158 blows for 0.7 ft. Roller Coned ahead from 98.7-100.0 ft bgs.	
								90.30	99.80	
100	R1	60/60	100.00 - 105.00	RQD = 47%			NQ-2 CORE		Top of Bedrock at Elev. 90.3 ft. R1 and R2 Bedrock: Grey-black, fine grained, metamorphic SLATE and METASILTSTONE, complexly bedded and folded, soft, slightly weathered, with very close to close, open joints. The bedrock contains fractures that are oriented from horizontal to near vertical, with some slickensides evident, numerous calcite seams and veins, no in-filling or iron-staining. [Vassalboro Formation] R1:Core Times (min:sec) 100.0-101.0 ft (4:00) 101.0-102.0 ft (4:00) 102.0-103.0 ft (2:30) 103.0-104.0 ft (3:00) 104.0-105.0 ft (3:00) 100% Recovery	
105	R2	60/60	105.00 - 110.00	RQD = 33%					R2:Core Times (min:sec) 105.0-106.0 ft (3:00) 106.0-107.0 ft (4:00) 107.0-108.0 ft (3:30) 108.0-109.0 ft (4:30) 109.0-110.0 ft (4:30) 100% Recovery	
110								80.10	110.00 Bottom of Exploration at 110.00 feet below ground surface.	
115										
120										
125										

Remarks:
Hole caved at 28.8 ft bgs, no H2O, 1/10/2012

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 2 of 2
Boring No.: BB-LBAR-103

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-102
WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	182.9	Auger ID/OD:	2.5"/6"
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/11/2012-1/12/2012	Drilling Method:	Hollow Stem Auger	Core Barrel:	N/A
Boring Location:	35+60.6, 32.6 ft Rt.	Casing ID/OD:	N/A	Water Level*:	18.9 ft bgs.
Hammer Efficiency Factor: 0.6		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							HSA			
	1D	24/18	2.00 - 4.00	27/33/27/33	60	60			Brown, damp, very dense, GRAVEL, some fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	G#244753 A-1-a, GW-GM WC=1.9%
								178.90	4.00	
5	2D	24/12	5.00 - 7.00	22/22/23/45	45	45			Brown, damp, dense, fine to coarse sandy GRAVEL, trace silt, cobbly, (Glacial Stream Deposit).	G#244754 A-1-a, GW-GM WC=1.7%
									Small Boulder from 7.08-7.67 ft bgs.	
								174.90	8.00	
10	3D	24/3	10.00 - 12.00	21/23/19/24	42	42			Brown, damp, loose to dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
15	4D	24/18	15.00 - 17.00	6/8/9/10	17	17			Similar to above.	G#244755 A-2-4, SP WC=3.6%
20	5D	24/16	20.00 - 22.00	4/5/5/4	10	10			Similar to above.	G#244756 A-2-4, SP WC=19.7%
25	6D	24/16	25.00 - 27.00	7/10/8/9	18	18			Brown-grey, wet, loose to medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
									Running sand at 30.0 ft bgs, used head of water to keep it out when pulling rods up and down.	
30	7D	24/9	30.00 - 32.00	4/4/5/9	9	9			Similar to above.	
								148.90	34.00	
35	8D	24/8	35.00 - 37.00	6/7/9/12	16	16			Brown-grey, medium dense, fine to coarse SAND, little fine gravel, trace silt, (Glacial Stream Deposit).	G#244757 A-3, SP-SM WC=21.3%
								144.90	38.00	
40	9D	24/8	40.00 - 42.00	7/7/6/4	13	13			Brown-grey, medium dense, fine to coarse sandy GRAVEL, trace silt, (Glacial Stream Deposit).	G#244758 A-1-a, GW WC=8.6%
45	10D	24/6	45.00 - 47.00	9/8/8/7	16	16			Similar to above.	
50	11D	24/12	50.00 - 52.00	4/9/10/15	19	19				
								130.90	52.00 Bottom of Exploration at 52.00 feet below ground surface. NO REFUSAL	
55										
60										
65										
70										
75										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-LBAR-102

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

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-101
WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	204.5	Auger ID/OD:	2.5"/6"
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/11/2012-1/11/2012	Drilling Method:	Hollow Stem Auger	Core Barrel:	N/A
Boring Location:	36+84.5, 27.3 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.6		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
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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
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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	24/8	0.00 - 2.00	4/5/9/12	14	14	HSA	204.25	3" TOPSOIL. 0.25 Brown, damp, medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
5	2D	24/15	5.00 - 7.00	6/6/5/16	11	11			Similar to above.	G#244751 A-3, SP-SM WC=5.3%
								196.50	8.00	
10	3D	24/12	10.00 - 12.00	12/13/15/17	28	28			Brown, damp, medium dense to dense, fine to coarse sandy GRAVEL, trace silt, with cobbles, (Glacial Stream Deposit).	
15	4D	24/3	15.00 - 17.00	14/16/21/16	37	37			Similar to above.	
20	5D	24/10	20.00 - 22.00	20/27/20/47	47	47			Similar to above.	G#244752 A-1-a, GW-GM WC=0.6%
								182.50	22.00 Bottom of Exploration at 22.00 feet below ground surface. NO REFUSAL	
25										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-LBAR-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1823(700)X
BRIDGE NO. 3867
WIN 18237.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER ----------		
DESIGN-DETAILED M.MOREAU	T.WHITE	FEB 2012
CHECKED-REVIEWED	-----	-----
DESIGN2-DETAILED2	-----	-----
DESIGN3-DETAILED3	-----	-----
REVISIONS 1	-----	-----
REVISIONS 2	-----	-----
REVISIONS 3	-----	-----
REVISIONS 4	-----	-----
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

B&A OVERHEAD BRIDGE
ROUTE 16 OVER FORMER B&A RAILROAD
LAGRANGE PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-101
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	204.5	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012-1/11/2012	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	36+84.5, 27.3 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**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	24/8	0.00 - 2.00	4/5/9/12	14	14	HSA	204.25		3" TOPSOIL. 0.25 Brown, damp, medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
5	2D	24/15	5.00 - 7.00	6/6/5/16	11	11				Similar to above.	G#244751 A-3, SP-SM WC=5.3%
								196.50		8.00	
10	3D	24/12	10.00 - 12.00	12/13/15/17	28	28				Brown, damp, medium dense to dense, fine to coarse sandy GRAVEL, trace silt, with cobbles, (Glacial Stream Deposit).	
15	4D	24/3	15.00 - 17.00	14/16/21/16	37	37				Similar to above.	
20	5D	24/10	20.00 - 22.00	20/27/20/47	47	47				Similar to above.	G#244752 A-1-a, GW-GM WC=0.6%
								182.50		22.00 **Bottom of Exploration at 22.00 feet below ground surface.** NO REFUSAL	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-102
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	182.9	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012-1/12/2012	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	35+60.6, 32.6 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	18.9 ft bgs.
Hammer Efficiency Factor:	0.6	**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							HSA				
	1D	24/18	2.00 - 4.00	27/33/27/33	60	60				Brown, damp, very dense, GRAVEL, some fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	G#244753 A-1-a, GW-GM WC=1.9%
								178.90		4.00	
5	2D	24/12	5.00 - 7.00	22/22/23/45	45	45				Brown, damp, dense, fine to coarse sandy GRAVEL, trace silt, cobbly, (Glacial Stream Deposit).	G#244754 A-1-a, GW-GM WC=1.7%
										Small Boulder from 7.08-7.67 ft bgs.	
								174.90		8.00	
10	3D	24/3	10.00 - 12.00	21/23/19/24	42	42				Brown, damp, loose to dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
15	4D	24/18	15.00 - 17.00	6/8/9/10	17	17				Similar to above.	G#244755 A-2-4, SP WC=3.6%
20	5D	24/16	20.00 - 22.00	4/5/5/4	10	10				Similar to above.	G#244756 A-2-4, SP WC=19.7%
25											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-102 WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	182.9	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012-1/12/2012	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	35+60.6, 32.6 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	18.9 ft bgs.
Hammer Efficiency Factor:	0.6	**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.
25	6D	24/16	25.00 - 27.00	7/10/8/9	18	18				Brown-grey, wet, loose to medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
										Running sand at 30.0 ft bgs, used head of water to keep it out when pulling rods up and down.	
30	7D	24/9	30.00 - 32.00	4/4/5/9	9	9				Similar to above.	
34								148.90		34.00	
35	8D	24/8	35.00 - 37.00	6/7/9/12	16	16				Brown-grey, medium dense, fine to coarse SAND, little fine gravel, trace silt, (Glacial Stream Deposit).	G#244757 A-3, SP-SM WC=21.3%
38								144.90		38.00	
40	9D	24/8	40.00 - 42.00	7/7/6/4	13	13				Brown-grey, medium dense, fine to coarse sandy GRAVEL, trace silt, (Glacial Stream Deposit).	G#244758 A-1-a, GW WC=8.6%
45	10D	24/6	45.00 - 47.00	9/8/8/7	16	16				Similar to above.	
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-102
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	182.9	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012-1/12/2012	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	35+60.6, 32.6 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	18.9 ft bgs.
Hammer Efficiency Factor:	0.6	**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
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	11D	24/12	50.00 - 52.00	4/9/10/15	19	19					
								130.90		52.00 **Bottom of Exploration at 52.00 feet below ground surface.** NO REFUSAL	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-103 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	190.1	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	Casing ID/OD:	NW & HW	Water Level*:	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/24	0.00 - 2.00	10/4/4/5	8	8	SSA	189.85		3" TOPSOIL. 0.25 Brown, damp, loose, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	
5	2D	24/18	5.00 - 7.00	2/3/5/5	8	8	17			Similar to above.	
							14				
							24				
							26				
							29	181.10		9.00	
10	3D	24/12	10.00 - 12.00	6/8/8/12	16	16	8			Brown, moist, medium dense, fine to medium SAND, trace fine gravel, trace to little silt, (Glacial Stream Deposit).	
							20				
							27				
							43				
							42				
15	4D	24/15	15.00 - 17.00	4/8/8/9	16	16	20			Similar to above.	G#244759 A-2-4, SP WC=17.9%
							33				
							36				
							36				
							35				
20	5D	24/12	20.00 - 22.00	5/6/7/14	13	13	21			Similar to above.	G#244760 A-2-4, SM WC=9.1%
							40				
							41				
							53				
25							56				

Remarks:

Hole caved at 28.8 ft bgs, no H2O. 1/10/2012

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: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-103 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	190.1	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	Casing ID/OD:	NW & HW	Water Level*:	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	15.6/2	25.00 - 26.30	8/10/50(3.6")	---		19				
							23			Cobble from 26.3-26.8 ft bgs. Roller Coned ahead to 30.0 ft bgs.	
							30				
							18	161.60		28.50	
							23				
30	7D	24/15	30.00 - 32.00	7/8/8/11	16	16	37			Brown, wet, medium dense, fine to medium SAND, trace silt. (Glacial Stream Deposit).	G#244761 A-2-4, SP WC=24.1%
							41				
							47				
							52				
							67				
35	8D	24/13	35.00 - 37.00	4/9/10/10	19	19	41			Similar to above.	
							62				
							71				
							73				
							81	151.10		39.00	
40	9D	24/12	40.00 - 42.00	8/9/12/14	21	21	48			Brown, wet, medium dense to dense, fine to coarse SAND, trace gravel, little silt, (Glacial Stream Deposit).	G#244762 A-2-4, SM WC=22.6%
							61				
							139				
							142				
							153				
45	10D	24/7	45.00 - 47.00	12/13/18/23	31	31	89			Similar to above.	
							141				
							138				
							140				
50							139				

Remarks:

Hole caved at 28.8 ft bgs, no H2O. 1/10/2012

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-103
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	190.1	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor:	0.6	**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.
50	11D	24/13	50.00 - 52.00	16/21/11/13	32	32	91	140.10		50.00 Brown-grey, wet, dense, GRAVEL, some fine to coarse sand, trace silt, (Till).	G#244763 A-1-a, GW-GM WC=5.8%
							126				
							143				
							157				
							139				
55	12D	24/4	55.00 - 57.00	8/14/17/26	31	31	97			Similar to above.	
							127				
							131				
							128	132.10		58.00	
							131				
60	13D	24/8	60.00 - 62.00	24/48/40/29	88	88	13			Brown-grey, wet, very dense, GRAVEL, little fine to coarse sand, trace silt and cobbles, (Till). Changed to NW Casing at 60.0 ft bgs.	
							47				
							84				
							73				
							67				
65	14D	24/3	65.00 - 67.00	27/22/30/21	52	52	27	125.10		65.00 Grey, wet, very dense, GRAVEL, little fine to coarse sand, trace silt, cobbles, (Glacial Stream Deposit).	
							52				
							51				
							32	122.10		68.00	
							51				
70	15D	24/15	70.00 - 72.00	17/26/15/23	41	41	23			Grey-brown, wet, dense, fine to medium SAND, trace silt, (Glacial Stream Deposit).	
							34				
							65				
							71				
75							79				

Remarks:

Hole caved at 28.8 ft bgs, no H2O. 1/10/2012

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-103 WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	190.1	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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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)
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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	16D	20/12	75.00 - 76.67	14/19/42/50(.2')	61	61	20			Similar to above, but coarser.	
							75	114.10		76.00	
							104				
							114				
							108				
80	17D	24/8	80.00 - 82.00	21/22/18/18	40	40	39			Brown-grey, wet, dense to very dense, GRAVEL, some fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	
							67				
							83				
							94				
							129				
85	MD	4/0	85.00 - 85.33	50 (4.0")	---		63			Drill attitude indicates similar to above. Washed ahead from 85.0-90.0 ft bgs.	
							97				
							127				
							119				
							187				
90	18D	6/3.6	90.00 - 90.50	75 (6.0")	---		137			Similar to above.	
							141				
							101				
							121				
							102	96.10		94.00	
95	19D	24/15	95.00 - 97.00	26/38/58/42	96	96	69			Brown-grey, wet, very dense, gravelly fine to coarse sand, trace silt and cobbles, (Glacial Stream Deposit).	
							110				
							115				
							a158			a158 blows for 0.7 ft. Roller Coned ahead from 98.7-100.0 ft bgs.	
100								90.30		99.80	

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-103 WIN: 18237.00

Driller:	Maine Test Boring	Elevation (ft.)	190.1	Auger ID/OD:	4" Solid Stem
Operator:	Porter/Nason	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	Rig Type:	Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/5-6,9-10/2012	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	35+11.1, 33.3 ft Lt.	Casing ID/OD:	NW & HW	Water Level*:	37.0 ft bgs 1/9/2012
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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WOH = weight of 140lb. hammer
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WO1P = Weight of one person
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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.
100	R1	60/60	100.00 - 105.00	RQD = 47%			NQ-2 CORE			Top of Bedrock at Elev. 90.3 ft. R1 and R2 Bedrock: Grey-black, fine grained, metamorphic SLATE and METASILTSTONE, complexly bedded and folded, soft, slightly weathered, with very close to close, open joints. The bedrock contains fractures that are oriented from horizontal to near vertical, with some slickensides evident, numerous calcite seams and veins, no in-filling or iron-staining. [Vassalboro Formation] R1:Core Times (min:sec) 100.0-101.0 ft (4:00) 101.0-102.0 ft (4:00) 102.0-103.0 ft (2:30) 103.0-104.0 ft (3:00) 104.0-105.0 ft (3:00) 100% Recovery	
105	R2	60/60	105.00 - 110.00	RQD = 33%						R2:Core Times (min:sec) 105.0-106.0 ft (3:00) 106.0-107.0 ft (4:00) 107.0-108.0 ft (3:30) 108.0-109.0 ft (4:30) 109.0-110.0 ft (4:30) 100% Recovery	
110								80.10		110.00 **Bottom of Exploration at 110.00 feet below ground surface.**	
115											
120											
125											

Remarks:

Hole caved at 28.8 ft bgs, no H2O. 1/10/2012

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-104
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	197.9	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/2/12-1/5/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX Core
Boring Location:	34+60, 16.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**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)
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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.
0	1D	24/21	0.00 - 2.00	12/48/28/17	76	99	SSA	197.73		2" TOPSOIL. 0.17 Brown, moist, very dense, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit).	
5	2D	24/8	5.00 - 7.00	11/15/24/24	39	51	20			Similar to above, except wet.	
							22				
							29				
							33				
							31				
10	3D	24/8	10.00 - 12.00	29/37/25/8	62	81	11			Similar to above.	
							25				
							21				
							32				
							34				
15	4D	24/9	15.00 - 17.00	20/17/22/19	39	51	20			Similar to above.	
							33				
							36				
							39				
							55				
20	5D	9.6/9.6	20.00 - 20.80	22/50(3.6")	---		11	177.90		20.00 Brown, moist, very dense, fine to coarse SAND, trace fine gravel, occasional cobbles, (Glacial Stream Deposit). Cobble from 20.8-21.2 ft bgs. Roller Coned ahead from 21.2-25.0 ft bgs.	
							23				
							29				
							37				
25							50				

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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Boring No.: BB-LBAR-104

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad **Location:** Lagrange, Maine	**Boring No.:** BB-LBAR-104 **WIN:** 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	197.9	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/2/12-1/5/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX Core
Boring Location:	34+60, 16.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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)
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N-uncorrected = Raw field SPT N-value
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	16/13/15/15	28	37	8	172.90		25.00 Grey, wet, dense, fine to coarse SAND, trace gravel and silt, (Glacial Stream Deposit).	G#244764 A-3, SP-SM WC=14.9%
							13				
							32				
							28				
							47				
30	7D	24/12	30.00 - 32.00	14/13/17/16	30	39	14			Similar to above.	G#244765 A-2-4, SP-SM WC=13.9%
							19				
							35				
							32				
							50				
35	8D	24/15	35.00 - 37.00	18/14/17/16	31	40	24			Similar to above.	
							25				
							34				
							44				
							38				
40	9D	24/15	40.00 - 42.00	12/15/17/18	32	42	16			Similar to above.	G#244766 A-3, SP-SM WC=17.1%
							24				
							42				
							44				
							51				
45	10D	24/18	45.00 - 47.00	16/16/18/19	34	44	32			Similar to above.	
							29				
							39				
							47				
50							57				

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-104 WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	197.9	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/2/12-1/5/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX Core
Boring Location:	34+60, 16.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/15	50.00 - 52.00	21/23/34/38	57	74	43	147.90		50.00 Grey, wet, medium dense to very dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	G#244767 A-3, SP-SM WC=20.3%
							57				
							116				
							98				
							87				
55	12D	24/2	55.00 - 57.00	24/15/6/6	21	27	39			Similar to above.	
							37				
							37				
							41				
							57			Cobble from 59.0-60.0 ft bgs.	
60	13D	24/5	60.00 - 62.00	30/16/20/21	36	47	53	137.90		60.00 Brown and grey, wet, dense to very dense, GRAVEL, little to some fine to coarse sand, trace silt, (Glacial Stream Deposit).	G#244768 A-1-a, GW WC=3.7%
							64				
							45				
							35				
							30				
65	14D	24/1	65.00 - 67.00	11/20/31/37	51	67	32			Similar to above.	
							54				
							149			very gravelly or cobbly	
							137				
							113				
70	15D	24/7	70.00 - 72.00	25/23/22/19	45	59	70			Similar to above.	G#244769 A-1-a, GW-GM WC=6.7%
							70				
							68				
							64				
75							70				

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad	**Location:** Lagrange, Maine	**Boring No.:** BB-LBAR-104	**WIN:** 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	197.9	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/2/12-1/5/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX Core
Boring Location:	34+60, 16.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**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.
75	16D	24/7	75.00 - 77.00	19/31/16/27	47	61	60			Grey-brown, wet, very dense, fine to coarse sandy GRAVEL, trace silt, occasional cobbles, (Glacial Stream Deposits).	
							73				
							98				
							132				
							178			Cobble from 79.7-80.5 ft bgs.	
80	17D	24/12	80.50 - 82.50	17/20/25/25	45	59	58	117.40		80.50 Grey-brown, wet, very dense, fine to medium SAND, trace coarse sand, trace gravel, trace to little silt, (Glaciomarine).	
							61				
							105				
							125	113.90		84.00	
							162				
85	18D	24/13	85.00 - 87.00	31/28/22/16	50	65	54			Grey brown, wet, very dense, GRAVEL, some fine to coarse sand, little silt, numerous cobbles, (Till). Washed ahead to 90.0 ft bgs.	
							81				
							97				
							95				
							103				
90	19D	24/13	90.00 - 92.00	38/56/34/17	90	117	51			Similar to above. Washed ahead to 95.0 ft bgs. Switched to Rope and Cathead with 140# safety hammer after taking 90.0 ft sample.	
							62				
							78				
							84				
							92				
95	20D	24/?	95.00 - 97.00				67			Similar to above. Blow counts at 95-97 ft bgs taken with rope and cathead: 24/31/35/27 So N60 = 66.	
							83				
							104				
							121				
100							136				

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad Location: Lagrange, Maine	Boring No.: BB-LBAR-104 WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	197.9	**Auger ID/OD:**	4" Solid Stem
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere/M. Moreau	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/2/12-1/5/12	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX Core
Boring Location:	34+60, 16.6 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.783	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100	MD	4/0	100.00 - 100.33	50	---		93			Roller Coned ahead from 100.0-102.0 ft bgs.	
							a173	96.50		a173 blows for 0.1 ft. ——101.40	
	R1b	43.2/43.2	102.00 - 105.60	RQD = N/A			BX CORE			Top of Bedrock at Elev. 96.5 ft. Could not get NQ-2 Core Barrel down to bottom of casing, switched to BX Core Barrel. b No RQD values are provided because the BX core diameter is insufficient for estimating competent RQD values.	
105	R2b	24/24	105.60 - 107.60	RQD = N/A						R1 and R2 Bedrock: Grey-black, fine grained, metamorphic SLATE and METASILTSTONE, complexly bedded and folded, soft, slightly weathered, with very close to close, open joints. The bedrock contains fractures that are oriented from horizontal to near vertical, with some slickensides evident, numerous calcite seams and veins, no in-filling or iron-staining. [Vassalboro Formation]	
								90.30		R1:Core Times (min:sec) 102.0-103.0 ft (5:35) 103.0-104.0 ft (4:30) 104.0-105.0 ft (5:05) 105.0-105.6 ft (2:42) 100% Recovery Core Blocked at 105.6 ft bgs. R2:Core Times (min:sec) 105.6-106.6 ft (5:35) 106.6-107.6 ft (2:45) 100% Recovery Core Blocked at 107.6 ft bgs. ——107.60	
										Bottom of Exploration at 107.60 feet below ground surface.	
110											
115											
120											
125											

Remarks:

Boring Conducted with Automatic Hammer MTB#1.
Casing Driven with 300 LB Safety Hammer.

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: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-105
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	183.0	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau/E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/10/2012-1/10/2012	**Drilling Method:**	HSA/CWB	**Core Barrel:**	N/A
Boring Location:	34+57.3, 49.1 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	18.5 ft bgs.
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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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	18/18	0.00 - 1.50	24/34/34	68	68	HSA			Brown, damp, very dense, fine to coarse SAND, trace to little gravel, little silt, cobbles, (Glacial Stream Deposit). Auger cuttings indicate former gravel Railroad ballast near surface embeded into the Glacial Stream Deposits. Ground frozen to 6" bgs. Cobble at 1.5 ft bgs, throwing spoon out of alighment.	
								179.50		3.50	
5	2D	24/19	5.00 - 7.00	4/5/6/7	11	11				Brown, damp, medium dense, fine to medium SAND, trace coarse sand and silt, (Glacial Stream Deposit).	G#244770 A-2-4, SP WC=3.0%
10	3D	24/16	10.00 - 12.00	7/7/9/10	16	16				Similar to above.	G#244771 A-3, SP WC=3.8%
								170.00		13.00	
15	4D	24/11	15.00 - 17.00	16/15/15/18	30	30				Grey-brown, medium dense, damp, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit).	
										Cobble at 19.0 ft bgs.	
20	5D	24/8	20.00 - 22.00	11/12/9/10	21	21				Similar to above, except wet.	G#244772 A-1-a, SW WC=8.0%
								159.50		23.50	
25											

Remarks:

CWB = Cased Wash Boring

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-105
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	183.0	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau/E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/10/2012-1/10/2012	**Drilling Method:**	HSA/CWB	**Core Barrel:**	N/A
Boring Location:	34+57.3, 49.1 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	18.5 ft bgs.
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/8	25.00 - 27.00	16/19/14/19	33	33				Grey-brown, damp, dense, fine to coarse SAND, little gravel, trace silt, (Glacial Stream Deposit).	
										Some running sand coming up in auger. Cobbles from 27.0-29.0 ft bgs.	
								154.50		28.50	
30	7D	24/14	30.00 - 32.00	7/6/9/8	15	15				Grey-brown, medium dense, damp, gravelly fine to coarse SAND, trace silt, (Glacial Stream Deposit).	G#244773 A-1-a, SW WC=10.2%
35	8D	24/3	35.00 - 37.00	10/9/10/13	19	19	23			Running sand in auger prevents getting sample from hollow stem, switched to NW Casing and continue boring.	
							24				
							28				
							39	144.50		38.50	
							44				
40	9D	24/13	40.00 - 42.00	9/9/10/13	19	19	31			Grey, moist, medium dense to dense, fine to medium SAND, trace coarse sand, trace gravel, trace silt, (Glacial Stream Deposit).	G#244774 A-3, SP-SM WC=21.3%
							39				
							44				
							60				
							61				
45	10D	24/12	45.00 - 47.00	9/10/12/17	22	22	37			Similar to above, but sand is coarser.	
							52				
							a63			a63 blows for 0.6 ft. Washed ahead from 47.6-49.0 ft bgs. Cobble from 47.6-47.9 ft bgs.	
	11D	24/12	49.00 - 51.00	23/21/25/26	46	46				Similar to above.	
50											

Remarks:

CWB = Cased Wash Boring

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad			**Boring No.:**	BB-LBAR-105
Location:	Lagrange, Maine			**WIN:**	18237.00
Driller:	Maine Test Boring	**Elevation (ft.)**	183.0	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Moreau/E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/10/2012-1/10/2012	**Drilling Method:**	HSA/CWB	**Core Barrel:**	N/A
Boring Location:	34+57.3, 49.1 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	18.5 ft bgs.
Hammer Efficiency Factor:	0.6	**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.
50								132.00		51.00	
										Bottom of Exploration at 51.00 feet below ground surface. NO REFUSAL	
55											
60											
65											
70											
75											

Remarks:

CWB = Cased Wash Boring

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: B&A Overhead Bridge #3867 carries Route 16 over Former B&A Railroad
Location: Lagrange, Maine

Boring No.: BB-LBAR-106
WIN: 18237.00

Driller:	Maine Test Boring	**Elevation (ft.)**	205.4	**Auger ID/OD:**	2.5"/6"
Operator:	Porter/Nason	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012-1/11/2012	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	33+05, 1.3 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**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							HSA	204.82		7" PAVEMENT. —0.58	
	1D	1/1	2.00 - 2.08	50(1")	---					Brown, damp, dense, fine to coarse SAND, little gravel and silt, (Fill).	
5	2D	24/18	5.00 - 7.00	17/26/17/33	43	43				Similar to above.	G#244775 A-1-b, SW-SM WC=6.8%
								196.40		—9.00	
10	3D	24/15	10.00 - 12.00	21/24/12/11	36	36				Brown, damp, dense, fine to coarse SAND, trace fine gravel, trace silt and cobbles, (Glacial Stream Deposit).	
15	4D	24/12	15.00 - 17.00	17/23/21/27	44	44				Similar to above.	
								189.40		—16.00 Brown, damp, dense to very dense, fine to coarse SAND, some fine gravel, little silt, (Glacial Stream Deposit).	G#261974 A-2-4, SM WC=8.3%
20	5D	24/8	20.00 - 22.00	26/31/25/33	56	56				Similar to above.	
								183.40		—22.00 **Bottom of Exploration at 22.00 feet below ground surface.** NO REFUSAL	
25											

Remarks:

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Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lagrange   Work Number: 18237.00

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-LBAR-101, 2D	36+84.5	27.3 Rt.	5.0-7.0	244751	1	5.3			SP-SM	A-3	0
BB-LBAR-101, 5D	36+84.5	27.3 Rt.	20.0-22.0	244752	1	0.6			GW-GM	A-1-a	0
BB-LBAR-102, 1D	35+60.6	32.6 Rt.	2.0-4.0	244753	2	1.9			GW-GM	A-1-a	0
BB-LBAR-102, 2D	35+60.6	32.6 Rt.	5.0-7.0	244754	2	1.7			GW-GM	A-1-a	0
BB-LBAR-102, 4D	35+60.6	32.6 Rt.	15.0-17.0	244755	2	3.6			SP	A-2-4	0
BB-LBAR-102, 5D	35+60.6	32.6 Rt.	20.0-22.0	244756	2	19.7			SP	A-2-4	0
BB-LBAR-102, 8D	35+60.6	32.6 Rt.	35.0-37.0	244757	2	21.3			SP-SM	A-3	0
BB-LBAR-102, 9D	35+60.6	32.6 Rt.	40.0-42.0	244758	2	8.6			GW	A-1-a	0
BB-LBAR-103, 4D	35+11.1	33.3 Lt.	15.0-17.0	244759	3	17.9			SP	A-2-4	0
BB-LBAR-103, 5D	35+11.1	33.3 Lt.	20.0-22.0	244760	3	9.1			SM	A-2-4	II
BB-LBAR-103, 7D	35+11.1	33.3 Lt.	30.0-32.0	244761	3	24.1			SP	A-2-4	0
BB-LBAR-103, 9D	35+11.1	33.3 Lt.	40.0-42.0	244762	3	22.6			SM	A-2-4	II
BB-LBAR-103, 11D	35+11.1	33.3 Lt.	50.0-52.0	244763	3	5.8			GW-GM	A-1-a	0
BB-LBAR-104, 6D	34+60	16.6 Rt.	25.0-27.0	244764	4	14.9			SP-SM	A-3	0
BB-LBAR-104, 7D	34+60	16.6 Rt.	30.0-32.0	244765	4	13.9			SP-SM	A-2-4	0
BB-LBAR-104, 9D	34+60	16.6 Rt.	40.0-42.0	244766	4	17.1			SP-SM	A-3	0
BB-LBAR-104, 11D	34+60	16.6 Rt.	50.0-52.0	244767	4	20.3			SP-SM	A-3	0
BB-LBAR-104, 13D	34+60	16.6 Rt.	60.0-62.0	244768	4	3.7			GW	A-1-a	0
BB-LBAR-104, 15D	34+60	16.6 Rt.	70.0-72.0	244769	4	6.7			GW-GM	A-1-a	0
BB-LBAR-105, 2D	34+57.3	49.1 Lt.	5.0-7.0	244770	5	3.0			SP	A-2-4	II
BB-LBAR-105, 3D	34+57.3	49.1 Lt.	10.0-12.0	244771	5	3.8			SP	A-3	0
BB-LBAR-105, 5D	34+57.3	49.1 Lt.	20.0-22.0	244772	5	8.0			SW	A-1-a	0
BB-LBAR-105, 7D	34+57.3	49.1 Lt.	30.0-32.0	244773	5	10.2			SW	A-1-a	0
BB-LBAR-105, 9D	34+57.3	49.1 Lt.	40.0-42.0	244774	5	21.3			SP-SM	A-3	0
BB-LBAR-106, 2D	33+05	1.3 Rt.	5.0-7.0	244775	6	6.8			SW-SM	A-1-b	0
BB-LBAR-106, 4D	33+05	1.3 Rt.	16.0-17.0	261974	6	8.3			SM	A-2-4	II
Borings are not in Station order.											

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

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

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

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

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

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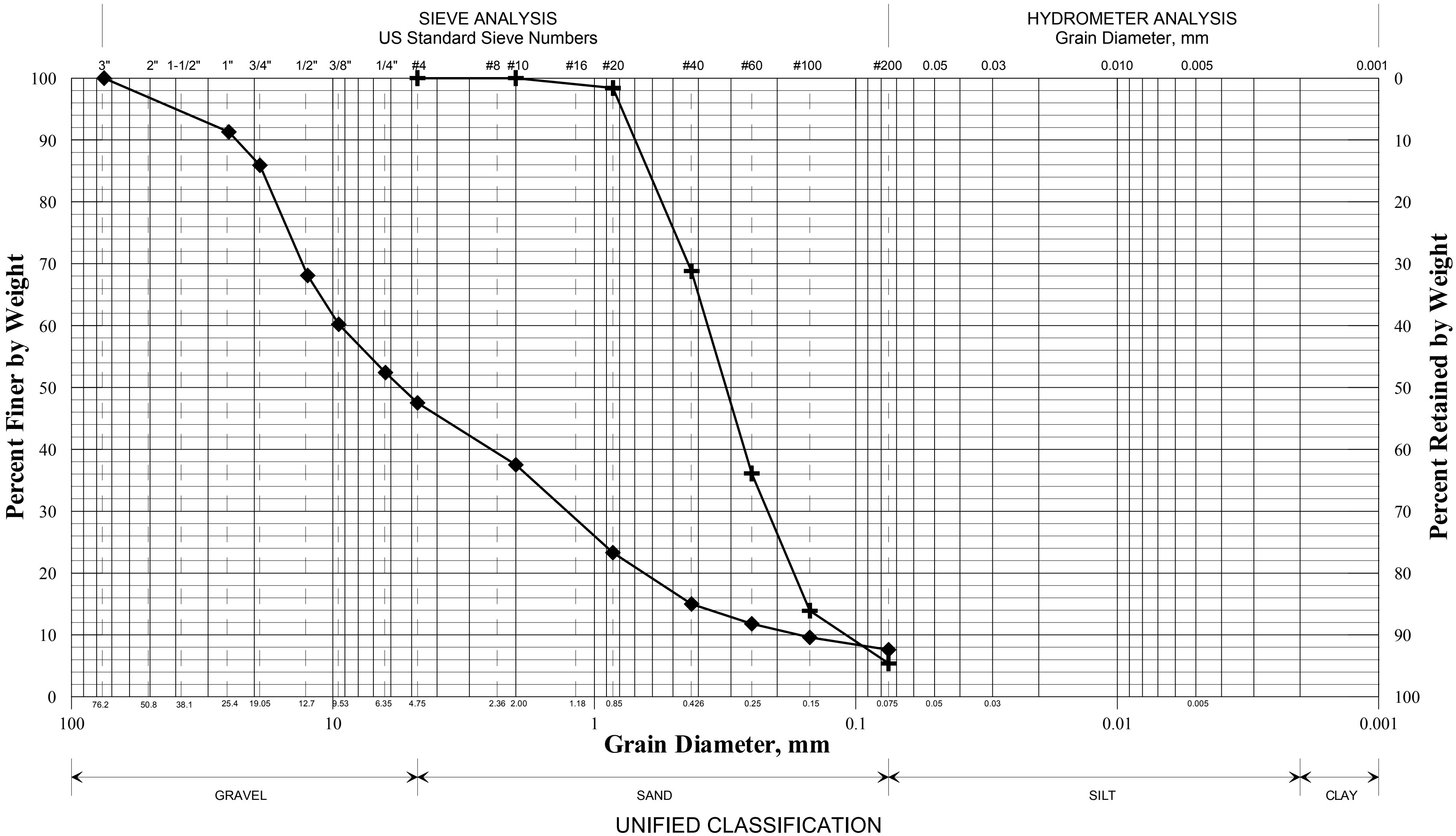
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-101/2D	36+84.5	27.3 RT	5.0-7.0	SAND, trace silt.	5.3			
◆	BB-LBAR-101/5D	36+84.5	27.3 RT	20.0-22.0	Sandy GRAVEL, trace silt.	0.6			
■									
●									
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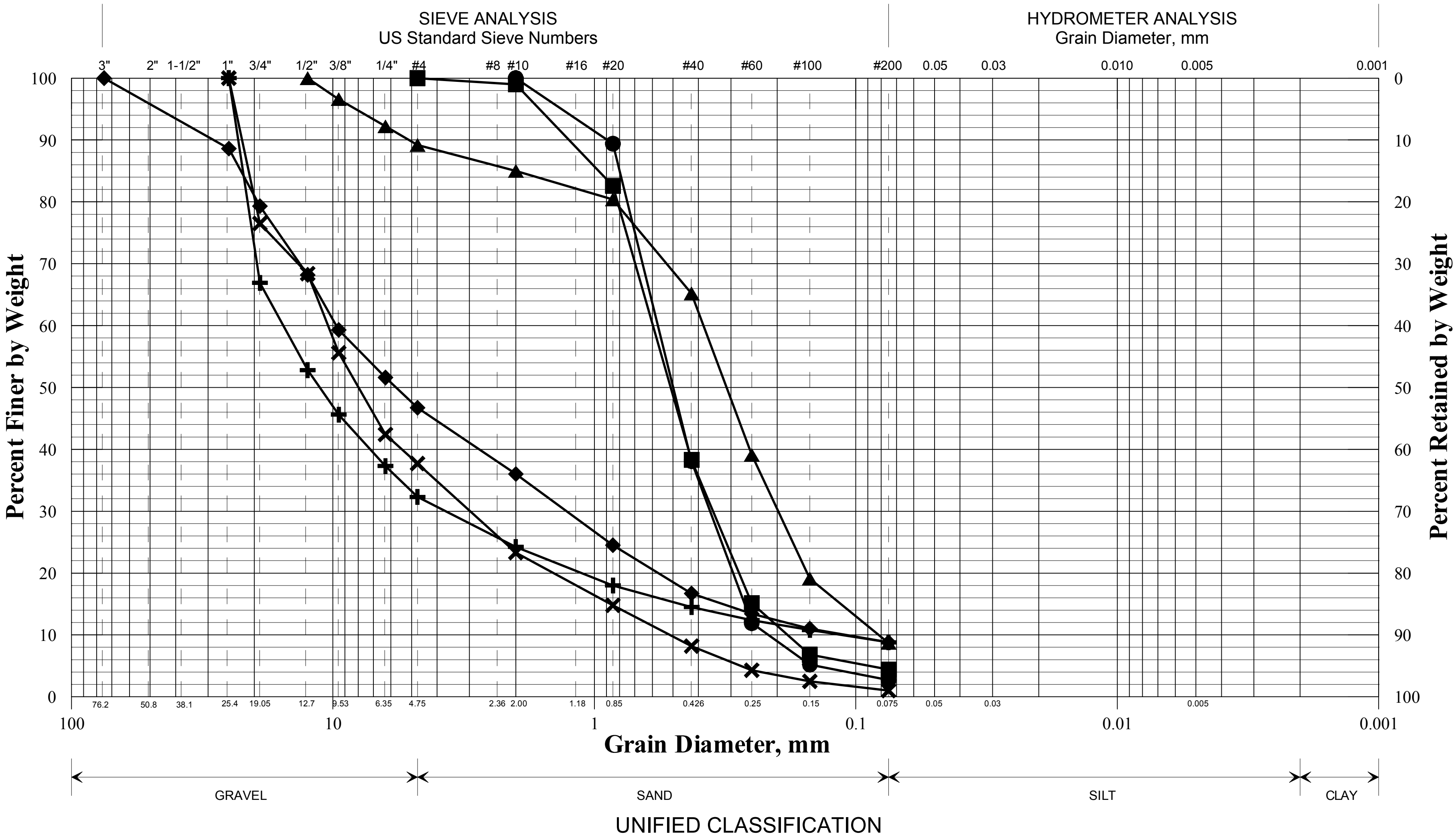
SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-102/1D	35+60.6	32.6 RT	2.0-4.0	GRAVEL, some sand, trace silt.	1.9			
◆	BB-LBAR-102/2D	35+60.6	32.6 RT	5.0-7.0	Sandy GRAVEL, trace silt.	1.7			
■	BB-LBAR-102/4D	35+60.6	32.6 RT	15.0-17.0	SAND, trace silt.	3.6			
●	BB-LBAR-102/5D	35+60.6	32.6 RT	20.0-22.0	SAND, trace silt.	19.7			
▲	BB-LBAR-102/8D	35+60.6	32.6 RT	35.0-37.0	SAND, little gravel, trace silt.	21.3			
✖	BB-LBAR-102/9D	35+60.6	32.6 RT	40.0-42.0	Sandy GRAVEL, trace silt.	8.6			

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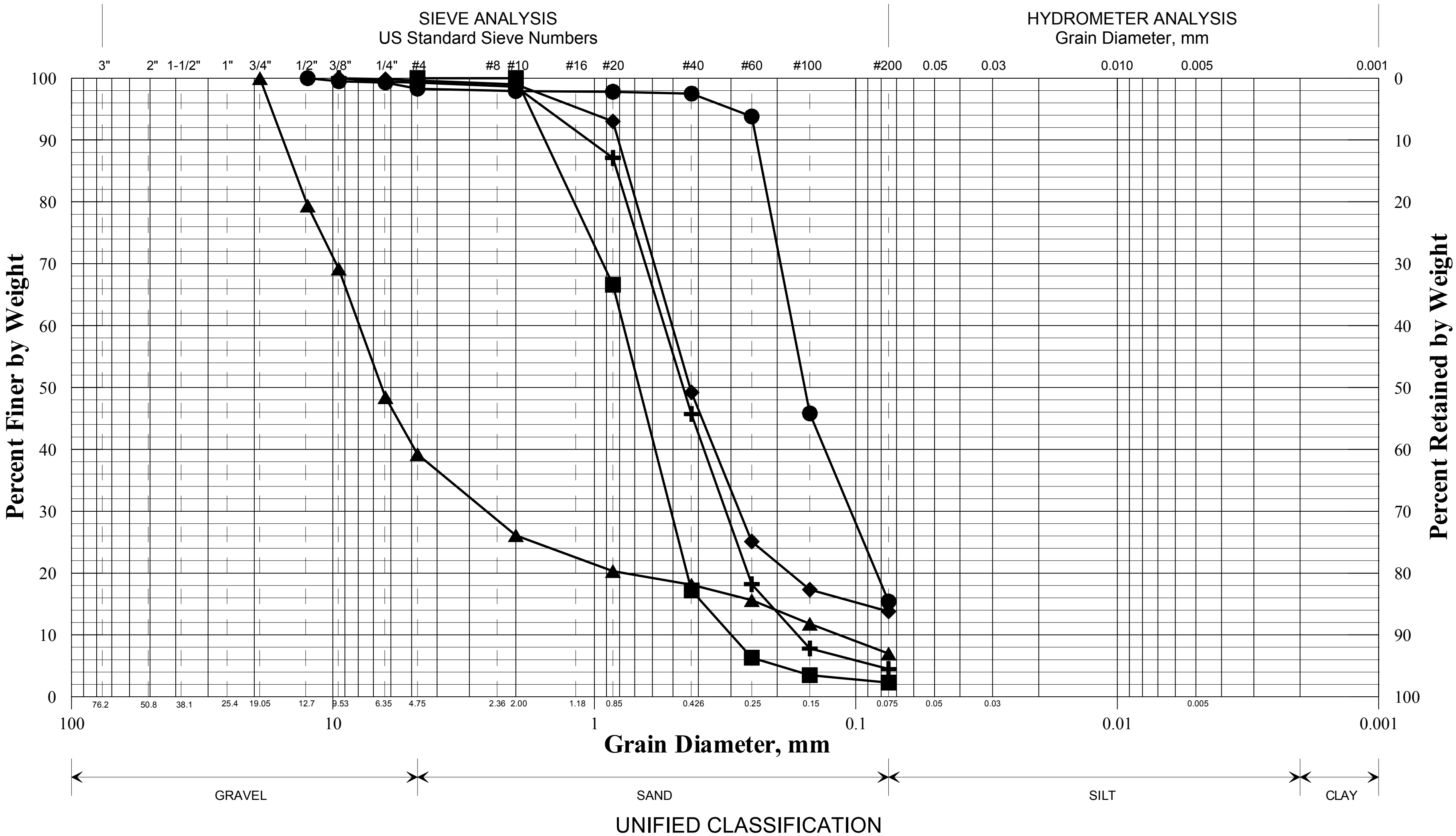
SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-103/4D	35+11.1	33.3 LT	15.0-17.0	SAND, trace silt, trace gravel.	17.9			
◆	BB-LBAR-103/5D	35+11.1	33.3 LT	20.0-22.0	SAND, little silt, trace gravel.	9.1			
■	BB-LBAR-103/7D	35+11.1	33.3 LT	30.0-32.0	SAND, trace silt.	24.1			
●	BB-LBAR-103/9D	35+11.1	33.3 LT	40.0-42.0	SAND, little silt, trace gravel.	22.6			
▲	BB-LBAR-103/11D	35+11.1	33.3 LT	50.0-52.0	GRAVEL, some sand, trace silt.	5.8			
✖									

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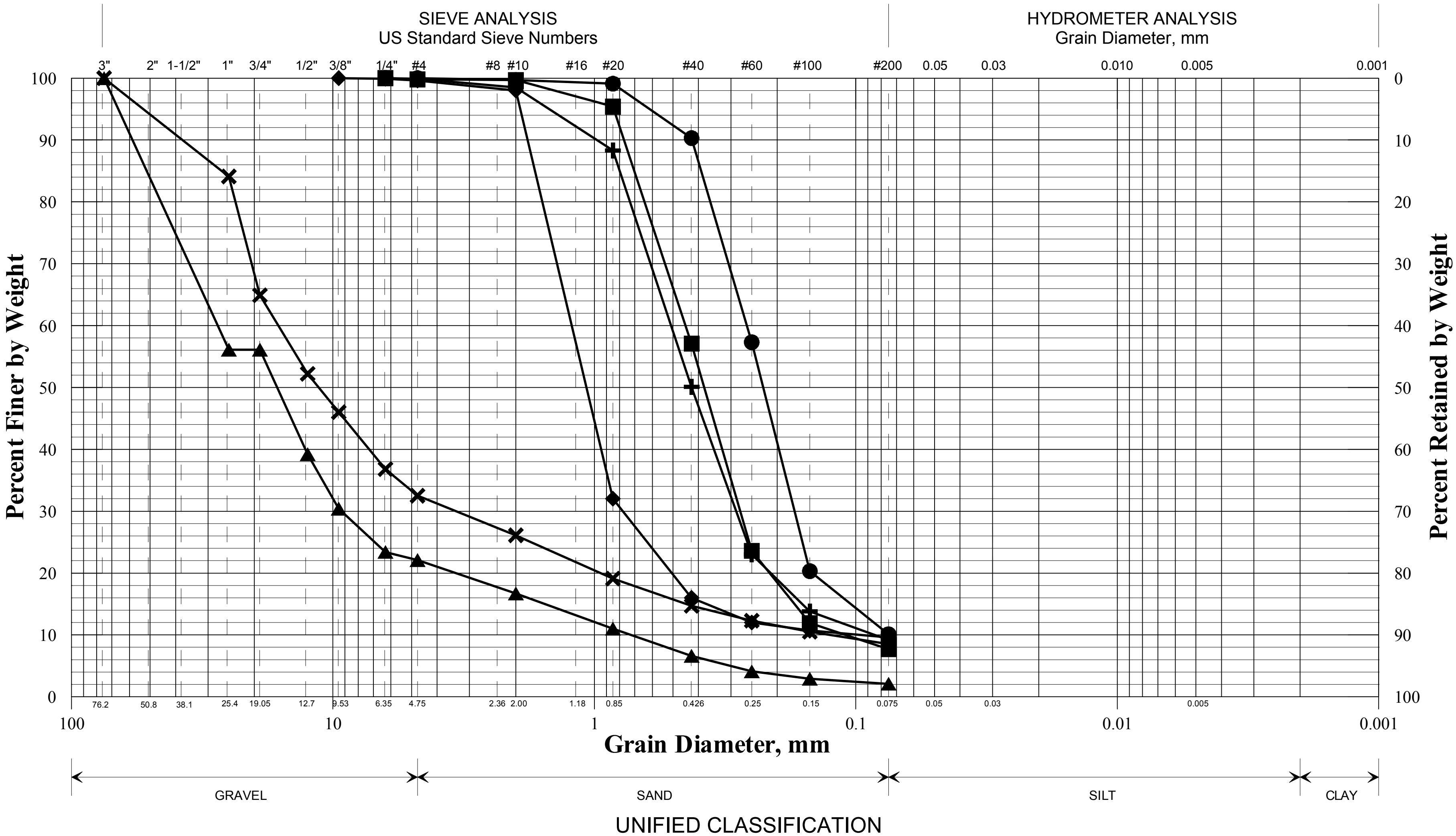
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-104/6D	34+60	16.6 RT	25.0-27.0	SAND, trace silt, trace gravel.	14.9			
◆	BB-LBAR-104/7D	34+60	16.6 RT	30.0-32.0	SAND, trace silt, trace gravel.	13.9			
■	BB-LBAR-104/9D	34+60	16.6 RT	40.0-42.0	SAND, trace silt, trace gravel.	17.1			
●	BB-LBAR-104/11D	34+60	16.6 RT	50.0-52.0	SAND, trace silt.	20.3			
▲	BB-LBAR-104/13D	34+60	16.6 RT	60.0-62.0	GRAVEL, little sand, trace silt.	3.7			
✖	BB-LBAR-104/15D	34+60	16.6 RT	70.0-72.0	GRAVEL, some sand, trace silt.	6.7			

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SHEET 4

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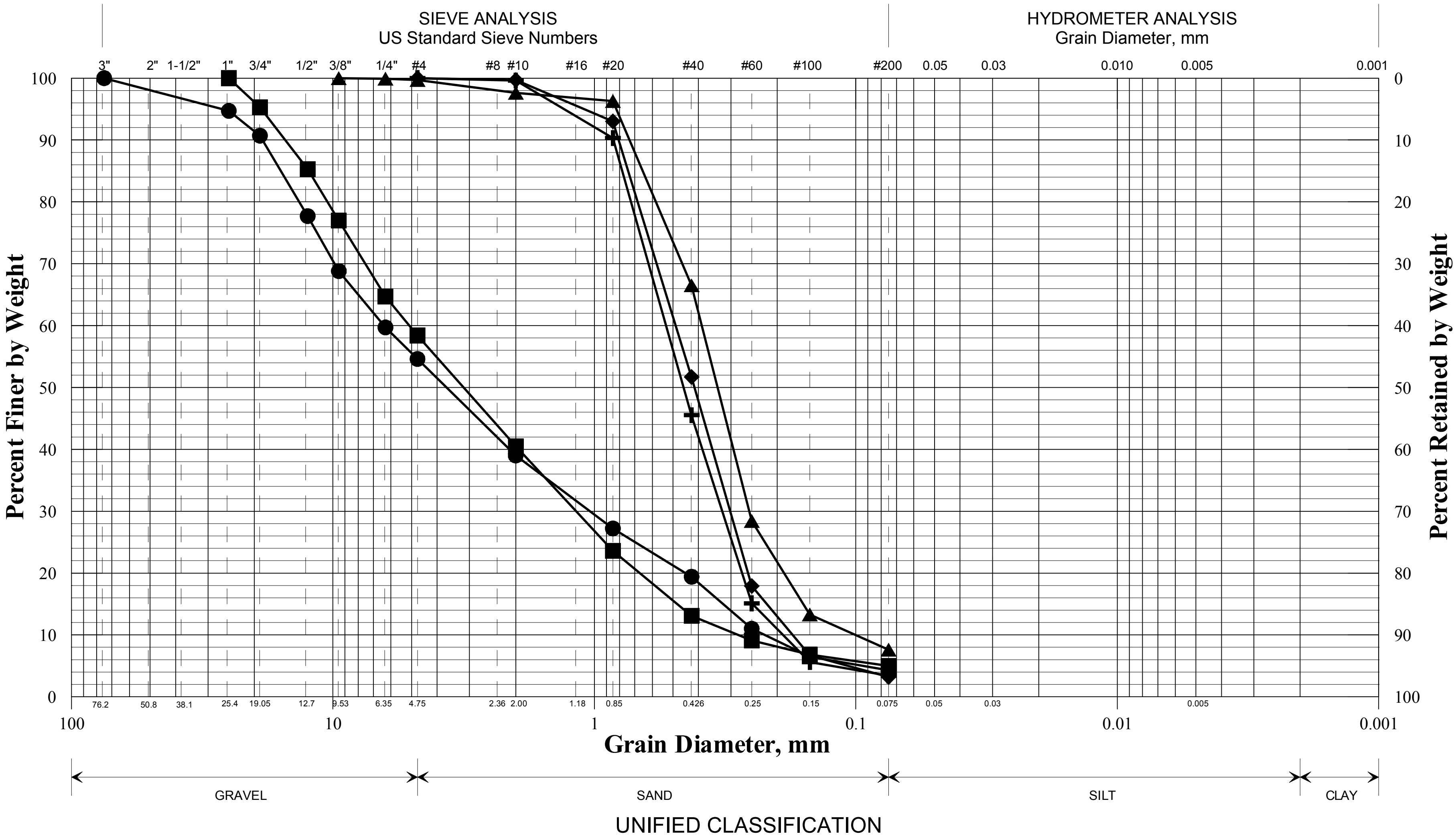
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-105/2D	34+57.3	49.1 LT	5.0-7.0	SAND, trace silt.	3.0			
◆	BB-LBAR-105/3D	34+57.3	49.1 LT	10.0-12.0	SAND, trace silt.	3.8			
■	BB-LBAR-105/5D	34+57.3	49.1 LT	20.0-22.0	Gravelly SAND, trace silt.	8.0			
●	BB-LBAR-105/7D	34+57.3	49.1 LT	30.0-32.0	Gravelly SAND, trace silt.	10.2			
▲	BB-LBAR-105/9D	34+57.3	49.1 LT	40.0-42.0	SAND, trace silt, trace gravel.	21.3			
✖									

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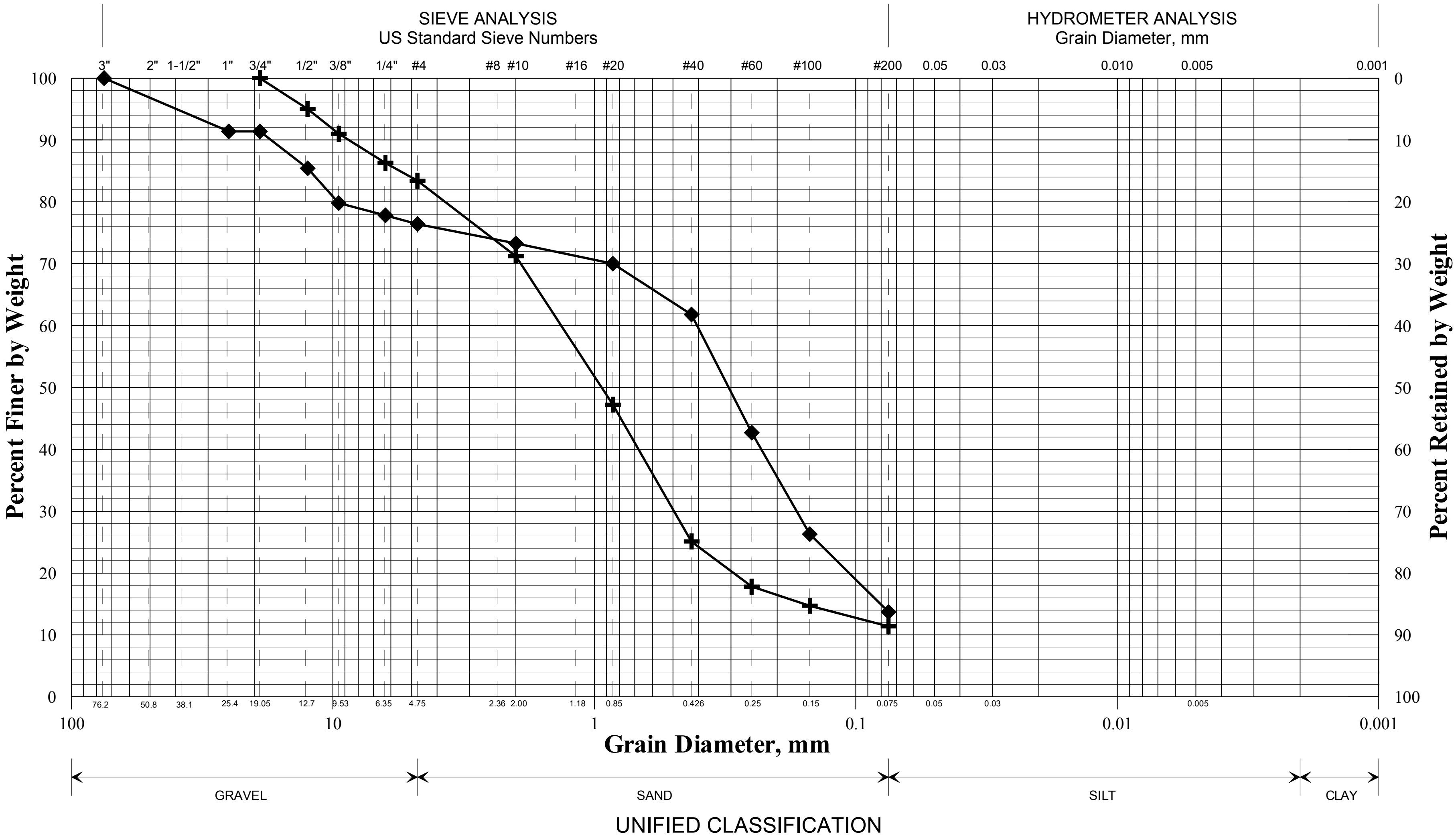
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LBAR-106/2D	33+05	1.3 RT	5.0-7.0	SAND, little gravel, little silt.	6.8			
◆	BB-LBAR-106/4D	33+05	1.3 RT	16.0-17.0	SAND, some gravel, little silt.	8.3			
■									
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SHEET 6

Appendix C

Calculations

ARCH AND RETAINING WALL ACTIVE AND PASSIVE EARTH PRESSURE:

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

For Lagrange, arch has little skin friction and all soil is granular so very little cohesion. These conditions are consistent with Rankine Earth Pressure Theory. So use Rankine coefficients.

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

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

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

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

$K_a = 0.31$

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

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

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

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

$K_{p\_rank} = 3.25$

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

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

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

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

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

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

$K_p = 6.89$

At-Rest Earth Pressure

Effective friction angle of soil $\phi_f := 32 \cdot \text{deg}$

$K_o := 1 - \sin(\phi_f)$ $K_o = 0.47$

Lagrange
Preliminary Thrust Block Force Summary

Footing Properties

B 8.00 ft x-axis is parallel to CL Brg

Applied Loads (kip, kip-ft) - Unfactored (From MathCAD Sheet)

Load	Fy	Mx
EH (At-Rest)	0.00	69.60
DC (Self Weight)	9.22	1.40
EV	12.33	-22.50
DC*	2.27	-6.70
DW*	0.85	-2.00
LL*	4.03	-17.10
E*	20.80	-34.70
Total	49.50	-12.00

*Given arch reaction

Service I Bearing Pressures I (KSF)

Load	Fy	Mx	Load Factor	
EH (At-Rest)	0.00	69.60	**1.00**	
DC (Self Weight)	9.22	1.40	**1.00**	
EV	12.33	-22.50	**1.00**	
DC*	2.27	-6.70	**1.00**	
DW*	0.85	-2.00	**1.00**	
LL*	3.34	-14.19	**0.83**	**
E*	20.80	-34.70	**1.00**	

**Factor is proportion by HL-93/MDOT LL Ratio

Total Pressures

Toe

6.40 ksf

Eccentricity: 0.19 ft

B': 7.63 ft

Strength I Bearing Pressures (KSF)

Load	Fy	Mz	Load Factor
EH (At-Rest)	0.00	93.96	**1.35**
DC (Self Weight)	11.53	1.75	**1.25**
EV	16.65	-30.38	**1.35**
DC*	2.84	-8.38	**1.25**
DW*	1.28	-3.00	**1.50**
LL*	7.05	-29.93	**1.75**
E*	28.08	-46.85	**1.35**

Total Pressures

Toe

9.21 ksf

Eccentricity: 0.34 ft

B': 7.32 ft

Note: Pressures calculated as sum(Fy)/B'

Lagrange
Preliminary Thrust Block Force Summary

Sliding Friction Reduction Factor
ϕ_τ 0.80

x-axis is parallel to CL Brg
z-axis is normal to CL Brg (positive is toward trail)

Applied Loads (kip, kip-ft) - Unfactored (From MathCAD Sheet)

Load	Fx	Fz	Fy
EH (At-Rest)	0.00	14.34	0.00
DC (Self Weight)	0.00	0.00	9.22
EV	0.00	0.00	12.33
DC*	-1.20	-0.80	2.27
DW*	-0.38	-0.25	0.85
LL*	-3.81	-2.53	4.03
E*	-7.20	-4.79	20.80
Total	-12.59	5.97	49.50

*Given arch reaction

Factored Loads - No LL

Load	Fx	Fz	Fy	Load Factor
EH (At-Rest)	0.00	19.36	0.00	1.35
DC (Self Weight)	0.00	0.00	8.30	0.90
EV	0.00	0.00	12.33	1.00
DC*	-1.08	-0.72	2.04	0.90
DW*	-0.24	-0.16	0.55	0.65
LL*	0.00	0.00	0.00	0.00
E*	-7.20	-4.79	20.80	1.00
Total	-8.5	13.7	44.0	

*Given arch reaction

Reqired Nominal Friction Coefficient, μ

0.46

Factored Loads

Load	Fx	Fz	Fy	Load Factor
EH (At-Rest)	0.00	19.36	0.00	1.35
DC (Self Weight)	0.00	0.00	8.30	0.90
EV	0.00	0.00	12.33	1.00
DC*	-1.08	-0.72	2.04	0.90
DW*	-0.24	-0.16	0.55	0.65
LL*	-6.66	-4.43	7.05	1.75
E*	-7.20	-4.79	20.80	1.00
Total	-15.2	9.3	51.1	

*Given arch reaction

Reqired Nominal Friction Coefficient, μ

0.44

ARCH SPREAD FOOTING BEARING RESISTANCE ON COMPACTED FILL OR GLACIAL STREAM SOILS:

Consider this for use with Arch Spread Footings.

Reference Preliminary Thrust Block Force Summary by TY Lin Above

SERVICE LIMIT STATE:

Method 1:

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

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommended Value
Coarse to Medium sand, little gravel	Very dense	8 to 12	8 ksf
	Medium dense to dense	4 to 8	6 ksf
	Loose	2 to 4	3 ksf

By this method, Use **6.0 ksf** to control settlements for **Service Limit State** analyses and for preliminary footing sizing.

Method 2:

Evaluate service limit state bearing resistance for the allowable settlement specified below using SPT Method by Burland and Burbridge (1985) provided in Terzaghi, Peck and Mesri, pg 396
Assume Limit Settlement S to 1/2-inch

From Borings BB-LBAR 102 through BB-LBAR-105, N_{60} at Footing Bearing Level: 16, 42, 51, 11

W/O Low and High => (16+42)/2=29, This Average Value

$N_{60} := 29$

$S := 0.5in$ Assume Limit Settlement S to 1/2-inch and convert to mm for Equation 50.9 below

$S_{mm} := S \cdot \frac{1}{1mm}$ $S_{mm} = 12.7$

Range of Effective Footing Widths B' in Feet and Meters. Note value of B' = 7.32 ft and 7.63 ft provided by TY Lin from their Strength and Service Limit State Calculations.

$$B' := \begin{pmatrix} 4 \\ 6 \\ 7.32 \\ 7.63 \\ 8 \\ 10 \end{pmatrix} \cdot ft \qquad B'_m := B' \cdot \frac{1}{1m} \qquad B'_m = \begin{pmatrix} 1.22 \\ 1.83 \\ 2.23 \\ 2.33 \\ 2.44 \\ 3.05 \end{pmatrix}$$

Correction Factor to increase strip footing compared to square footing with same bearing pressure, Terzaghi Eq. 50.14, pg 397

$$f_{cor} := 1.56$$

Equation 50.9 Terzaghi, Peck, Mesri, pg 396

$$q_{service} := S_{mm} \cdot (N_{60})^{1.4} \cdot \frac{f_{cor} \cdot kPa}{(1.7) \cdot (B'_m)^{0.75}}$$

$$q_{service} = \begin{pmatrix} 23.4 \\ 17.3 \\ 14.9 \\ 14.4 \\ 13.9 \\ 11.8 \end{pmatrix} \cdot ksf \qquad \text{For} \qquad B' := \begin{pmatrix} 4 \\ 6 \\ 7.32 \\ 7.63 \\ 8 \\ 10 \end{pmatrix} \cdot ft$$

For up to B' = 8.0 feet, Use Report Recommendation of **13.9 ksf** to control settlements for **Service Limit State** analyses and for preliminary footing sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for spread footings on glacial stream deposit or compacted fill at the Strength Limit State:

Assumptions:

1. Footings will be embedded 6.5 feet for frost protection. Although footings are also deeply embedded in embankment, the shallow soil between footing and inside arch finish grade is limiting factor.

$$D_f := 6.5ft$$

2. Assumed parameters for soils:
   Assume Glacial Stream Deposits

Moist unit weight:	$\gamma_m := 120pcf$
Saturated unit weight:	$\gamma_{sat} := 130pcf$
Soil angle of internal friction:	$\phi_{ns} := 32$
Undrained shear strength (cohesion):	$c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w \geq 10ft$

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

Effective Stress at the footing bearing level: $q_{eff\_str} := 6.5ft \cdot \gamma_m$

$q_{eff\_str} = 0.78 \cdot ksf$

Assume footing width:

$$B := \begin{pmatrix} 4 \\ 6 \\ 7.32 \\ 7.63 \\ 8 \\ 10 \end{pmatrix} ft$$

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

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 32 deg — Bowles 5th Ed. Table 4-4 pg. 223

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

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

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

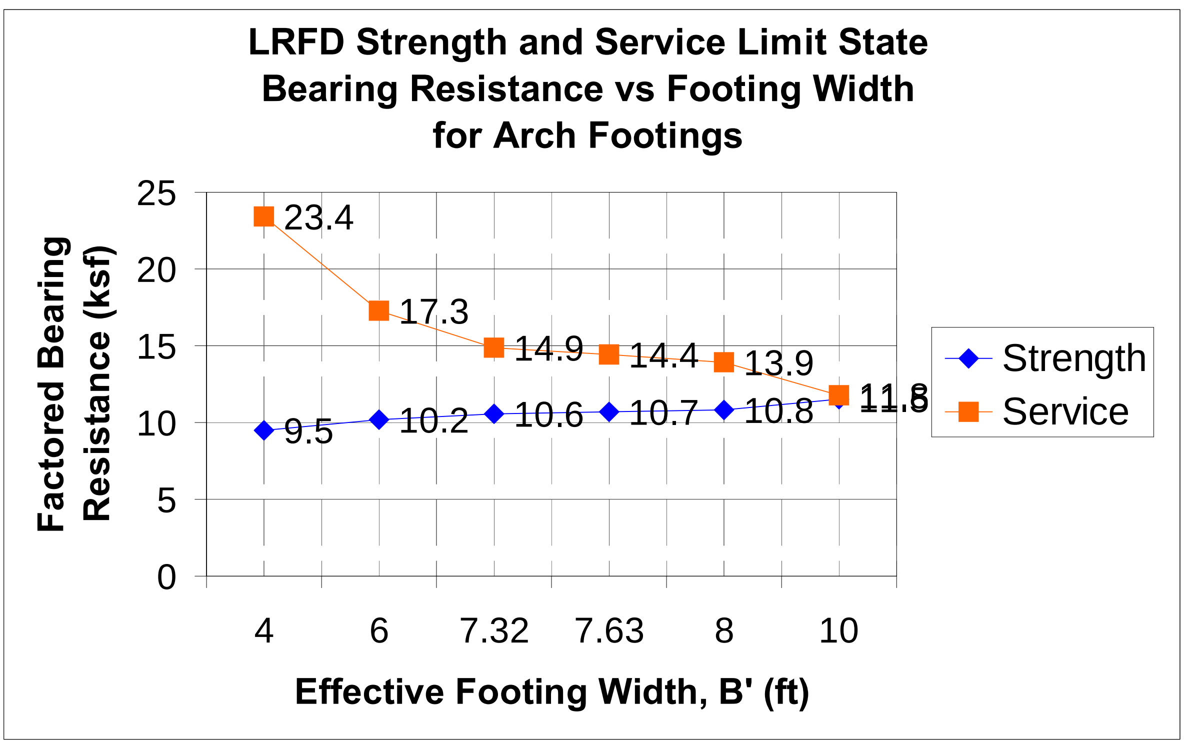
$$q_{nom} = \begin{pmatrix} 21.1 \\ 22.6 \\ 23.5 \\ 23.8 \\ 24 \\ 25.5 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32: $\phi_b := 0.45$

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

$$q_{fac} = \begin{pmatrix} 9.5 \\ 10.2 \\ 10.6 \\ 10.7 \\ 10.8 \\ 11.5 \end{pmatrix} \cdot ksf \quad \text{For} \quad B := \begin{pmatrix} 4 \\ 6 \\ 7.32 \\ 7.63 \\ 8 \\ 10 \end{pmatrix} ft$$

The **Strength Limit State** Factored Bearing Resistances for Arch Footings 4-10 feet wide.



MSE WALL BASE BEARING RESISTANCE ON COMPACTED FILL OR GLACIAL STREAM SOILS:

Consider this for use with Arch Headwalls and Retaining Walls.

SERVICE LIMIT STATE:

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

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommended Value
Coarse to Medium sand, little gravel	Very dense	8 to 12	8 ksf
	Medium dense to dense	4 to 8	6 ksf
	Loose	2 to 4	3 ksf

Recommend **6.0 ksf** to control settlements for **Service Limit State** analyses and for preliminary MSE Wall Base sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for MSE WAall Bases on glacial stream deposit or compacted fill at the Strength Limit State:

Assumptions:

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

$D_f := 6.5ft$

2. Assumed parameters for soils:
   Assume Glacial Stream Deposits

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

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

   Soil angle of internal friction: $\phi_{ns} := 32$

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

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w \geq 10ft$

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

Effective Stress at the footing bearing level: $q_{eff\_str} := 6.5ft \cdot \gamma_m$

$q_{eff\_str} = 0.78 \cdot ksf$

Assume footing width:

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$$

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

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For $\phi = 32$ deg   Bowles 5th Ed. Table 4-4 pg. 223

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

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

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

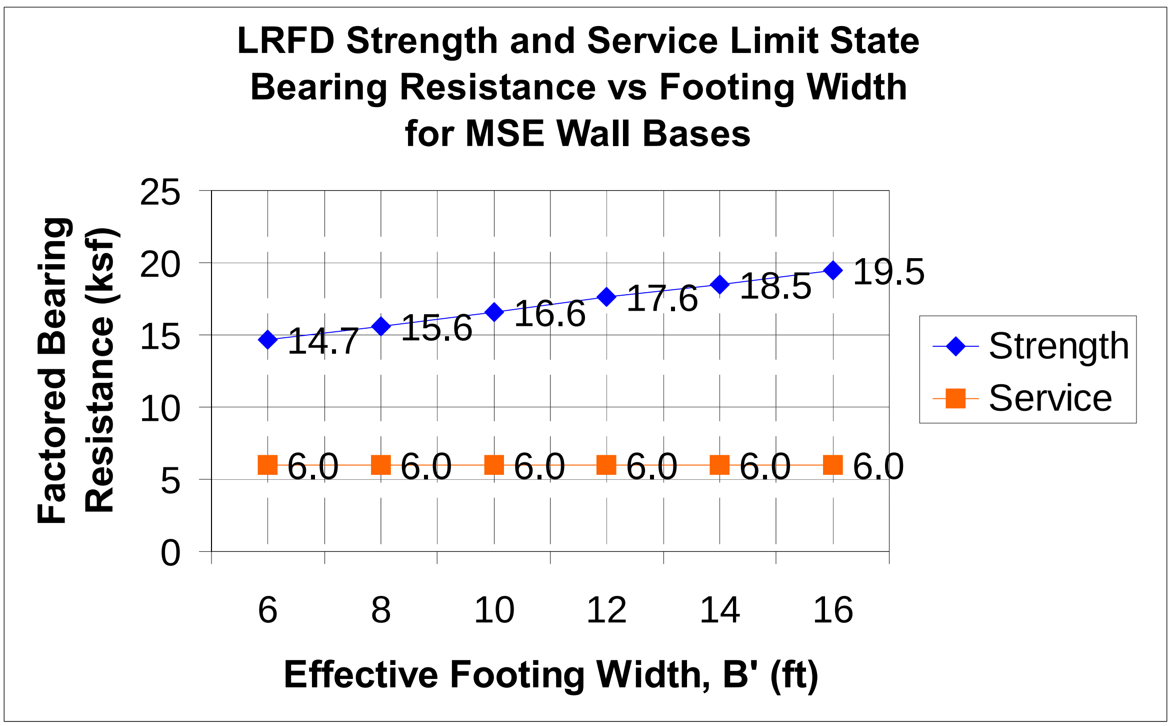
$$q_{nom} = \begin{pmatrix} 22.6 \\ 24 \\ 25.5 \\ 27 \\ 28.5 \\ 30 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 11.5.6-1 pg. 11-11: $\phi_b := 0.65$

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

$$q_{fac} = \begin{pmatrix} 14.7 \\ 15.6 \\ 16.6 \\ 17.6 \\ 18.5 \\ 19.5 \end{pmatrix} \cdot ksf \qquad \text{For} \qquad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} ft$$

The **Strength Limit State** Factored Bearing Resistances for MSE Wall Bases 6-16 feet wide.



EMBANKMENT SETTLEMENT ANALYSIS:

Determine the Stress-Strain Modulus $E_{s,}$ From Eqns in Bowles 5th Ed. p. 316:

$E_s = 500(N+15)$ All These in kPa,
$E_s = 7000\sqrt{N}$ Divide by 50 to Obtain ksf
$E_s = 6000N$

From Borings BB-LBAR 102 through BB-LBAR-105, N_{60} at Footing Bearing Level:
16, 42, 51, 11

W/O Low and High => (16+42)/2=29 Also Look at Low Side Value N=16

$$E_{s16.1} := \frac{[500\text{kPa}\cdot(16+15)]}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s16.1} = 310\cdot\text{ksf} \qquad E_{s29.1} := \frac{[500\text{kPa}\cdot(29+15)]}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s29.1} = 440\cdot\text{ksf}$$

$$E_{s16.2} := \frac{\left(7000\text{kPa}\cdot\sqrt{16}\right)}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s16.2} = 560\cdot\text{ksf} \qquad E_{s29.2} := \frac{\left(7000\text{kPa}\cdot\sqrt{29}\right)}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s29.2} = 753.92\cdot\text{ksf}$$

$$E_{s16.3} := 6000\text{kPa}\cdot\frac{16}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s16.3} = 1920\cdot\text{ksf} \qquad E_{s29.3} := 6000\text{kPa}\cdot\frac{29}{50\frac{\text{kPa}}{\text{ksf}}} \qquad E_{s29.3} = 3480\cdot\text{ksf}$$

Winterkorn & Fang, Foundation Engineering Handbook 1st Ed. P. 567:

E_s in Loose Sand - 200 to 500 ksf
E_s in Dense Sand - 1000 to 1600 ksf

We are in Medium Dense to Dense Sand and Gravel and Considering All Above Values, Conservative to Use E_s = 800 ksf in FOSSA Settlement Analysis

FoSSA -- Foundation Stress & Settlement Analysis

B&A Overhead Bridge

B&A Overhead Bridge

PROJECT IDENTIFICATION

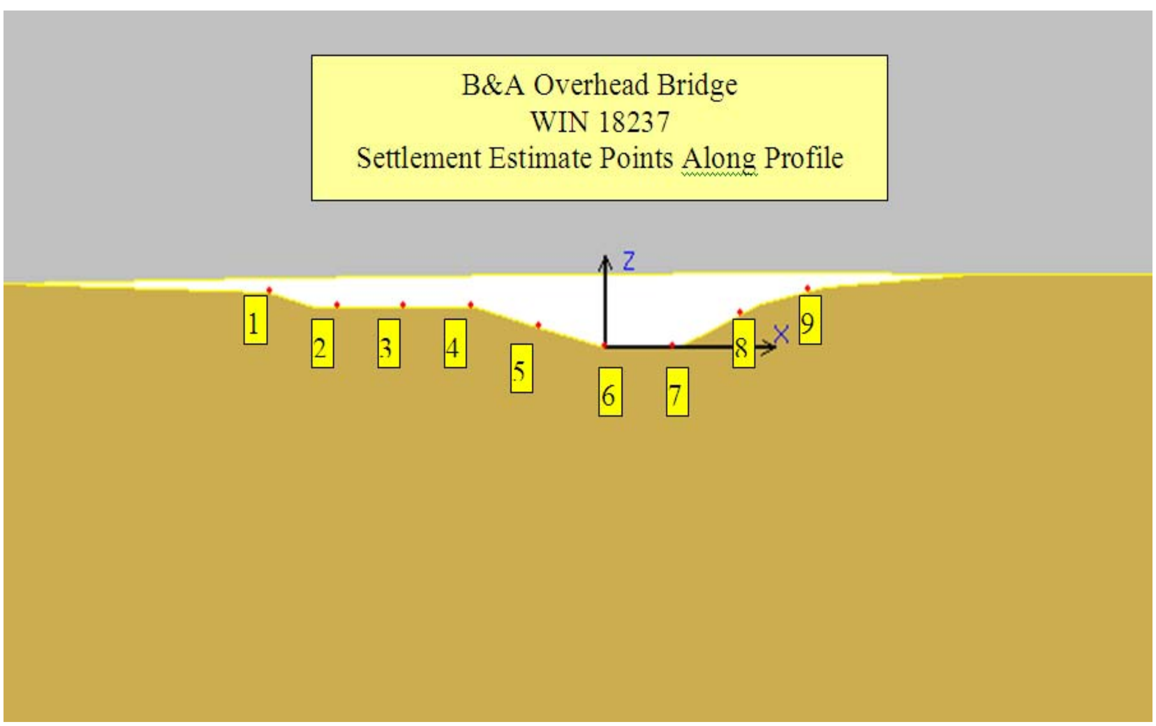
Title:	B&A Overhead Bridge
Project Number:	WIN 18237 -
Client:	MaineDOT
Designer:	Mike Moreau, PE
Station Number:	

Description: Estimate Settlement for Filling Railroad Cut

Company's information:

Name:	MaineDOT
Street:	16 State House Station Augusta, ME 04333-0016
Telephone #:	
Fax #:	
E-Mail:	

Original file path and name: C:\FoSSA\18237 Lagrange B&A Overhead Bridge.F2S
Original date and time of creating this file: Fri Mar 02 11:43:08 2012



IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section: X [ft.]	Y [ft.]	Layer #	Young's Modulus, E [lb/ft²]	Poisson's Ratio, μ	Si(k) [ft.]	Z initial [ft.]	Z final [ft.]
1	200.00	0.00	1	800000	0.3000	0.0292	327.00	326.97
2	225.00	0.00	1	800000	0.3000	0.0464	322.00	321.95
3	250.00	0.00	1	800000	0.3000	0.0521	322.00	321.95
4	275.00	0.00	1	800000	0.3000	0.0618	322.00	321.94
5	300.00	0.00	1	800000	0.3000	0.0837	315.00	314.92
6	325.00	0.00	1	800000	0.3000	0.0925	308.00	307.91
7	350.00	0.00	1	800000	0.3000	0.0902	308.00	307.91
8	375.00	0.00	1	800000	0.3000	0.0750	319.00	318.92
9	400.00	0.00	1	800000	0.3000	0.0379	327.25	327.21

Settlement Inches
Pt. 1 - 0.35
Pt. 2 - 0.56
Pt. 3 - 0.63
Pt. 4 - 0.74
Pt. 5 - 1.00
Pt. 6 - 1.11
Pt. 7 - 1.08
Pt. 8 - 0.90
Pt. 9 - 0.45

INPUT DATA -- FOUNDATION LAYERS -- 1 layers

	Wet Unit Weight, γ [lb/ft³]	Poisson's Ratio μ	Description of Soil
1	125.00	0.30	F-C Sand OR F-C Sand and Gravel

INPUT DATA -- EMBANKMENT LAYERS -- 1 layers

	Wet Unit Weight, γ [lb/ft³]	Description of Soil
1	125.00	Sand and Gravel Fill

TABULTAED GEOMETRY INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	100.00	330.00	F-C Sand OR F-C Sand and Gravel
	2	200.00	327.00	
	3	217.00	322.00	
	4	277.00	322.00	
	5	300.00	315.00	
	6	325.00	308.00	
	7	352.00	308.00	
	8	367.00	315.00	
	9	383.00	323.00	
	10	407.00	329.00	
	11	462.00	333.00	

TABULTAED GEOMETRY INPUT OF EMBANKMENT SOILS

Embank. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	300.00	333.00	Sand and Gravel Fill
	2	400.00	334.00	
	3	3779.27	334.00	

Settlement as a result of embankment construction over natural soils will be on the order of 1-inch or less and will occur during construction. Post-construction settlement will be negligible.

FROST PROTECTION

Method 1:

From the Maine Design Freezing Index Map:
DFI = 1910 degree-days
Any Foundations Will Be Backfilled With Coarse-Grained Soils
Foundations are deep in Embankment and Considder Wet Periods of Year. Also Consider Low Frost Susceptibility. Assume Wn = 20%;

From the 2003 Bridge Design Guide Table 5-1:

$$\text{Frost\_depth} := [0.1\cdot(78.7\text{in} - 76.6\text{in}) + 76.6\text{in}]$$

$$\text{Frost\_depth} = 76.81\cdot\text{in}$$

$$\text{Frost\_depth} = 6.4\cdot\text{ft}$$

Method 2:

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

 Project Location: Orono, Maine

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

 ---------------------------------------------------------
 Layer
 #:Type            t    w%     d    Cf   Cu    Kf    Ku      L
 ---------------------------------------------------------
 1-Coarse        81.1 20.0 125.0   34   46   3.8   1.9  3,600
 ---------------------------------------------------------

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

 *********************************************************
   Total Depth of Frost Penetration = 6.76 ft = 81.1 in.
 *********************************************************
```

Use 6.5 feet

<u>Appendix D</u>

Special Provisions

Lagrange
B&A Overhead Bridge
WIN 18237.00
April 25, 2012

SPECIAL PROVISION
SECTION 203
CRUSHED STONE

Description This work shall consist of constructing a leveling pad of crushed stone in accordance with these specifications and in reasonably close conformity with the width, grade and thickness shown on the plans or established by the Resident.

MATERIALS

Aggregate Crushed stone material shall meet the requirements of ASTM Standard Specification C33, Standard Specification for Concrete Aggregates.

The aggregate shall meet the following gradation requirements:

Particle size	Percent by Weight Passing
1 inch	100
¾ inch	90 – 100
½ inch	20 – 55
⅜ inch	0 – 15
No. 4	0 - 5

Construction Requirements The crushed stone shall be placed and graded as shown on the plans or as directed by the Resident. The crushed stone shall be compacted as required to ensure that all voids in the stone are filled, as approved by the Resident.

Method of Measurement Aggregate for crushed stone will be measured by the cubic yard complete in place.

Basis of Payment The accepted quantity of crushed stone will be paid for at the contract unit price per cubic yard of aggregate complete in place.

Payment will be under

Pay Item	Unit
203.35 Crushed Stone	Cubic Yard

SPECIAL PROVISION
SECTION 636
MECHANICALLY STABILIZED EARTH RETAINING WALL

The following replaces Standard Specification Section 636 in its entirety:

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. The MSE headwall over the arch shall be constructed using galvanized wire mesh facing. A cast-in-place concrete facing shall be constructed in front of the MSE wall conforming to the requirements of Standard Specification 502, Structural Concrete. 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.

<u>636.03 Design Requirements</u> 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-10-024, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, November 2009,
7. FHWA-NHI-10-025, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, November 2009,
8. FHWA-NHI-09-087, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009.

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 4.0 ft 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 in.

The MSE walls shall be dimensioned so that the factored bearing 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) <u>Failure Plane</u> 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.

<u>(b) External Stability - Load and Resistance Factors</u> 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.

<u>(c) Internal Stability - Load and Resistance Factors</u> 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 8 ft, but shall not be less than 70 percent of the wall height, H, for walls with level surcharges, 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:

	mils/year/side
Zinc galvanizing (first 2 years)	0.58
Zinc galvanizing (subsequent years to depletion):	0.16
Carbon Steel (after zinc depletion to 100 yrs):	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 11.10, and shall be checked against the nominal tensile strength multiplied by a resistance factor per LRFD Table 11.5.7-1. Transverse and longitudinal grid members shall be sized in accordance with ASTM A185/A158M.

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 in and 3.5 in elsewhere. The minimum concrete cover shall be 1.5 in. 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 ft.

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 in. 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 in. All uncoated steel projecting from the panel unit shall be galvanized in accordance with ASTM A123/A123M (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 in.

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

636.043 Reinforcing and Wire Mesh Facing All reinforcing, wire mesh facing, 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 and Wire Mesh Facing shall be shop fabricated from cold drawn steel wire conforming to the requirements of AASHTO M 32 (ASTM A82/A82M) yield strength minimum of 65 ksi and shall be welded into the finished mesh fabric in accordance with AASHTO M 55 (ASTM A185/A185M). Galvanizing shall be in accordance with AASHTO M 111 (ASTM A123/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 A572/A572M) Grade 65, or approved equal. Reinforcing strips shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A123/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 and reinforcement connectors shall be fabricated of hot rolled steel conforming to ASTM A1011/A1011M, Grade 50 or equivalent. Tie strips and reinforcement connectors shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A123/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 A510, UNS G 10350 or AASHTO M 32 (ASTM A82/A82M). Loop embeds shall be welded in accordance with AASHTO M 55 (ASTM A185/A185M). Both shall have electrodeposited coatings of zinc applied in accordance with ASTM B633.

B. Fasteners shall consist of hexagonal cap screw bolts and nuts, which are galvanized and conform to the requirements of AASHTO M 164 (ASTM A325) or equivalent.

C. Connector pins and mat bars shall be fabricated from AASHTO M 183 (ASTM A36/A36M) steel and welded to the soil reinforcement mats as shown on the plans. Galvanization shall conform to AASHTO M111 (ASTM A123/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 A82/A82M (AASHTO M 32) and galvanized in accordance with ASTM A123/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 636.043.

<u>636.045 Joint Materials</u> 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 D2000 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 D1505

<u>636.046 Nonwoven Drainage Geotextile</u> 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 in. Lap fabric at least 12 in. 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.

Soil Retention Fabric as supplied by the MSE Wall supplier shall be used behind the welded wire mesh facing to retain the soil over the arch. This material has a temporary function until the cast-in-place concrete facing is placed.

<u>636.047 Concrete Leveling Pad</u> The cast-in-place leveling pad shall be constructed of Class A concrete conforming to the requirements of Section 502 - Structural Concrete. Leveling

pad shall have minimum dimensions of 6 in thickness and 12 in width and be placed at the design elevation shown on the shop drawings within a 1/8 in 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 the following additional requirements:

A. The maximum aggregate size is limited to 4 in (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 C88), 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 a peak angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T 236 (ASTM D3080-72), on the portion finer than the 2 mm [#10 sieve], compacted to 95 percent of AASHTO T 99, 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 in. (19 mm) 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.

636.049 Crushed Stone for Abutment Foundation Crushed stone for use in the foundation layer below the abutment shall be crushed stone conforming to the requirements of MaineDOT Standard Specification Section 703.31.

636.050 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 ft beyond the MSE reinforcement. The geomembrane shall have a minimum thickness of 30 mil (0.003 in)

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.

636.051 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 ft 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 ft, 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 703.20, Gravel Borrow. 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 ft and -0.02 ft 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 and welded wire facing units 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 ft straightedge (¼ in/yd). During construction, the maximum allowable offset in any panel joint shall be ¾ in. The overall vertical tolerance of the wall (from top to bottom) shall not exceed ½ inch per 10 ft 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 T 180, 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 19 mm [3/4 in] in size, AASHTO T 180 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 ft below the bottom of footing elevation shall be compacted to 98 percent of the maximum density as determined by AASHTO T 180, Method C or D (with oversize correction, as outlined in Note 7 of that test).

The moisture content (determined in accordance with AASHTO T 180, Method C or D) of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer. Backfill materials shall 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 in. 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 ft of the wall face. Compaction within 3 ft 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 ft 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 or arch deck 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.

<u>636.14 Basis of Payment</u> 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, welded wire facing units, fasteners, reinforcing mesh, reinforcing strips, tie strips, hardware, joint fillers, coping, woven drainage geotextile, soil retention fabric, impervious membrane, select granular backfill, excavation to subgrade 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.

Any 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 in the reinforced zone 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, welded wire facing units, MSE reinforcing elements, attachment devices, fasteners, bearing blocks and shims, joint materials, soil retention fabric, 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
636.40	Mechanically Stabilized Earth Retaining Wall	Square foot