MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

# GEOTECHNICAL DESIGN REPORT

*For the Replacement of:*

**LOWER SANDY STREAM BRIDGE**
**OVER SANDY STREAM**
**LEXINGTON TOWNSHIP, MAINE**


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## GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Lower Sandy Steam Bridge over Sandy Stream in Lexington Township, Maine. The replacement structure will consist of a single-span, steel superstructure founded on H-pile supported integral abutments constructed behind the location of the existing abutments. The existing abutments will be removed down to the Q1.1 elevation and sheet piling will be driven behind the portion of the abutment to remain for scour protection. The following design recommendations are discussed in detail in the attached report:

**Integral Abutment H-Piles** – The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. The H-piles shall be design for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral, and flexural resistance. An L-Pile® analysis is recommended to evaluate the combined axial compression and flexure with factored axial loads, moments and pile head displacements applied. As the proposed integral H-piles will be modeled as fully fixed at the pile head, the resistance of the piles should be evaluated for structural compliance with the interaction equation.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, $\varphi_{dyn}$, of 0.65. The maximum factored axial pile load should be shown on the plans.

**Integral Stub Abutments** – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, $K_p$, of 3.25 anticipating that integral abutments will experience some movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.005, then the calculation of lateral earth pressure should assume a Coulomb passive earth pressure coefficient, $K_p$, of 6.89. All abutment designs shall include a drainage system to intercept any water. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted.

**Sheet Pile Walls** - The existing abutments will be removed down to the Q1.1 elevation and a permanent sheet pile wall will be installed behind the portion of the abutments to remain for scour protection. A riprap slope will be constructed between the proposed abutments and the sheet pile walls behind the existing abutments remaining. The sheet pile walls will be designed to support the bridge and roadway embankment in the event that material in front of the existing abutment remaining is scoured away. It is estimated that the sheet pile walls will have a length of approximately 37 feet. The sheet pile walls shall be designed to withstand lateral earth pressures. Uncoated sheet piles are permitted. The selected sheet pile section should consider a sacrificial steel loss. The use of hot-rolled sheets is recommended.

**Prefabricated Concrete Modular Block Gravity Wall** – The use of a Precast Concrete Modular Gravity (PCMG) wall is proposed on the downstream south corner of Abutment No. 1 to retain the roadway section and minimize impacts. Precast Concrete Modular Gravity (PCMG) walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be embedded for frost protection and designed in accordance with LRFD and Special Provision 635.

**Scour and Riprap** – The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. For scour protection and protection of pile groups and PCMG walls, the bridge approach slopes and slopes at abutments should be armored with 3 feet of plain riprap. For scour protection of the bridge approaches, permanent sheet pile walls will be installed in front of the new abutments as detailed above. The riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

**Settlement** - The roadway profile will be raised approximately 1.2 feet at the abutments. Potential settlement due the placement of the proposed fill is estimated to be approximately 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill will occur during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

**Frost Protection** - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Foundations for PCMG walls placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

**Seismic Design Considerations** – A seismic analysis is not required for single-span bridges regardless of seismic zone. The Lower Sandy Stream Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criteria eliminates the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

**Construction Considerations** – Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. Using the excavated native soils as structural backfill should not be permitted. Materials excavated from the existing subbase and subgrade fill soils in approaches should not be used to re-base the new bridge approaches.

A layer of wood was encountered in the area of proposed Abutment No. 1 and wood fragments were sampled in the fills at proposed Abutment No. 2.. It is likely that the presence of wood at either abutment will impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations, installation of sheet piles for cofferdams and installation of permanent sheet piles for scour protection. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident. The potential for obstructions to slow construction activities should be considered by the Contractor.

## 1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of the Lower Sandy Stream Bridge over Sandy Stream in Lexington Township, Maine. A subsurface investigation at the site has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing Lower Sandy Stream Bridge carries Long Falls Dam Road over Sandy Stream and was constructed in 1929. The bridge consists of an approximately 75 foot long, single span through-girder structure with painted steel girders and floor beams. The bridge substructure consists of full height, unreinforced mass concrete abutments and wingwalls supported on timber piles. The pile cap has one layer of reinforcing above the piles. The 2011 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and substructure are in poor condition (rating of 4) and the superstructure is in serious condition (rating of 3). The Bridge Sufficiency Rating is 29.9. The structure has a scour critical rating of “7 – Countermeasures” meaning that countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Inspection records note that the structure is in poor/serious condition with extensive rusting of floor beams at the east ends. There is evidence of abutment scaling and spalling. Scour has also been a serious at the bridge site.

The replacement structure will consist of a single-span, steel superstructure founded on H-pile supported integral abutments constructed behind the location of the existing abutments. The existing abutments will be removed down to the Q1.1 elevation and sheet piling will be driven behind the portion of the abutments to remain for scour protection. The span of the proposed replacement structure will be approximately 108 feet. The proposed horizontal alignment will approximately match the existing alignment. The roadway profile will be raised approximately 1.2 feet at proposed abutments. The proposed bridge will be constructed using a temporary bridge located west of the existing structure.

## 2.0 GEOLOGIC SETTING

Lower Sandy Steam Bridge in Lexington Township carries Long Falls Dam Road over Sandy Stream 4.2 miles north of State Route 16 as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic map entitled New Portland Quadrangle, Maine Open File No. 09-47 (2009) published by the Maine Geological Survey the surficial soils in the vicinity of the site consist of stream alluvium with local contacts to regressive marine delta deposits. Stream alluvium is comprised of sand, gravel, silt and organic sediment deposited on flood plains of modern streams. Regressive marine deltas were deposited during regression of the sea due to isostatic emergence of the land and are characterized by very low angle sands and silt bedding.

According to the Bedrock Geologic Map of Maine (1985) published by the Maine Geologic Survey, the bedrock in the vicinity of the site consists of Devonian muscovite granite known as the Lexington pluton.

## 3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings. Test boring BB-LSS-101 was conducted approximately 20 feet behind Abutment No. 1 (south) and test boring BB-LSS-102 was conducted approximately 20 feet behind Abutment No. 2 (north). The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The borings were drilled between May 2 and 21, 2012 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in March of 2010 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value are shown on the boring logs. Undisturbed tube samples were obtained in the soft soil deposits in boring BB-LSS-102 where possible. In-situ vane shear tests were made where possible in soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated.

Two soil samples were obtained from the streambed in order to develop scour parameters. These samples were obtained by wading into the stream and sampling the streambed using a spade. The samples were placed in jars and transported with the test boring samples to the MaineDOT laboratory for grain size testing.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector or the geotechnical team member logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the exploration programs.

# 4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of seven (7) standard grain size analyses with water content, thirteen (13) grain size analyses with hydrometer and water content and five (5) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 4 and 5 – Boring Logs found at the end of this report.

# 5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of deep deposits of regressive marine delta sands and glaciomarine clays, silts and sands underlain by bedrock. The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the generalized site stratigraphy is shown on Sheet 3 – Interpretive Subsurface Profile both found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

## 5.1 Fill

A layer of fill was encountered beneath the pavement in both of the borings. The fill consisted of:

- Brown, damp to wet, fine to coarse sand, little to some gravel, trace to some silt;
- Brown, damp, gravelly, fine to coarse sand, trace silt;
- Grey, wet, fine sand, trace medium sand, little silt;
- Olive-grey, wet, very soft, silt, some fine sand, trace gravel; and
- Grey, wet, fine to coarse sand, trace silt.

The thickness of the fill was approximately 14.0 feet in boring BB-LSS-101 and approximately 19.0 feet in boring BB-LSS-102. Corrected SPT N-values in the fill ranged from weight of hammer (WOH) to 14 blows per foot (bpf) indicating that the fill is very loose to medium dense in consistency. Water contents obtained from fill samples ranged from approximately 31% to 40%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-2-4 or A-4 by the AASHTO Classification System and an SM or ML by the Unified Soil Classification System.

## 5.2 Stream Alluvium

A layer of reworked stream alluvium was encountered beneath the fill in both of the borings. The stream alluvium consisted of:

- Grey, wet, fine to coarse sand, trace gravel, trace wood and
- Grey, wet, fine to coarse sand, some gravel, trace silt, trace wood fragments.

A 0.7 foot thick layer of wood was encountered at the bottom of the reworked stream alluvium layer in boring BB-LSS-101 and wood fragments were observed within the layer in boring BB-LSS-102. The thickness of the stream alluvium layer was approximately 6.7 feet in boring BB-LSS-101 and approximately 10.0 feet in boring BB-LSS-102. Corrected SPT N-values in the stream alluvium ranged from 7 to 11 bpf indicating that the reworked stream alluvium is loose to medium dense in consistency.

### 5.3 Marine Delta Deposits

A layer of marine delta deposits was encountered beneath the stream alluvium in both of the borings. The layer generally consisted of sand, silty sand and silt and is comprised of:

- Grey, wet, gravelly, fine to coarse sand, trace silt;
- Grey, wet, fine sand, trace medium to coarse sand, trace to little silt, trace clay, trace gravel;
- Grey, wet, fine to medium sand, trace coarse sand, trace to some silt, trace gravel, trace clay;
- Grey, wet, silty fine sand; and
- Grey, wet, silt, little to some fine sand, trace clay, trace medium sand, trace wood fragments.

The thickness of the layer was approximately 55.3 feet in boring BB-LSS-101 and approximately 49.0 feet in boring BB-LSS-102. Corrected SPT N-values in the granular soils encountered in the layer ranged from 4 to 18 indicating that the granular soils are very loose to medium dense in consistency. Corrected SPT N-values in the cohesive soils encountered in the layer ranged from 7 to 13 indicating that the cohesive soils are stiff in consistency. Water contents from samples obtained within the layer range from approximately 20% to 23%. Grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-2-4, A-3 or A-4 by the AASHTO Classification System and an SP-SC, SP-SM, SM, SC-SM, SP, or ML by the Unified Soil Classification System.

### 5.4 Glaciomarine Deposit

Glaciomarine deposits were encountered beneath the marine delta deposits in both of the borings. The glaciomarine deposits consisted of:

- Grey, wet, clayey silt, trace to little fine sand in layers and
- Grey, wet, silt, some clay, trace fine sand.

The thickness of the glaciomarine deposits was approximately 48.0 feet in boring BB-LSS-101 and approximately 33.0 feet in boring BB-LSS-102. Vane shear testing conducted within the glaciomarine deposits showed undrained shear strengths ranging from approximately 402 psf to 1652 psf while the remolded shear strengths ranged from approximately 112 psf to 402 psf. These shear strength values indicate that the undisturbed glaciomarine deposits are soft to stiff in consistency. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the glaciomarine deposits were determined to have sensitivities ranging from approximately 2.6 to 12.4 and is classified as medium sensitive to slightly quick. Water contents from samples obtained within the layer range from approximately 27% to 28%. Grain size analyses conducted on the samples indicate that the soil is classified as an A-4 or A-6 by the AASHTO Classification System and a CL-ML, CL or ML by the Unified Soil Classification System.

Table 5-1 below summarizes the results of the Atterberg Limits tests from samples of the glaciomarine deposits:

| Sample No. | Water Content (%) | Liquid Limit | Plastic Limit | Plasticity Index | Liquidity Index |
|---|---|---|---|---|---|
| BB-LSS-101 17D | 28.0 | 26 | 19 | 7 | 1.29 |
| BB-LSS-101 21D | 27.7 | 29 | 15 | 14 | 0.91 |
| BB-LSS-101 22D | 26.5 | 27 | 21 | 6 | 0.92 |
| BB-LSS-102 17D | 27.9 | 25 | 23 | 2 | 2.45 |
| BB-LSS-102 18D | 27.1 | 28 | 21 | 7 | 0.87 |

**Table 5-1 – Summary of Atterberg Limits Testing Results for Silt Samples**

Interpretation of these results indicates that the soils with liquidity indices of 1 or less are normally consolidated while those with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as "quick".

### 5.5 Outwash Sands

A layer of outwash sand was encountered beneath the glaciomarine deposits. The layer generally consisted of:

- Grey, wet, fine to coarse sand, little to some silt, trace gravel;
- Grey, wet, fine sand, trace silt; and
- Grey, wet, silty fine to medium sand.

The thickness of the layer was approximately 7.4 feet in boring BB-LSS-101 and approximately 16.0 feet in boring BB-LSS-102. Corrected SPT N-values in the outwash sand ranged from weight of hammer to 28 bpf indicating that the outwash sands are very loose to medium dense in consistency. Water contents from samples obtained within the layer ranged from approximately 12% to 15%. Grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-2-4 by the AASHTO Classification System and as an SM by the Unified Soil Classification System.

### 5.6 Glacial Till

A lower layer of glacial till was encountered beneath the outwash sands in boring BB-LSS-102. The glacial till consisted of:

- Grey, wet, gravelly, fine to coarse sand, little silt, with cobbles.

The thickness of the layer was approximately 11.4 feet. Corrected SPT N-values in the glacial till ranged from greater than 50 to 106 bpf indicating that the glacial till is very dense in consistency. Glacial till was not encountered in boring BB-LSS-101.

### 5.7 Bedrock

Bedrock was encountered and cored in both of the borings. The Table 5-1 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD:

| Boring Number | Approximate Depth to Bedrock | Approximate Bedrock Elevation | RQD |
|---|---|---|---|
| BB-LSS-101 | 131.4 feet | 253.0 feet | 100% |
| BB-LSS-102 | 138.4 feet | 245.6 feet | 93% |

**Table 5-2 - Summary of Bedrock Depths, Elevations and RQD**

The bedrock is identified as white to light grey colored, medium grained, muscovite granite, hard, massive, and fresh. The rock quality designation (RQD) of the bedrock was determined to be 93 to 100 percent indicating a rock mass quality of excellent.

### 5.8 Groundwater

Groundwater was observed at a depth of approximately 14.0 to 15.0 feet below the existing ground surface in the borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

## 6.0 Foundation Alternatives

The following foundation alternatives were considered for the bridge replacement:

- Reuse of existing abutments,
- Integral driven H-pile supported foundation at existing abutment locations, and
- Integral driven H-pile supported foundations located behind the existing abutments.

The reuse of the existing abutments was ruled out due to age and scour issues. Building the new abutments at the existing abutments locations was ruled out due to hydraulic, cost and construction issues. The use of H-pile supported integral abutments located behind the existing abutments was selected. This report addresses only this foundation type. The existing abutments will be removed to the Q1.1 elevation and riprap slopes will be constructed behind the remaining concrete and sheet piling will be driven behind the portion of the abutment to remain for scour protection.

# 7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for H-pile supported integral abutments driven to bedrock located behind the existing abutments. The existing abutments will be removed down to the Q1.1 elevation and sheet piling will be driven behind the portion of the abutment to remain for scour protection.

## 7.1 Integral Abutment H-Piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with pile tips to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 7-1 below:

| Location | Estimated Pile Cap Bottom Elevation | Approximate Depth to Bedrock From Ground Surface | Approximate Top of Rock Elevation | Estimated Pile Length |
|---|---|---|---|---|
| Abutment #1 BB-LSS-101 | 375.5 feet | 131.4 feet | 253.0 feet | 125 feet |
| Abutment #2 BB-LSS-102 | 375.5 feet | 138.4 feet | 245.6 feet | 130 feet |

**Table 7-1 – Estimated Pile Lengths for Plumb H-Piles**

These pile lengths do not take into account the length of pile embedded in the pile cap, the additional two (2) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate damaged pile lengths, bedrock deeper than that encountered in the borings and the Contractor's leads and driving equipment.

### 7.1.1 Strength Limit State Design

The design of pile foundations bearing on or within the bedrock at the strength limit state shall consider:

- Structural resistance of individual piles in axial compression
- Structural resistance of individual piles in combined axial loading and flexure
- Compressive axial geotechnical resistance of individual piles bearing on rock

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Since the H-piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in AASHTO LRFD Bridge Design Specifications 6$^{th}$ Edition (LRFD) Articles 6.9.2.2 and 6.15.2. The analysis shall assign a fixed condition at the pile tip. The H-piles shall also be checked for fixity and combined axial and flexure using LPile® software.

**Structural Resistance.** The nominal axial structural compressive resistance ($P_n$) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the factored axial structural compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, $\phi_c$, of 0.6 (good driving conditions) and an unbraced length ($\ell$) of 1 foot and an effective length factor (K) of 1.2. This factored axial structural compressive resistance is presented in Table 7-2 below. It is the responsibility of the structural engineer to recalculate the nominal axial structural compressive resistance ($P_n$) based on "actual unbraced pile length ($\ell$) and effective length factor (K)" or "on the actual elastic critical buckling resistance, $P_e$".

**Geotechnical Resistance.** The nominal axial geotechnical compressive resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states that "The nominal bearing resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving ($\phi_c$=0.50)." These factored axial geotechnical compressive resistances are presented in Table 7-2 below.

**Drivability Resistance.** The drivability of the five (5) proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is $\varphi_{dyn}$= 0.65. This factored drivability resistance is presented in Table 7-2 below.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections for the strength limit state is presented in Table 7-2 below. Supporting calculations are included in Appendix C-Calculations found at the end of this report.

| Pile Section | Strength Limit State Factored Axial Pile Resistance (kips) | | | |
|---|---|---|---|---|
| | Structural Resistance[1] $\phi_c$=0.60 | Controlling Geotechnical Resistance[2] $\phi_c$=0.50 | Drivability Resistance $\varphi_{dyn}$=0.65 | Governing Resistance |
| HP 12x53 | 464 | 387 | 343 | 343 |
| HP 12x74 | 653 | 544 | 432 | 432 |
| HP 14x73 | 641 | 534 | 406 | 406 |
| HP 14x89 | 782 | 652 | 468 | 468 |
| HP 14x117 | 1031 | 859 | 632 | 632 |

1 Based on preliminary assumption of $\ell$=1 foot and K=1.2
2 Calculated using LRFD Article 10.7.3.2.3

**Table 7-2 - Factored Axial Resistances for Abutment Piles at the Strength Limit State**

Local experience supports the estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the last column of Table 7-2 above.

The piles shall also be checked for resistance against combined axial compression and flexure accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor $\phi_c$=0.7 and the flexural resistance factor $\phi_f$ =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

### 7.1.2 Service and Extreme Limit State Design

The design of the H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall stability of the pile group and pile group movements/stability considering changes in foundation conditions due to scour at the design flood event.

Extreme limit state design checks for the H-piles shall include pile axial bearing resistance, failure of the pile group by overturning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. The design and check floods for scour are defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

For the service and extreme limit states resistance factors, ϕ, of 1.0 are recommended for structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.1 and 10.5.5.3. It is the responsibility of the structural engineer to recalculate $P_n$ based on refined elastic critical buckling resistance ($P_e$) evaluations. The nominal axial geotechnical resistance in the service and extreme limit states was calculated using Canadian Foundation Engineering Manual and the guidance in LRFD Article 10.7.3.2.3.

For the service and extreme limit states, the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 7-3 below. Supporting calculations are included in Appendix C-Calculations found at the end of this report.

| Pile Section | Service and Extreme Limit States Factored Axial Pile Resistance (kips) | | | |
|---|---|---|---|---|
| | Structural Resistance[1] ϕ=1.0 | Controlling Geotechnical Resistance[2] ϕ=1.0 | Drivability Resistance ϕ=1.0 | Governing Resistance |
| HP 12x53 | 774 | 744 | 528 | 528 |
| HP 12x74 | 1088 | 1088 | 665 | 665 |
| HP 14x73 | 1069 | 1069 | 624 | 624 |
| HP 14x89 | 1303 | 937 | 720 | 720 |
| HP 14x117 | 1718 | 1226 | 972 | 972 |

1 Based on preliminary assumption of $\ell$=1 foot and K=1.2
2 Calculated using LRFD Article 10.7.3.2.3

**Table 7-3 - Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States**

Local experience supports the estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the last column of Table 7-3 above.

### 7.1.3 Driven Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each integral abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the Contractor in the wave equation analysis. Restrikes will not be required as a part of the field quality control program unless pile behavior indicates the pile is not seated firmly on bedrock or if piles "walk" out of position. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The maximum factored axial pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident and verified by dynamic pile test measurements. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

### 7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral loads, vehicular loads, dead and live loads and lateral forces transferred through the integral structure. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider changes in foundation conditions and pile group resistance after scour due to the design flood.

A resistance factor of $\phi$= 1.0 shall be used to assess abutment design at the service limit state including: settlement, excessive horizontal movement and movement resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, $\varphi$, of 0.65.

Extreme limit state design checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, $\phi$, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The Designer may assume Soil Type 4 (MaineDOT Bridge Design Guide [BDG] Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi$ = 32 degrees, $\gamma$ = 125 pcf and a soil-concrete friction angle of 20 degrees. Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive earth pressure state. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, $K_p$, of 3.25 anticipating that integral abutments will experience some movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.005, then the calculation of lateral earth pressure should assume a Coulomb passive earth pressure coefficient, $K_p$, of 6.89. For designing the integral abutment backwall reinforcing steel, use a maximum load factor ($\gamma_{EH}$) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height ($h_{eq}$) taken from Table 7-4 below:

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

**Table 7-4 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic**

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Weep holes should be constructed approximately 6 inches above the Q1.1 elevation (normal high water). The approach slab should be positively connected to the integral abutment. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.4.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the 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.

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V unless project specific slope stability analyses are performed.

### 7.3 Sheet Pile Wall

The existing abutments will be removed down to the Q1.1 elevation and a permanent sheet pile wall will be installed behind the portion of the abutments to remain for scour protection. A riprap slope will be constructed between the proposed abutments and the sheet pile walls behind the existing abutments remaining. The sheet pile walls will be designed to support the bridge and roadway embankment in the event that material in front of the existing abutment remaining is scoured away. It is estimated that the sheet pile walls will have a length of approximately 37 feet. Based on the subsurface conditions encountered at the site the following recommendations are made:

Unanchored cantilever sheet pile walls shall be designed to meet the requirements of AASHTO LRFD Bridge Design Specifications 6$^{th}$ Edition (LRFD) Article 11.8 and to withstand lateral earth pressures. The design of the sheet pile wall shall be consistent with the apparent earth pressure diagrams provided in LRFD Article 3.11.5.6. Earth loads shall be calculated using an active earth pressure coefficient, $K_a$, calculated using Rankine Theory. Where passive earth pressure in front of the wall can be considered, a passive earth pressure coefficient, $K_p$, calculated using Rankine Theory may be used. Table 7-5 presents the recommended earth pressure coefficients:

| Internal Friction Angle $\phi$ | $K_a$ Rankine | $K_p$ Rankine |
|---|---|---|
| 32 degrees | 0.307 | 3.25 |
| 34 degrees | 0.283 | 3.54 |

**Table 7-5 – Recommended Earth Pressure Coefficients**

Anchored sheet pile walls shall be designed to meet the requirements of LRFD Article 11.9 using the apparent earth pressure diagrams provided in LRFD Article 3.11.5.7.

Uncoated sheet piles are permitted. The selected sheet pile section should consider a sacrificial steel loss per the MaineDOT BDG. Water moving through in the retained slope is likely to induce corrosion of the steel.

The use of hot-rolled sheets is recommended. Cold rolled sheet piles are not recommended for permanent applications. Cold rolled piles are typically thinner for the same section modulus. Section loss from corrosion could have a greater effect on cold rolled steel. The use of a ball and socket interlock system is recommended over the hook-type interlock system as the ball and socket system is less likely to unhook and separate underground due to driving pressure or obstructions. The use of American Society for Testing and Materials (ASTM) A 572 Grade 50 steel is recommended.

### 7.4 Precast Concrete Modular Block Retaining Wall

The use of a Precast Concrete Modular Gravity (PCMG) wall is proposed on the downstream south corner of Abutment No. 1 to retain the roadway section and minimize impacts. The wall shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The wall shall be designed in accordance with LRFD and Special Provision 635 which is included in Appendix D found at the end of this report.

The PCMG wall design shall consider a live load surcharge estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($h_{eq}$) taken from Table 7-6 below:

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

**Table 7-6 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls**

The factored bearing resistance at the strength limit state for a PCMG wall founded on compacted sand fill vs. foundation width is shown by the dashed line in Figure 7-1 below.

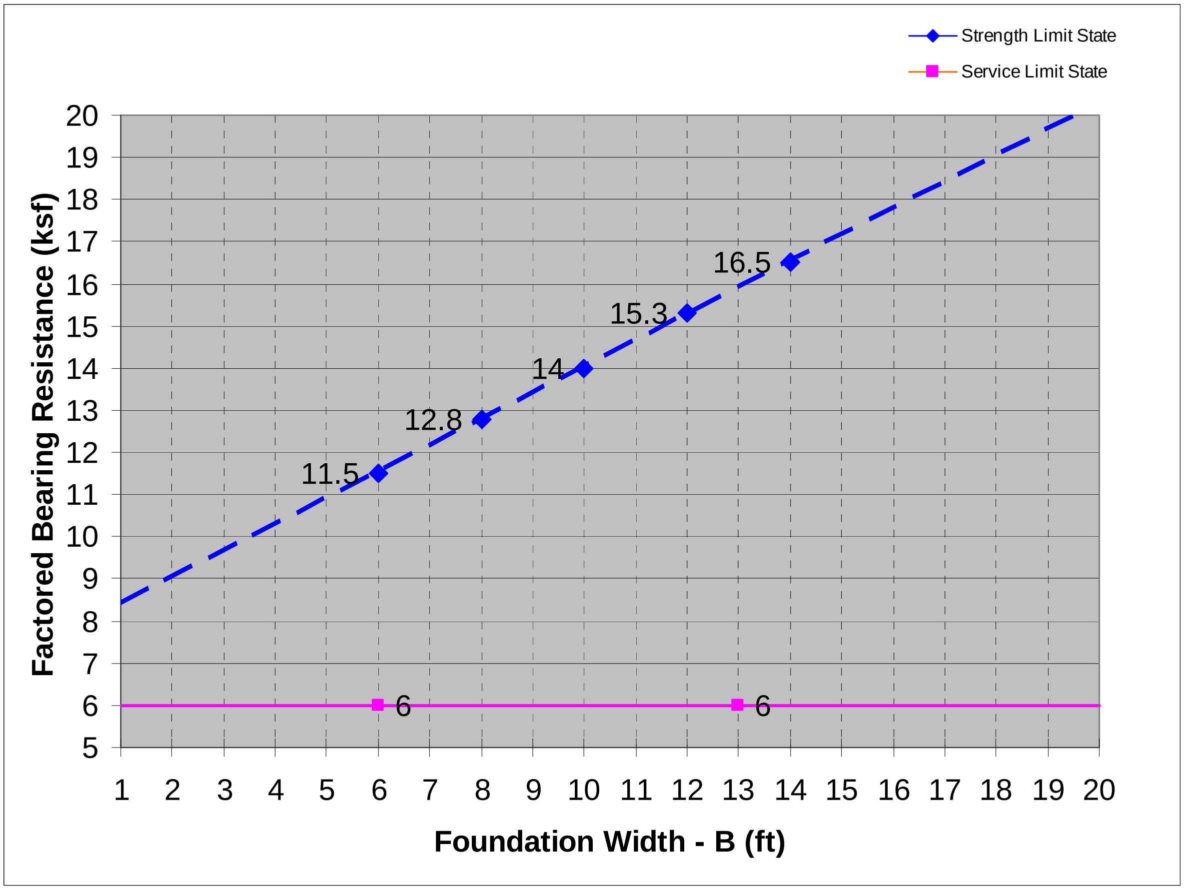



**Figure 7-1 – Factored Bearing Resistance of PCMG Wall Bearing on Compacted Sand vs. Foundation Width**

Once the dimensions of the PCMG wall are determined, a factored bearing resistance can be determined from the figure. This factored bearing resistance must be greater than the applied factored vertical bearing pressure determined by the structural designer. The factored bearing resistance at the service limit state is shown by the solid line in Figure 7-1. A factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing as allowed in LRFD C10.6.2.1. See Appendix C - Calculations for supporting calculations.

The bearing resistance for PCMG bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The PCMG units shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor $\phi$, of 0.65.

The designer shall apply a sliding resistance factor $\varphi_T$ of 0.90 to the nominal sliding resistance of precast concrete wall segments founded on sand. The eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions in either direction (LRFD Article 10.6.3.3). Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of tan 30º at the foundation soil to soil infill interface and a maximum frictional coefficient of 0.8x(tan 30º) at the foundation soil to concrete module interface. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

### 7.5 Scour and Riprap

Grain size analyses were performed on soil samples taken from the streambed to generate grain size curves for determining parameters to be used in scour analyses. The samples were similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, $D_{50}$ = 0.13 mm
- Average diameter of particle at 95 percent passing, $D_{95}$ = 0.37 mm
- Soil Classification AASHTO Soil Type A-2-4

The grain size curves are included in Appendix B- Laboratory Data found at the end of this report.

The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

For scour protection and protection of pile groups and PCMG walls, the bridge approach slopes, slopes at abutments and PCMG walls should be armored with 3 feet of plain riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design. The existing abutments will be removed down to the Q1.1 elevation and a permanent sheet pile wall will be installed behind the portion of the abutments to remain as a scour countermeasure.

Bridge approach slopes, slopes at wingwalls and at PCMG walls shall be armored with 3 feet of plain riprap conforming to MaineDOT Supplemental Specification Section 703.26 Plain and Heavy Riprap and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 Erosion Control Geotextile per Standard Details 610(02) through 610(04).

### 7.6 Settlement

The roadway profile will be raised approximately 1.2 feet at the abutments. Potential settlement due the placement of the proposed fill is estimated to be approximately 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill will occur during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

### 7.7 Frost Protection

Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG.

PCMG walls placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has design-freezing index of approximately 2000 F-degree days. In a granular soil with a water content of approximately 30%, this correlates to a frost depth of approximately 6.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection. See Appendix C - Calculations at the end of this report for supporting documentation.

### 7.8 Seismic Design Considerations

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

- Peak ground acceleration coefficient (PGA) = 0.073g
- Site Class E (soil profile with average N-value for the upper 100 feet of soil profile less than 15 blows per foot)
- Acceleration coefficient ($A_s$) = 0.182g
- Design spectral acceleration coefficient at 0.2-second period, $S_{DS}$ = 0.399g
- Design spectral acceleration coefficient at 1.0-second period, $S_{D1}$ = 0.171g
- Seismic Zone 2, based on: $0.15g < S_{D1} < 0.30g$ (LRFD Table 3.10.6-1)

In conformance with LRFD Table 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, the Lower Sandy Stream Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criterion eliminates the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

### 7.9 Construction Considerations

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill prior to pile driving.

A layer of wood was encountered in the area of proposed Abutment No. 1 and wood fragments were encountered in the lower fill soils at proposed Abutment No. 2. It is likely that the presence of wood at either abutment will impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations, installation of sheet piles for cofferdams and installation of permanent sheet pile for scour countermeasures. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident. The potential for obstructions to slow construction activities should be considered by the Contractor.

In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

Using the excavated native soils as structural backfill should not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

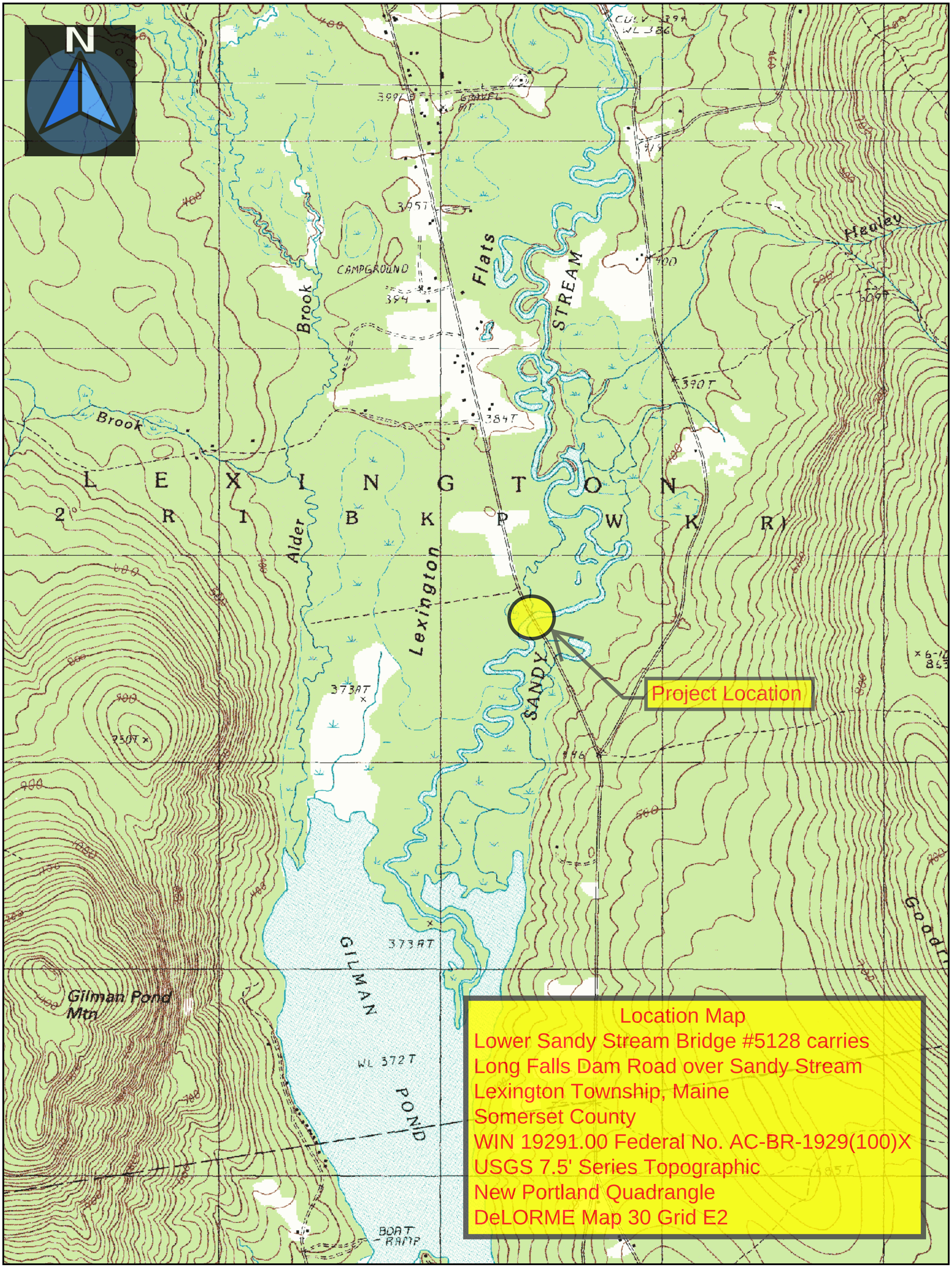
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

## 8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Lower Sandy Stream Bridge in Lexington Township in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

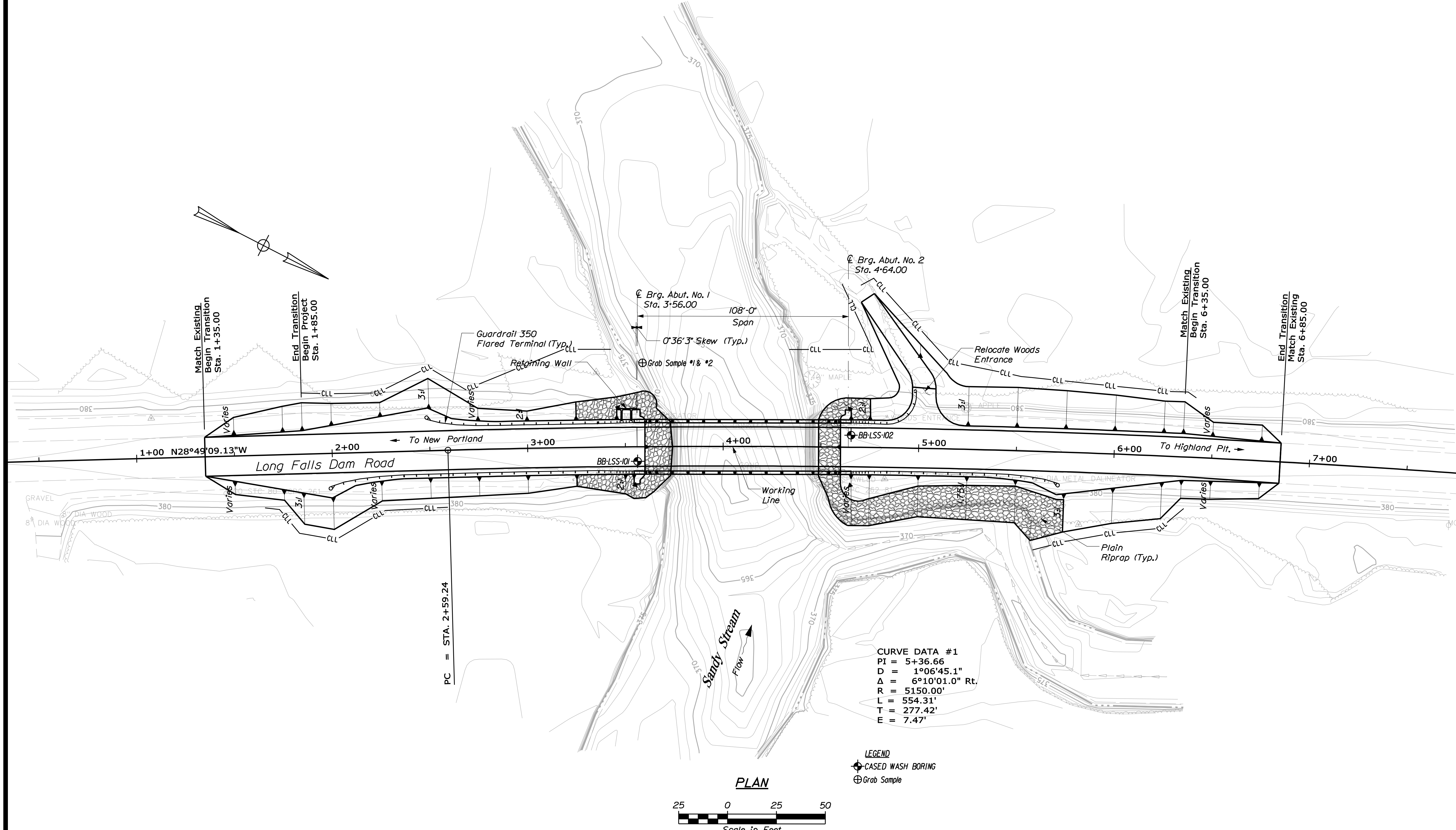
It is also recommend that the geotechnical engineer be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

**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.*




STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1929(100)X

BRIDGE NO. 5128 | WIN 19291.00 | BRIDGE PLANS

| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | S.BODGE | |
| DESIGN-DETAILED | R. MYERS | |
| CHECKED-REVIEWED | B. NICHOLS | |
| DESIGN2-DETAILED2 | K. MAGUIRE | |
| DESIGN3-DETAILED3 | T. WHITE | AUG 2012 |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

SIGNATURE

P.E. NUMBER

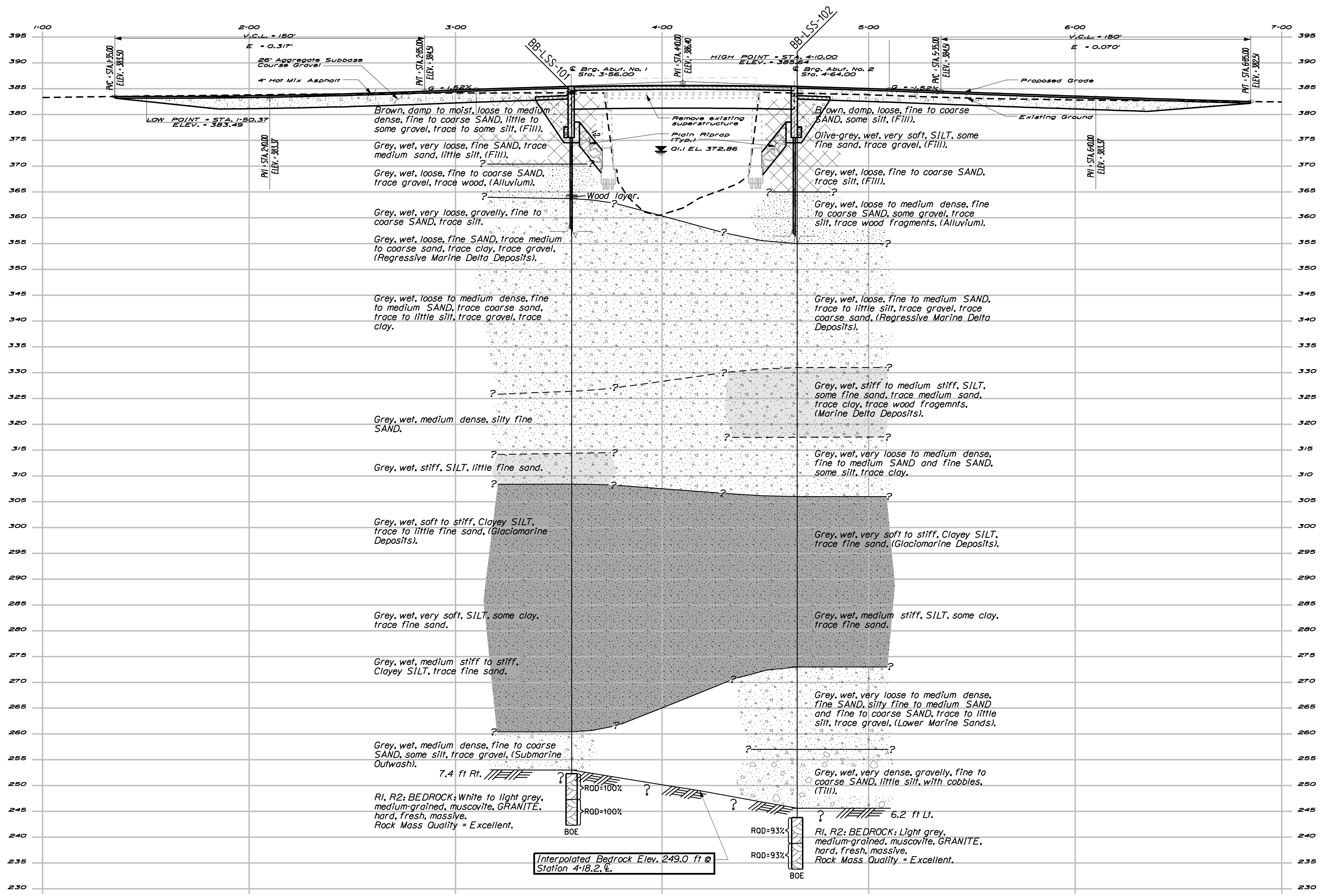
DATE

LOWER SANDY STREAM BRIDGE
SANDY STREAM
LEXINGTON TWP SOMERSET COUNTY

# BORING LOCATION PLAN

SHEET NUMBER
2
OF 5

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012




## 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

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

AC-BR-1929(100)X

WIN 19291.00

BRIDGE PLANS

BRIDGE NO. 5128

| | | | |
|---|---|---|---|
| | | BY | DATE |
| PROJ. MANAGER | S.BODGE | | |
| DESIGN-DETAILED | R. MYERS | B. NICHOLS | |
| CHECKED-REVIEWED | | | |
| DESIGN2-DETAILED2 | K. MAGUIRE | T. WHITE | AUG 2012 |
| DESIGN3-DETAILED3 | | | |
| REVISIONS 1 | | | |
| REVISIONS 2 | | | |
| REVISIONS 3 | | | |
| REVISIONS 4 | | | |
| FIELD CHANGES | | | |

SIGNATURE

P.E. NUMBER

DATE

LOWER SANDY STREAM BRIDGE
SANDY STREAM
LEXINGTON TWP SOMERSET COUNTY

# INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012

| Maine Department of Transportation<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over<br>Location: Lexington Township, Maine | Boring No.: BB-LSS-101<br>WIN: 19291.00 |
|---|---|---|
| Driller: MaineDOT | Elevation (ft.) 384.4 | Auger ID/OD: 5" Solid Stem |
| Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon |
| Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30" |
| Date Start/Finish: 5/2,3,7,8/2012 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2" |
| Boring Location: 3+56.2, 7.4 ft Rt. | Casing ID/OD: HW & NW | Water Level*: 15.0 ft bgs. |
| Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 383.98 | 5" Pavement 0.42 | |
| | 1D | 24/12 | 1.00 - 3.00 | 5/5/5/6 | 10 | 14 | | | Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill). | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | 380.40 | 4.00 | |
| 5 | 2D | 24/14 | 5.00 - 7.00 | 2/3/3/2 | 6 | 8 | 18 | | Brown, moist, loose, fine to coarse SAND, little gravel, some silt, (Fill). | |
| | | | | | | | 11 | | | |
| | | | | | | | 11 | | | |
| | | | | | | | 8 | | | |
| | | | | | | | 11 | 375.40 | 9.00 | |
| 10 | 3D | 24/14 | 10.00 - 12.00 | 1/2/1/2 | 3 | 4 | 13 | | Grey, wet, very loose, fine SAND, trace medium sand, little silt, (Fill) | G#261888 A-2-4, SM WC=31.1% |
| | | | | | | | 12 | | | |
| | | | | | | | 16 | | | |
| | | | | | | | 19 | | | |
| | | | | | | | 25 | 370.40 | 14.00 | |
| 15 | 4D | 24/13 | 15.00 - 17.00 | 3/4/3/4 | 7 | 10 | 15 | | Grey, wet, loose, fine to coarse SAND, trace gravel, trace wood fragments, (piles, cribbing?), (Reworked Alluvial). | |
| | | | | | | | 26 | | | |
| | | | | | | | 26 | | | |
| | | | | | | | 25 | | | |
| | | | | | | | 36 | 364.40 | 20.00 | |
| 20 | 5D | 24/4 | 20.70 - 22.70 | 1/1/2/2 | 3 | 4 | 23 | 363.70 | WOOD layer from 20.0-20.7 ft bgs. 20.70 Grey, wet, very loose, gravelly, fine to coarse SAND, trace silt. | |
| | | | | | | | 28 | | | |
| | | | | | | | 33 | | | |
| | | | | | | | 35 | | | |
| | | | | | | | 53 | | | |
| 25 | 6D | 24/14 | 25.00 - 27.00 | 2/2/2/2 | 4 | 6 | 41 | | Grey, wet, loose, fine SAND, trace medium to coarse sand, trace silt, trace clay, trace gravel. (Regressive Marine Delta Deposits) | G#261889 A-2-4, SP-SC WC=19.9% |
| | | | | | | | 54 | | | |
| | | | | | | | 73 | | | |
| | | | | | | | 79 | | | |
| | | | | | | | 84 | | | |
| 30 | 7D | 24/18 | 30.00 - 32.00 | 2/2/2/2 | 4 | 6 | 46 | | Grey, wet, loose, fine to medium SAND, trace coarse sand, trace silt, trace gravel. | G#261890 A-3, SP-SM WC=18.4% |
| | | | | | | | 63 | | | |
| | | | | | | | 92 | | | |
| | | | | | | | 126 | | | |
| | | | | | | | 131 | | | |
| 35 | 8D | 24/17 | 35.00 - 37.00 | 2/2/3/3 | 5 | 7 | 78 | | Grey, wet, loose, fine to medium SAND, trace coarse sand, little silt, trace gravel. | G#261891 A-2-4, SM WC=20.2% |
| | | | | | | | 83 | | | |
| | | | | | | | 99 | | | |
| | | | | | | | 114 | | | |
| | | | | | | | 156 | | | |
| 40 | 9D | 24/20 | 40.00 - 42.00 | 4/4/4/4 | 8 | 11 | 8 | | Similar to above, except medium dense. Switched to NW casing at 40.0 ft bgs. | |
| | | | | | | | 11 | | | |
| | | | | | | | 32 | | | |
| | | | | | | | 52 | | | |
| | | | | | | | 56 | | | |
| 45 | 10D | 24/18 | 45.00 - 47.00 | 1/2/3/3 | 5 | 7 | 21 | | Similar to above, except loose. | |
| | | | | | | | 72 | | 2" running sand | |
| | | | | | | | 193 | | | |
| | | | | | | | 219 | | | |
| | | | | | | | 224 | | | |
| 50 | 11D | 24/21 | 50.00 - 52.00 | 2/3/5/6 | 8 | 11 | 90 | | Grey, wet, medium dense, fine to medium SAND, little silt, trace coarse sand, trace gravel, trace clay. | G#261892 A-2-4, SC-SM WC=23.2% |
| | | | | | | | 76 | | | |
| | | | | | | | 176 | | | |
| | | | | | | | 221 | | | |
| | | | | | | | 230 | | | |
| 55 | 12D | 24/18 | 55.00 - 57.00 | 3/4/4/5 | 8 | 11 | 122 | | Similar to above. | |
| | | | | | | | 162 | | | |
| | | | | | | | 232 | 326.40 | 58.00 | |
| | | | | | | | 272 | | | |
| | | | | | | | 281 | | | |
| 60 | 13D | 24/20 | 60.00 - 62.00 | 5/6/5/6 | 11 | 15 | 135 | | Grey, wet, medium dense, Silty fine SAND. | |
| | | | | | | | 129 | | | |
| | | | | | | | 165 | | | |
| | | | | | | | 230 | | | |
| | | | | | | | 281 | | | |
| 65 | 14D | 24/15 | 65.00 - 67.00 | 4/4/4/5 | 8 | 11 | 165 | | Grey, wet, medium dense, Silty fine SAND. | |
| | | | | | | | 162 | | | |
| | | | | | | | 165 | | | |
| | | | | | | | 170 | | | |
| | | | | | | | 216 | 314.40 | 70.00 | |
| 70 | 15D | 24/19 | 70.50 - 72.50 | 4/2/6/6 | 8 | 11 | 195 | | Grey, wet, stiff, SILT, little fine sand, trace clay. | G#261893 A-4, ML WC=21.3% |
| | | | | | | | 99 | | | |
| | | | | | | | 101 | | | |
| | | | | | | | 93 | | | |
| | | | | | | | 99 | | | |
| 75 | | | | | | | | | | |

Remarks:

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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 2

Boring No.: BB-LSS-101


| Maine Department of Transportation<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over<br>Location: Lexington Township, Maine | Boring No.: BB-LSS-101<br>WIN: 19291.00 |
|---|---|---|
| Driller: MaineDOT | Elevation (ft.) 384.4 | Auger ID/OD: 5" Solid Stem |
| Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon |
| Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30" |
| Date Start/Finish: 5/2,3,7,8/2012 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2" |
| Boring Location: 3+56.2, 7.4 ft Rt. | Casing ID/OD: HW & NW | Water Level*: 15.0 ft bgs. |
| Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | |


| 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 | 24/20 | 75.00 - 77.00 | 3/2/WOR/WOR | 2 | 3 | 132 | | | |
| | | | | | | | 107 | 308.40 | 76.00 Grey, wet, soft, Clayey SILT, little fine sand. (Glaciomarine Deposit) | |
| | | | | | | | 103 | | | |
| | | | | | | | 103 | | | |
| | | | | | | | 106 | | | |
| 80 | 17D V1 | 24/22 | 80.00 - 82.00 80.63 - 81.00 | WOR/WOR/WOR/WOR Su=1652/134 psf | --- | | 139 | | Grey, wet, stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V1: 37.0/3.0 ft-lbs | G#261894 A-4, CL-ML WC=28.0% LL=26 PL=19 PI=7 |
| | V2 | | 81.63 - 82.00 | Su=1384/112 psf | | | 127 | | V2: 31.0/2.5 ft-lbs | |
| | | | | | | | 125 | | | |
| | | | | | | | 126 | | | |
| | | | | | | | 116 | | | |
| 85 | | | | | | | 135 | | Washed ahead to 86.0 ft bgs. | |
| | 18D V3 | 24/20 | 86.00 - 88.00 86.63 - 87.00 | WOR/WOR/6/4 Su=915/223 psf | 6 | 8 | 120 | | 55x110 mm vane raw torque readings: V3: 20.5/5.0 ft-lbs | |
| | MV | | | Would not push | | | 77 | | Grey, wet, medium stiff, Clayey SILT, with 1"-2.5" fine sand layers. Failed 55x110 mm vane attempt. | |
| | | | | | | | 81 | | | |
| | | | | | | | 76 | | | |
| 90 | 19D V4 | 24/24 | 90.00 - 92.00 90.63 - 91.00 | WOR/WOR/WOR/WOR Su=759/268 psf | --- | | 78 | | Grey, wet, medium stiff, Clayey SILT, trace fine sand in layers. 55x110 mm vane raw torque readings: V4: 17.0/6.0 ft-lbs | |
| | V5 | | 91.63 - 92.00 | Su=625/223 psf | | | 79 | | V5: 14.0/5.0 ft-lbs | |
| | | | | | | | 78 | | | |
| | | | | | | | 62 | | | |
| | | | | | | | 63 | | | |
| 95 | 20D V6 | 24/20 | 95.00 - 97.00 95.63 - 96.00 | WOR/WOR/WOR/WOR Su=714/179 psf | --- | | 90 | | Similar to above. 55x110 mm vane raw torque readings: V6: 16.0/4.0 ft-lbs | |
| | MV | | | Would not push | | | 76 | | Failed 55x110 mm vane attempt. | |
| | | | | | | | 67 | | | |
| | | | | | | | 65 | | | |
| | | | | | | | 70 | | | |
| 100 | 21D V7 | 24/24 | 100.00 - 102.00 100.63 - 101.00 | WOR/WOR/WOR/WOR Su=937/246 psf | --- | | 81 | | Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand in layers. 55x110 mm vane raw torque readings: V7: 21.0/5.5 ft-lbs | G#261895 A-6, CL WC=27.7% LL=29 PL=15 PI=14 |
| | V8 | | 101.63 - 102.00 | Su=1161/268 psf | | | 73 | | V8: 26.0/6.0 ft-lbs | |
| | | | | | | | 70 | | | |
| | | | | | | | 76 | | | |
| | | | | | | | 78 | | | |
| 105 | 22D MV | 24/20 | 105.00 - 107.00 | WOR/WOR/WOR/WOR Would not push | --- | | 113 | | Grey, wet, very soft, SILT, some clay, trace fine sand. Failed 55x110 mm vane attempt. | G#261896 A-4, CL-ML WC=26.5% LL=27 PL=21 PI=6 |
| | | | | | | | 111 | | | |
| | | | | | | | 81 | | | |
| | | | | | | | 83 | | | |
| | | | | | | | 83 | | | |
| 110 | 23D V9 | 24/24 | 110.00 - 112.00 110.63 - 111.00 | WOR/WOR/WOR/WOR Su=1116/312 psf | --- | | 116 | | Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V9: 25.0/7.0 ft-lbs | |
| | V10 | | 111.63 - 112.00 | Su=982/246 psf | | | 85 | | V10: 22.0/5.5 ft-lbs | |
| | | | | | | | 79 | | | |
| | | | | | | | 79 | | | |
| | | | | | | | 108 | | | |
| 115 | 24D V11 | 24/24 | 115.00 - 117.00 115.63 - 116.00 | WOR/WOR/WOR/WOR Su=1161/402 psf | --- | | 116 | | Similar to above, stiff. 55x110 mm vane raw torque readings: V11: 26.0-9.0 ft-lbs | |
| | MV | | | Would not push | | | 117 | | Failed 55x110 mm vane attempt. | |
| | | | | | | | 111 | | | |
| | | | | | | | 115 | | | |
| | | | | | | | 119 | | | |
| 120 | 25D V12 | 24/17 | 120.00 - 122.00 120.63 - 121.00 | WOR/WOR/WOR/WOR Su=1339/246 psf | --- | | 130 | | Similar to above. 55x110 mm vane raw torque readings: V12: 30.0/5.5 ft-lbs | |
| | | | | | | | 126 | | | |
| | | | | | | | 111 | | | |
| | | | | | | | 110 | 260.40 | 124.00 | |
| | | | | | | | 127 | | | |
| 125 | 26D | 24/18 | 125.00 - 127.00 | 6/10/10/11 | 20 | 28 | 135 | | Grey, wet, medium dense, fine to coarse SAND, some silt, trace gravel, (Submarine outwash). | G#261897 A-2-4, SM WC=14.7% |
| | | | | | | | 116 | | | |
| | | | | | | | 111 | | | |
| | | | | | | | 84 | | | |
| | | | | | | | 111 | | | |
| 130 | | | | | | | 94 | | | |
| | | | | | | | a110 | 253.00 | a110 blows for 0.4 ft. 131.40 | |
| | R1 | 60/60 | 132.10 - 137.10 | RQD = 100% | | | NQ-2 | | Top of Bedrock at Elev. 253.0 ft. Roller Coned ahead to 132.1 ft bgs. R1:Bedrock: White to light grey, medium-grained, muscovite GRANITE, hard, fresh, massive. Rock Mass Quality = Excellent. R1:Core Times (min:sec) 132.1-133.1 ft (2:00) 133.1-134.1 ft (2:15) 134.1-135.1 ft (2:00) 135.1-136.1 ft (2:00) 136.1-137.1 ft (2:15) 100% Recovery | |
| | | | | | | | | | | |
| 135 | | | | | | | | | | |
| | | | | | | | | | | |
| | R2 | 60/60 | 137.10 - 142.10 | RQD = 100% | | | | | R2:Bedrock: Similar to R1. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 137.1-138.1 ft (2:00) 138.1-139.1 ft (1:50) 139.1-140.1 ft (1:40) 140.1-141.1 ft (1:40) 141.1-142.1 ft (1:55) 100% Recovery | |
| | | | | | | | | | | |
| 140 | | | | | | | | | | |
| | | | | | | | | 242.30 | 142.10 Bottom of Exploration at 142.10 feet below ground surface. | |
| 145 | | | | | | | | | | |
| 150 | | | | | | | | | | |

Remarks:

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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 2 of 2

Boring No.: BB-LSS-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1929(100)X

BRIDGE PLANS
WIN 19291.00
BRIDGE NO. 5128

| | | BY | DATE |
|---|---|---|---|
| PROJ. MANAGER | S.BODGE | | |
| DESIGN-DETAILED | R. MYERS | B. NICHOLS | |
| CHECKED-REVIEWED | K. MAGUIRE | T. WHITE | DEC 2012 |
| DESIGN2-DETAILED2 | | | |
| DESIGN3-DETAILED3 | | | |
| REVISIONS 1 | | | |
| REVISIONS 2 | | | |
| REVISIONS 3 | | | |
| REVISIONS 4 | | | |
| FIELD CHANGES | | | |

SIGNATURE
P.E. NUMBER
DATE

LOWER SANDY STREAM BRIDGE
SANDY STREAM
LEXINGTON TWP SOMERSET COUNTY

# BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\msta\009_BORING LOGS2.dgn

Division: GEOTECH

Username: terry.white

Date:12/20/2012

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

Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over
Location: Lexington Township, Maine

Boring No.: BB-LSS-102
WIN: 19291.00

| | | | | | |
|---|---|---|---|---|---|
| Driller: | MaineDOT | Elevation (ft.) | 384.0 | Auger ID/OD: | 5" Solid Stem |
| Operator: | Giguere/Giles/Daggett | Datum: | NAVD88 | Sampler: | Standard Split Spoon |
| Logged By: | B. Wilder | Rig Type: | CME 45C | Hammer Wt./Fall: | 140#/30" |
| Date Start/Finish: | 5/14,16,17,21/2012 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ-2" |
| Boring Location: | 4+65.4, 6.2 ft Lt. | Casing ID/OD: | HW & NW | Water Level*: | 14.0 ft bgs. |
| Hammer Efficiency Factor: 0.84 | | Hammer Type: | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 383.58 | 5" Pavement —0.42<br>Brown, damp, gravelly, fine to coarse SAND, trace silt, (Fill.) | |
| 5 | 1D | 24/13 | 5.00 - 7.00 | 1/2/3/1 | 5 | 7 | | | Brown, damp, loose, fine to coarse SAND, some silt, (Fill). | |
| | | | | | | | | 375.50 | —8.50 | |
| 10 | 2D | 24/24 | 10.00 - 12.00 | WOH/WOH/WOH/WOH | --- | | 7 | | Olive-grey, wet, very soft, SILT, some fine sand, trace clay, (Fill; Reworked native soils).<br>Set in HW Casing at 10.0 ft bgs. | G#261898<br>A-4, ML<br>WC=40.4% |
| | | | | | | | 8 | | | |
| | | | | | | | 8 | | | |
| | | | | | | | 11 | | | |
| | | | | | | | 28 | 370.00 | —14.00 | |
| 15 | 3D | 24/16 | 15.00 - 17.00 | 3/3/3/2 | 6 | 8 | 33 | | Grey, wet, loose, fine to coarse SAND, trace silt, (Fill; Reworked native soils). | |
| | | | | | | | 27 | | | |
| | | | | | | | 32 | | | |
| | | | | | | | 33 | | | |
| | | | | | | | 54 | 365.00 | —19.00 | |
| 20 | 4D | 24/13 | 20.00 - 22.00 | 2/3/5/5 | 8 | 11 | 62 | | Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt, trace wood fragments, (Alluvium; reworked?). | |
| | | | | | | | 62 | | | |
| | | | | | | | 60 | | | |
| | | | | | | | 58 | | | |
| | | | | | | | 59 | | | |
| 25 | 5D | 24/12 | 25.00 - 27.00 | 2/3/2/2 | 5 | 7 | 57 | | Similar to above, except loose. | |
| | | | | | | | 60 | | | |
| | | | | | | | 59 | | | |
| | | | | | | | 67 | | | |
| | | | | | | | 60 | 355.00 | —29.00 | |
| 30 | 6D | 24/15 | 30.00 - 32.00 | 3/2/3/3 | 5 | 7 | 62 | | Grey, wet, loose, fine SAND, little silt, trace gravel, (Regressive Marine Delta Deposits) | G#261899<br>A-2-4, SM<br>WC=22.6% |
| | | | | | | | 72 | | | |
| | | | | | | | 78 | | | |
| | | | | | | | 73 | | | |
| | | | | | | | 75 | | | |
| 35 | 7D | 24/17 | 35.00 - 37.00 | 1/2/3/2 | 5 | 7 | 101 | | Grey, wet, loose, fine to medium SAND, trace silt, trace gravel, trace coarse sand, (Regressive Marine Delta Deposits). | |
| | | | | | | | 79 | | | |
| | | | | | | | 79 | | | |
| | | | | | | | 104 | | | |
| | | | | | | | 122 | | | |
| 40 | 8D | 24/18 | 40.00 - 42.00 | 3/3/3/3 | 6 | 8 | 110 | | Grey, wet, loose, fine to medium SAND, trace silt, trace gravel, trace coarse sand. | G#261900<br>A-3, SP<br>WC=18.8% |
| | | | | | | | 111 | | | |
| | | | | | | | 111 | | | |
| | | | | | | | 111 | | | |
| | | | | | | | 112 | | | |
| 45 | 9D | 24/16 | 45.00 - 47.00 | 1/2/2/2 | 4 | 6 | 116 | | Similar to above. | |
| | | | | | | | 131 | | | |
| | | | | | | | 169 | | | |
| | | | | | | | 222 | | | |
| | | | | | | | 243 | | | |
| 50 | 10D | 24/19 | 50.00 - 52.00 | 2/2/2/2 | 4 | 6 | 126 | | Grey, wet, loose, fine to medium SAND, trace silt, trace coarse sand, trace gravel. | |
| | | | | | | | 162 | | | |
| | | | | | | | 221 | | | |
| | | | | | | | 283 | 331.00 | —53.00 | |
| | | | | | | | 354 | | | |
| 55 | 11D | 24/17 | 55.00 - 57.00 | 5/5/4/4 | 9 | 13 | 180 | | Grey, wet, stiff, SILT, some fine sand, trace clay, trace medium sand, trace wood fragments, (Marine Delta Deposits). | |
| | | | | | | | 163 | | | |
| | | | | | | | 229 | | | |
| | | | | | | | 336 | | | |
| | | | | | | | 396 | | | |
| 60 | 12D | 24/20 | 60.00 - 62.00 | 3/4/5/5 | 9 | 13 | 231 | | Grey, wet, stiff, SILT, some fine sand, trace medium sand, trace clay. | G#267501<br>A-4, ML<br>WC=21.2% |
| | | | | | | | 229 | | | |
| | | | | | | | 287 | | | |
| | | | | | | | 384 | | | |
| | | | | | | | 393 | | | |
| 65 | 13D | 24/19 | 65.00 - 67.00 | 2/2/3/4 | 5 | 7 | 273 | | Similar to above, except medium stiff. | |
| | | | | | | | 242 | 317.50 | —66.50<br>Grey, wet, loose, fine to medium SAND, some silt, trace clay. | |
| | | | | | | | 233 | | | |
| | | | | | | | 298 | | | |
| | | | | | | | 338 | | | |
| 70 | 14D | 24/18 | 70.50 - 72.50 | 3/5/8/8 | 13 | 18 | 277 | | Grey, wet, medium dense, fine SAND, some silt, trace medium sand, trace clay.<br>Roller coned ahead to 75.0 ft bgs. | G#267502<br>A-4, SC-SM<br>WC=22.3% |
| | | | | | | | 119 | | | |
| | | | | | | | 119 | | | |
| | | | | | | | 122 | | | |
| | | | | | | | 132 | | | |
| 75 | | | | | | | | | | |

Remarks:

100-200# down pressure on core barrel.

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

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



Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over
Location: Lexington Township, Maine
Boring No.: BB-LSS-102
WIN: 19291.00
Driller: MaineDOT | Elevation (ft.) 384.0 | Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5/14,16,17,21/2012 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 4+65.4, 6.2 ft Lt. | Casing ID/OD: HW & NW | Water Level*: 14.0 ft bgs.
Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

| 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 | 15D | 24/17 | 75.00 - 77.00 | WOR/WOR/3/4 | 3 | 4 | 176 | | Similar to above, except very loose. | |
| | | | | | | | 218 | | | |
| | | | | | | | 193 | | | |
| | | | | | | | 216 | 306.00 | —78.00 | |
| | | | | | | | 224 | | | |
| 80 | 16D | 24/24 | 80.00 - 82.00 | WOR/WOR/WOR/WOH | --- | | OPEN HOLE | | Grey, wet, very soft, Clayey SILT, trace fine sand, (Glaciomarine Deposit). | |
| 85 | 1U | 24/24 | 85.00 - 87.00 | WOR/WOR | --- | | | | Grey, wet, very soft, Clayey SILT with fine sand layers, (Glaciomarine Deposit). | |
| | V1 | | 87.63 - 88.00 | Su=1116/179 psf | | | | | 55x110 mm vane raw torque readings:<br>V1: 25.0/4.0 ft-lbs | |
| | V2 | | 88.63 - 89.00 | Su=491/134 psf | | | | | V2: 11.0/3.0 ft-lbs | |
| 90 | 17D | 24/24 | 90.00 - 92.00 | WOR/WOR/WOR/WOR | --- | | | | Grey, wet, stiff, Clayey SILT, trace sand, (Glaciomarine Deposit).<br>55x110 mm vane raw torque readings: | G#267504<br>A-4, ML<br>WC=27.9%<br>LL=25<br>PL=23<br>PI=2 |
| | V3 | | 90.63 - 91.00 | Su=1049/156 psf | | | | | V3: 23.5/3.5 ft-lbs | |
| | V4 | | 91.63 - 92.00 | Su=1384/223 psf | | | | | V4: 31.0/5.0 ft-lbs | |
| 95 | V5 | | 95.00 - 95.37 | Su=1451/179 psf | | | | | 55x110 mm vane raw torque readings:<br>V5: 32.5/4.0 ft-lbs | |
| | V6 | | 96.00 - 96.37 | Su=1406/312 psf | | | | | V6: 31.5/7.0 ft-lbs | |
| | 2U | 24/24 | 97.00 - 99.00 | WOR/WOR | --- | | | | Similar to above. | |
| 100 | 18D | 24/24 | 100.00 - 102.00 | WOR/WOR/WOR/WOR | --- | | | | 55x110 mm vane raw torque readings: | G#267503<br>A-4, CL-ML<br>WC=27.1%<br>LL=28<br>PL=21<br>PI=7 |
| | V7 | | 100.63 - 101.00 | Su=982/223 psf | | | | | V7: 22.0/5.0 ft-lbs | |
| | V8 | | 101.63 - 102.00 | Su=580/223 psf | | | | | Grey, wet, medium stiff, SILT, some clay, trace fine sand.<br>V8: 13.0./5.0 ft-lbs | |
| 105 | 3U | 24/24 | 105.00 - 107.00 | WOR/WOR | --- | | | | Similar to above. | |
| | V9 | | 107.00 - 107.37 | Su=402/134 psf | | | | | 55x110 mm vane raw torque readings:<br>V9: 29.0/3.0 ft-lbs | |
| | V10 | | 108.00 - 108.37 | Su=1161/312 psf | | | | | V10: 26.0/7.0 ft-lbs | |
| 110 | 19D | 24/20 | 110.00 - 112.00 | WOR/WOR/2/2 | 2 | 3 | 15 | | Set in NW Casing at 110.0 ft bgs.<br>55x110 mm vane raw torque readings: | |
| | V11 | | 110.63 - 111.00 | Su=1161/312 psf | | | | 273.00 | V11: 26.0/7.0 ft-lbs —111.00 | |
| | MV | | | Would not push | | | 16 | | Grey, wet, very loose, fine SAND, trace silt, (Lower Marine Sands).<br>Failed 55x110 mm vane attempt. | |
| | | | | | | | 14 | | | |
| | | | | | | | 16 | | | |
| | | | | | | | 16 | | | |
| 115 | 20D | 24/22 | 115.00 - 117.00 | WOR/WOH/WOH/WOH | --- | | 23 | | Grey, wet, very loose, Silty fine to medium SAND, (Lower Marine Sands). | |
| | | | | | | | 32 | | | |
| | | | | | | | 35 | | | |
| | | | | | | | 30 | | | |
| | | | | | | | 21 | | | |
| 120 | 21D | 24/21 | 120.00 - 122.00 | 1/3/4/6 | 7 | 10 | 59 | | Grey, wet, loose, fine to coarse SAND, little silt, trace gravel, (Lower Marine Sands). | G#267505<br>A-2-4, SM<br>WC=12.0% |
| | | | | | | | 55 | | | |
| | | | | | | | 62 | | | |
| | | | | | | | 48 | | | |
| | | | | | | | 37 | | | |
| 125 | 22D | 24/22 | 125.00 - 127.00 | 1/5/11/15 | 16 | 22 | 57 | | Similar to above, except medium dense. | |
| | | | | | | | 121 | | | |
| | | | | | | | 236 | 257.00 | —127.00<br>Roller Coned ahead from 127.5-130.0 ft bgs. | |
| | | | | | | | 279 | | | |
| | | | | | | | 299 | | | |
| 130 | 23D | 14.4/7 | 130.00 - 131.20 | 15/14/40(2.4") | --- | | 283 | | Grey, wet, very dense, Gravelly, fine to coarse SAND, little silt, with cobbles. (Glacial Till). | |
| | | | | | | | 150 | | | |
| | | | | | | | 191 | | | |
| | | | | | | | 195 | | Roller Coned ahead from 133.0-135.5 ft bgs. | |
| | | | | | | | 186 | | | |
| 135 | MD | 24/0 | 135.50 - 137.50 | 50/36/40/58 | 76 | 106 | 182 | | Roller Coned ahead to 140.2 ft bgs. | |
| | | | | | | | 184 | | | |
| | | | | | | | 271 | | | |
| | | | | | | | a134 | 245.60 | a134 blows for 0.4 ft. —138.40<br>Top of Bedrock at Elev. 245.6 ft. | |
| 140 | R1 | 60/58 | 140.20 - 145.20 | RQD = 93% | | | NQ-2 | | R1:Bedrock: Light grey, medium grained, muscovite GRANITE, hard, fresh, massive.<br>Rock Mass Quality = Excellent.<br>R1:Core Times (min:sec)<br>140.2-141.2 ft (2:00)<br>141.2-142.2 ft (2:00)<br>142.2-143.2 ft (2:00)<br>143.2-144.2 ft (1:55)<br>144.2-145.2 ft (2:10) 97% Recovery | |
| 145 | R2 | 60/59 | 145.20 - 150.20 | RQD = 93% | | | | | R2:Bedrock: Same as R1.<br>Rock Mass Quality = Excellent.<br>R2:Core Times (min:sec)<br>145.2-146.2 ft (2:00)<br>146.2-147.2 ft (2:00)<br>147.2-148.2 ft (2:00)<br>148.2-149.2 ft (1:55)<br>149.2-150.2 ft (2:10) 98% Recovery | |
| 150 | | | | | | | | | | |

Remarks:
100-200# down pressure on core barrel.
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.



Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over
Location: Lexington Township, Maine
Boring No.: BB-LSS-102
WIN: 19291.00

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|
| 150 | | | | | | | | 233.80 | —150.20<br>Bottom of Exploration at 150.20 feet below ground surface. | |
| 155 | | | | | | | | | | |
| 160 | | | | | | | | | | |
| 165 | | | | | | | | | | |
| 170 | | | | | | | | | | |
| 175 | | | | | | | | | | |

Remarks:
100-200# down pressure on core barrel.
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1929(100)X

BRIDGE NO. 5128 | WIN 19291.00 | BRIDGE PLANS

| | | BY | DATE |
|---|---|---|---|
| PROJ. MANAGER | S.BODGE | | |
| DESIGN-DETAILED | R. MYERS | B. NICHOLS | |
| CHECKED-REVIEWED | ----- | | ----- |
| DESIGN2-DETAILED2 | K.MAGUIRE | T.WHITE | DEC 2012 |
| DESIGN3-DETAILED3 | ----- | | |
| REVISIONS 1 | ----- | | ----- |
| REVISIONS 2 | ----- | | ----- |
| REVISIONS 3 | ----- | | ----- |
| REVISIONS 4 | ----- | | ----- |
| FIELD CHANGES | | | |

SIGNATURE

P.E. NUMBER

DATE

LOWER SANDY STREAM BRIDGE
SANDY STREAM
LEXINGTON TWP SOMERSET 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

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

## TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

**Rock Quality Designation (RQD):**

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

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

Correlation of RQD to Rock Mass Quality

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

**Desired Rock Observations: (in this order)**

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

**Sample Container Labeling Requirements:**

PIN
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:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-101
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.4 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 383.98 | | 5" Pavement —0.42 | |
| | 1D | 24/12 | 1.00 - 3.00 | 5/5/5/6 | 10 | 14 | | | | Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill). | |
| | | | | | | | | 380.40 | | —4.00 | |
| 5 | 2D | 24/14 | 5.00 - 7.00 | 2/3/3/2 | 6 | 8 | 18 | | | Brown, moist, loose, fine to coarse SAND, little gravel, some silt, (Fill). | |
| | | | | | | | 11 | | | | |
| | | | | | | | 11 | | | | |
| | | | | | | | 8 | 375.40 | | —9.00 | |
| | | | | | | | 11 | | | | |
| 10 | 3D | 24/14 | 10.00 - 12.00 | 1/2/1/2 | 3 | 4 | 13 | | | Grey, wet, very loose, fine SAND, trace medium sand, little silt, (Fill) | G#261888 A-2-4, SM WC=31.1% |
| | | | | | | | 12 | | | | |
| | | | | | | | 16 | | | | |
| | | | | | | | 19 | 370.40 | | —14.00 | |
| | | | | | | | 25 | | | | |
| 15 | 4D | 24/13 | 15.00 - 17.00 | 3/4/3/4 | 7 | 10 | 15 | | | Grey, wet, loose, fine to coarse SAND, trace gravel, trace wood fragments, (piles, cribbing?), (Reworked Alluvial). | |
| | | | | | | | 26 | | | | |
| | | | | | | | 26 | | | | |
| | | | | | | | 25 | | | | |
| | | | | | | | 36 | 364.40 | | —20.00 | |
| 20 | 5D | 24/4 | 20.70 - 22.70 | 1/1/2/2 | 3 | 4 | 23 | 363.70 | | WOOD layer from 20.0-20.7 ft bgs. —20.70 | |
| | | | | | | | 28 | | | Grey, wet, very loose, gravelly, fine to coarse SAND, trace silt. | |
| | | | | | | | 33 | | | | |
| | | | | | | | 35 | | | | |
| 25 | | | | | | | 53 | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-101
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.4 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | 6D | 24/14 | 25.00 - 27.00 | 2/2/2/2 | 4 | 6 | 41 | | | Grey, wet, loose, fine SAND, trace medium to coarse sand, trace silt, trace clay, trace gravel, (Regressive Marine Delta Deposits) | G#261889 A-2-4, SP-SC WC=19.9% |
| | | | | | | | 54 | | | | |
| | | | | | | | 73 | | | | |
| | | | | | | | 79 | | | | |
| | | | | | | | 84 | | | | |
| 30 | 7D | 24/18 | 30.00 - 32.00 | 2/2/2/2 | 4 | 6 | 46 | | | Grey, wet, loose, fine to medium SAND, trace coarse sand, trace silt, trace gravel. | G#261890 A-3, SP-SM WC=18.4% |
| | | | | | | | 63 | | | | |
| | | | | | | | 92 | | | | |
| | | | | | | | 126 | | | | |
| | | | | | | | 131 | | | | |
| 35 | 8D | 24/17 | 35.00 - 37.00 | 2/2/3/3 | 5 | 7 | 78 | | | Grey, wet, loose, fine to medium SAND, trace coarse sand, little silt, trace gravel. | G#261891 A-2-4, SM WC=20.2% |
| | | | | | | | 83 | | | | |
| | | | | | | | 99 | | | | |
| | | | | | | | 114 | | | | |
| | | | | | | | 156 | | | | |
| 40 | 9D | 24/20 | 40.00 - 42.00 | 4/4/4/4 | 8 | 11 | 8 | | | Similar to above, except medium dense. Switched to NW casing at 40.0 ft bgs. | |
| | | | | | | | 11 | | | | |
| | | | | | | | 32 | | | | |
| | | | | | | | 52 | | | | |
| | | | | | | | 56 | | | | |
| 45 | 10D | 24/18 | 45.00 - 47.00 | 1/2/3/3 | 5 | 7 | 21 | | | Similar to above, except loose. | |
| | | | | | | | 72 | | | | |
| | | | | | | | 193 | | | 2" running sand | |
| | | | | | | | 219 | | | | |
| 50 | | | | | | | 224 | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on boring.

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# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream | **Boring No.:** | BB-LSS-101 | | |
| **Location:** | Lexington Township, Maine | **WIN:** | 19291.00 | | |
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.4 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 11D | 24/21 | 50.00 - 52.00 | 2/3/5/6 | 8 | 11 | 90 | | | Grey, wet, medium dense, fine to medium SAND, little silt, trace coarse sand, trace gravel, trace clay. | G#261892 A-2-4, SC-SM WC=23.2% |
| | | | | | | | 76 | | | | |
| | | | | | | | 176 | | | | |
| | | | | | | | 221 | | | | |
| | | | | | | | 230 | | | | |
| 55 | 12D | 24/18 | 55.00 - 57.00 | 3/4/4/5 | 8 | 11 | 122 | | | Similar to above. | |
| | | | | | | | 162 | | | | |
| | | | | | | | 232 | | | | |
| | | | | | | | 272 | 326.40 | | 58.00 | |
| | | | | | | | 281 | | | | |
| 60 | 13D | 24/20 | 60.00 - 62.00 | 5/6/5/6 | 11 | 15 | 135 | | | Grey, wet, medium dense, Silty fine SAND. | |
| | | | | | | | 129 | | | | |
| | | | | | | | 165 | | | | |
| | | | | | | | 230 | | | | |
| | | | | | | | 281 | | | | |
| 65 | 14D | 24/15 | 65.00 - 67.00 | 4/4/4/5 | 8 | 11 | 165 | | | Grey, wet, medium dense, Silty fine SAND. | |
| | | | | | | | 162 | | | | |
| | | | | | | | 165 | | | | |
| | | | | | | | 170 | | | | |
| | | | | | | | 216 | | | | |
| 70 | 15D | 24/19 | 70.50 - 72.50 | 4/2/6/6 | 8 | 11 | 195 | 314.40 | | 70.00 Grey, wet, stiff, SILT, little fine sand, trace clay. | G#261893 A-4, ML WC=21.3% |
| | | | | | | | 99 | | | | |
| | | | | | | | 101 | | | | |
| | | | | | | | 93 | | | | |
| 75 | | | | | | | 99 | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on boring.

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# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-101
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
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| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 75 | 16D | 24/20 | 75.00 - 77.00 | 3/2/WOR/WOR | 2 | 3 | 132 | | | | |
| | | | | | | | 107 | 308.40 | | 76.00 Grey, wet, soft, Clayey SILT, little fine sand, (Glaciomarine Deposit) | |
| | | | | | | | 103 | | | | |
| | | | | | | | 103 | | | | |
| | | | | | | | 106 | | | | |
| 80 | 17D<br>~~V1~~ | 24/22 | 80.00 - 82.00<br>~~80.63 - 81.00~~ | WOR/WOR/WOR/ WOR<br>Su=1652/134 psf | --- | | 139 | | | Grey, wet, stiff, Clayey SILT, trace fine sand.<br>55x110 mm vane raw torque readings:<br>V1: 37.0/3.0 ft-lbs | G#261894<br>A-4, CL-ML<br>WC=28.0% |
| | V2 | | 81.63 - 82.00 | Su=1384/112 psf | | | 127 | | | V2: 31.0/2.5 ft-lbs | LL=26<br>PL=19<br>PI=7 |
| | | | | | | | 125 | | | | |
| | | | | | | | 126 | | | | |
| | | | | | | | 116 | | | | |
| 85 | | | | | | | 135 | | | Washed ahead to 86.0 ft bgs. | |
| | 18D<br>~~V3~~ | 24/20 | 86.00 - 88.00<br>~~86.63 - 87.00~~ | WOR/WOR/6/4<br>~~Su=915/223 psf~~ | 6 | 8 | 120 | | | 55x110 mm vane raw torque readings: | |
| | MV | | | Would not push | | | 77 | | | V3: 20.5/5.0 ft-lbs<br>Grey, wet, medium stiff, Clayey SILT, with 1"-2½" fine sand layers.<br>Failed 55x110 mm vane attempt. | |
| | | | | | | | 81 | | | | |
| | | | | | | | 76 | | | | |
| 90 | 19D<br>~~V4~~ | 24/24 | 90.00 - 92.00<br>~~90.63 - 91.00~~ | WOR/WOR/WOR/ WOR<br>Su=759/268 psf | --- | | 78 | | | Grey, wet, medium stiff, Clayey SILT, trace fine sand in layers.<br>55x110 mm vane raw torque readings:<br>V4: 17.0/6.0 ft-lbs | |
| | V5 | | 91.63 - 92.00 | Su=625/223 psf | | | 79 | | | V5: 14.0/5.0 ft-lbs | |
| | | | | | | | 78 | | | | |
| | | | | | | | 62 | | | | |
| | | | | | | | 63 | | | | |
| 95 | 20D<br>~~V6~~ | 24/20 | 95.00 - 97.00<br>~~95.63 - 96.00~~ | WOR/WOR/WOR/ WOR<br>Su=714/179 psf | --- | | 90 | | | Similar to above.<br>55x110 mm vane raw torque readings:<br>V6: 16.0/4.0 ft-lbs | |
| | MV | | | Would not push | | | 76 | | | Failed 55x110 mm vane attempt. | |
| | | | | | | | 67 | | | | |
| | | | | | | | 65 | | | | |
| 100 | | | | | | | 70 | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-101
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.4 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
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U = Thin Wall Tube Sample
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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. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 100 | 21D | 24/24 | 100.00 - 102.00 | WOR/WOR/WOR/ WOR | --- | | 81 | | | Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand in layers. | G#261895 A-6, CL WC=27.7% LL=29 PL=15 PI=14 |
| | V7 | | 100.63 - 101.00 | Su=937/246 psf | | | | | | 55x110 mm vane raw torque readings: V7: 21.0/5.5 ft-lbs | |
| | V8 | | 101.63 - 102.00 | Su=1161/268 psf | | | 73 | | | V8: 26.0/6.0 ft-lbs | |
| | | | | | | | 70 | | | | |
| | | | | | | | 76 | | | | |
| | | | | | | | 78 | | | | |
| 105 | 22D | 24/20 | 105.00 - 107.00 | WOR/WOR/WOR/ WOR | --- | | 113 | | | Grey, wet, very soft, SILT, some clay, trace fine sand. | G#261896 A-4, CL-ML WC=26.5% LL=27 PL=21 PI=6 |
| | MV | | | Would not push | | | 111 | | | Failed 55x110 mm vane attempt. | |
| | | | | | | | 81 | | | | |
| | | | | | | | 83 | | | | |
| | | | | | | | 83 | | | | |
| 110 | 23D | 24/24 | 110.00 - 112.00 | WOR/WOR/WOR/ WOR | --- | | 116 | | | Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand. | |
| | V9 | | 110.63 - 111.00 | Su=1116/312 psf | | | | | | 55x110 mm vane raw torque readings: V9: 25.0/7.0 ft-lbs | |
| | V10 | | 111.63 - 112.00 | Su=982/246 psf | | | 85 | | | V10: 22.0/5.5 ft-lbs | |
| | | | | | | | 79 | | | | |
| | | | | | | | 79 | | | | |
| | | | | | | | 108 | | | | |
| 115 | 24D | 24/24 | 115.00 - 117.00 | WOR/WOR/WOR/ WOR | --- | | 116 | | | Similar to above, stiff. | |
| | V11 | | 115.63 - 116.00 | Su=1161/402 psf | | | | | | 55x110 mm vane raw torque readings: V11: 26.0-9.0 ft-lbs | |
| | MV | | | Would not push | | | 117 | | | Failed 55x110 mm vane attempt. | |
| | | | | | | | 111 | | | | |
| | | | | | | | 115 | | | | |
| | | | | | | | 119 | | | | |
| 120 | 25D | 24/17 | 120.00 - 122.00 | WOR/WOR/WOR/ WOR | --- | | 130 | | | Similar to above. | |
| | V12 | | 120.63 - 121.00 | Su=1339/246 psf | | | 126 | | | 55x110 mm vane raw torque readings: V12: 30.0/5.5 ft-lbs | |
| | | | | | | | 111 | | | | |
| | | | | | | | 110 | | | | |
| | | | | | | | 127 | 260.40 | | 124.00 | |
| 125 | | | | | | | | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-101
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.4 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/2,3,7,8/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 3+56.2, 7.4 ft Rt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 15.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 125 | 26D | 24/18 | 125.00 - 127.00 | 6/10/10/11 | 20 | 28 | 135 | | | Grey, wet, medium dense, fine to coarse SAND, some silt, trace gravel, (Submarine outwash). | G#261897 A-2-4, SM WC=14.7% |
| | | | | | | | 116 | | | | |
| | | | | | | | 111 | | | | |
| | | | | | | | 84 | | | | |
| | | | | | | | 111 | | | | |
| 130 | | | | | | | 94 | | | | |
| | | | | | | | a110 | 253.00 | | a110 blows for 0.4 ft. 131.40 | |
| | R1 | 60/60 | 132.10 - 137.10 | RQD = 100% | | | NQ-2 | | | Top of Bedrock at Elev. 253.0 ft.<br>Roller Coned ahead to 132.1 ft bgs.<br>R1:Bedrock: White to light grey, medium-grained, muscovite GRANITE, hard, fresh, massive.<br>Rock Mass Quality = Excellent.<br>R1:Core Times (min:sec)<br>132.1-133.1 ft (2:00)<br>133.1-134.1 ft (2:15)<br>134.1-135.1 ft (2:00)<br>135.1-136.1 ft (2:00)<br>136.1-137.1 ft (2:15) 100% Recovery | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 135 | | | | | | | | | | | |
| | | | | | | | | | | | |
| | R2 | 60/60 | 137.10 - 142.10 | RQD = 100% | | | | | | R2:Bedrock: Similar to R1.<br>Rock Mass Quality = Excellent.<br>R2:Core Times (min:sec)<br>137.1-138.1 ft (2:00)<br>138.1-139.1 ft (1:50)<br>139.1-140.1 ft (1:40)<br>140.1-141.1 ft (1:40)<br>141.1-142.1 ft (1:55) 100% Recovery | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 140 | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | 242.30 | | 142.10<br>**Bottom of Exploration at 142.10 feet below ground surface.** | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 145 | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 150 | | | | | | | | | | | |

**Remarks:**

100-150# down pressure on core barrel.
Running sand kept ahead of water on 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.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
D = Split Spoon Sample
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U = Thin Wall Tube Sample
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G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 383.58 | | 5" Pavement —0.42 | |
| | | | | | | | | | | Brown, damp, gravelly, fine to coarse SAND, trace silt, (Fill.) | |
| 5 | 1D | 24/13 | 5.00 - 7.00 | 1/2/3/1 | 5 | 7 | | | | Brown, damp, loose, fine to coarse SAND, some silt, (Fill). | |
| | | | | | | | | 375.50 | | —8.50 | |
| 10 | 2D | 24/24 | 10.00 - 12.00 | WOH/WOH/WOH/ WOH | --- | | 7 | | | Olive-grey, wet, very soft, SILT, some fine sand, trace clay, (Fill; Reworked native soils).<br><br>Set in HW Casing at 10.0 ft bgs. | G#261898 A-4, ML WC=40.4% |
| | | | | | | | 8 | | | | |
| | | | | | | | 8 | | | | |
| | | | | | | | 11 | | | | |
| | | | | | | | 28 | 370.00 | | —14.00 | |
| 15 | 3D | 24/16 | 15.00 - 17.00 | 3/3/3/2 | 6 | 8 | 33 | | | Grey, wet, loose, fine to coarse SAND, trace silt, (Fill; Reworked native soils). | |
| | | | | | | | 27 | | | | |
| | | | | | | | 32 | | | | |
| | | | | | | | 33 | | | | |
| | | | | | | | 54 | 365.00 | | —19.00 | |
| 20 | 4D | 24/13 | 20.00 - 22.00 | 2/3/5/5 | 8 | 11 | 62 | | | Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt, trace wood fragments, (Alluvium; reworked?). | |
| | | | | | | | 62 | | | | |
| | | | | | | | 60 | | | | |
| | | | | | | | 58 | | | | |
| 25 | | | | | | | 59 | | | | |

**Remarks:**

100-200# down pressure on core barrel.

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Soil/Rock Exploration Log
US CUSTOMARY UNITS

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**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | 5D | 24/12 | 25.00 - 27.00 | 2/3/2/2 | 5 | 7 | 57 | | | Similar to above, except loose. | |
| | | | | | | | 60 | | | | |
| | | | | | | | 59 | | | | |
| | | | | | | | 67 | | | | |
| | | | | | | | 60 | 355.00 | | 29.00 | |
| 30 | 6D | 24/15 | 30.00 - 32.00 | 3/2/3/3 | 5 | 7 | 62 | | | Grey, wet, loose, fine SAND, little silt, trace gravel, (Regressive Marine Delta Deposits) | G#261899 A-2-4, SM WC=22.6% |
| | | | | | | | 72 | | | | |
| | | | | | | | 78 | | | | |
| | | | | | | | 73 | | | | |
| | | | | | | | 75 | | | | |
| 35 | 7D | 24/17 | 35.00 - 37.00 | 1/2/3/2 | 5 | 7 | 101 | | | Grey, wet, loose, fine to medium SAND, trace silt, trace gravel, trace coarse sand, (Regressive Marine Delta Deposits). | |
| | | | | | | | 79 | | | | |
| | | | | | | | 79 | | | | |
| | | | | | | | 104 | | | | |
| | | | | | | | 122 | | | | |
| 40 | 8D | 24/18 | 40.00 - 42.00 | 3/3/3/3 | 6 | 8 | 110 | | | Grey, wet, loose, fine to medium SAND, trace silt, trace gravel, trace coarse sand. | G#261900 A-3, SP WC=18.8% |
| | | | | | | | 111 | | | | |
| | | | | | | | 111 | | | | |
| | | | | | | | 111 | | | | |
| | | | | | | | 112 | | | | |
| 45 | 9D | 24/16 | 45.00 - 47.00 | 1/2/2/2 | 4 | 6 | 116 | | | Similar to above. | |
| | | | | | | | 131 | | | | |
| | | | | | | | 169 | | | | |
| | | | | | | | 222 | | | | |
| 50 | | | | | | | 243 | | | | |

**Remarks:**

100-200# down pressure on core barrel.

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

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

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 10D | 24/19 | 50.00 - 52.00 | 2/2/2/2 | 4 | 6 | 126 | | | Grey, wet, loose, fine to medium SAND, trace silt, trace coarse sand, trace gravel. | |
| | | | | | | | 162 | | | | |
| | | | | | | | 221 | | | | |
| | | | | | | | 283 | 331.00 | | 53.00 | |
| | | | | | | | 354 | | | | |
| 55 | 11D | 24/17 | 55.00 - 57.00 | 5/5/4/4 | 9 | 13 | 180 | | | Grey, wet, stiff, SILT, some fine sand, trace clay, trace medium sand, trace wood fragments, (Marine Delta Deposits). | |
| | | | | | | | 163 | | | | |
| | | | | | | | 229 | | | | |
| | | | | | | | 336 | | | | |
| | | | | | | | 396 | | | | |
| 60 | 12D | 24/20 | 60.00 - 62.00 | 3/4/5/5 | 9 | 13 | 231 | | | Grey, wet, stiff, SILT, some fine sand, trace medium sand, trace clay. | G#267501 A-4, ML WC=21.2% |
| | | | | | | | 229 | | | | |
| | | | | | | | 287 | | | | |
| | | | | | | | 384 | | | | |
| | | | | | | | 393 | | | | |
| 65 | 13D | 24/19 | 65.00 - 67.00 | 2/2/3/4 | 5 | 7 | 273 | | | Similar to above, except medium stiff. | |
| | | | | | | | 242 | 317.50 | | 66.50 Grey, wet, loose, fine to medium SAND, some silt, trace clay. | |
| | | | | | | | 233 | | | | |
| | | | | | | | 298 | | | | |
| | | | | | | | 338 | | | | |
| 70 | 14D | 24/18 | 70.50 - 72.50 | 3/5/8/8 | 13 | 18 | 277 | | | Grey, wet, medium dense, fine SAND, some silt, trace medium sand, trace clay. Roller coned ahead to 75.0 ft bgs. | G#267502 A-4, SC-SM WC=22.3% |
| | | | | | | | 119 | | | | |
| | | | | | | | 119 | | | | |
| | | | | | | | 122 | | | | |
| 75 | | | | | | | 132 | | | | |

**Remarks:**

100-200# down pressure on core barrel.

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# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 75 | 15D | 24/17 | 75.00 - 77.00 | WOR/WOR/3/4 | 3 | 4 | 176 | | | Similar to above, except very loose. | |
| | | | | | | | 218 | | | | |
| | | | | | | | 193 | | | | |
| | | | | | | | 216 | 306.00 | | 78.00 | |
| | | | | | | | 224 | | | | |
| 80 | 16D | 24/24 | 80.00 - 82.00 | WOR/WOR/WOR/ WOH | --- | | OPEN HOLE | | | Grey, wet, very soft, Clayey SILT, trace fine sand, (Glaciomarine Deposit). | |
| 85 | 1U | 24/24 | 85.00 - 87.00 | WOR/WOR | --- | | | | | Grey, wet, very soft, Clayey SILT with fine sand layers, (Glaciomarine Deposit). | |
| | V1 | | 87.63 - 88.00 | Su=1116/179 psf | | | | | | 55x110 mm vane raw torque readings: | |
| | V2 | | 88.63 - 89.00 | Su=491/134 psf | | | | | | V1: 25.0/4.0 ft-lbs<br>V2: 11.0/3.0 ft-lbs | |
| 90 | 17D | 24/24 | 90.00 - 92.00 | WOR/WOR/WOR/ WOR | --- | | | | | Grey, wet, stiff, Clayey SILT, trace sand, (Glaciomarine Deposit). | G#267504<br>A-4, ML<br>WC=27.9%<br>LL=25<br>PL=23<br>PI=2 |
| | V3 | | 90.63 - 91.00 | Su=1049/156 psf | | | | | | 55x110 mm vane raw torque readings:<br>V3: 23.5/3.5 ft-lbs | |
| | V4 | | 91.63 - 92.00 | Su=1384/223 psf | | | | | | V4: 31.0/5.0 ft-lbs | |
| 95 | V5 | | 95.00 - 95.37 | Su=1451/179 psf | | | | | | 55x110 mm vane raw torque readings:<br>V5: 32.5/4.0 ft-lbs | |
| | V6 | | 96.00 - 96.37 | Su=1406/312 psf | | | | | | V6: 31.5/7.0 ft-lbs | |
| | 2U | 24/24 | 97.00 - 99.00 | WOR/WOR | --- | | | | | Similar to above. | |
| 100 | | | | | | | | | | | |

**Remarks:**

100-200# down pressure on core barrel.

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# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 100 | 18D | 24/24 | 100.00 - 102.00 | WOR/WOR/WOR/ WOR | --- | | | | | 55x110 mm vane raw torque readings: | |
| | V7 | | 100.63 - 101.00 | Su=982/223 psf | | | | | | V7: 22.0/5.0 ft-lbs | G#267503 |
| | V8 | | 101.63 - 102.00 | Su=580/223 psf | | | | | | Grey, wet, medium stiff, SILT, some clay, trace fine sand. V8: 13.0./5.0 ft-lbs | A-4, CL-ML WC=27.1% LL=28 PL=21 PI=7 |
| 105 | 3U | 24/24 | 105.00 - 107.00 | WOR/WOR | --- | | | | | Similar to above. | |
| | V9 | | 107.00 - 107.37 | Su=402/134 psf | | | | | | 55x110 mm vane raw torque readings: V9: 29.0/3.0 ft-lbs | |
| | V10 | | 108.00 - 108.37 | Su=1161/312 psf | | | | | | V10: 26.0/7.0 ft-lbs | |
| 110 | 19D | 24/20 | 110.00 - 112.00 | WOR/WOR/2/2 | 2 | 3 | 15 | | | Set in NW Casing at 110.0 ft bgs. 55x110 mm vane raw torque readings: | |
| | V11 | | 110.63 - 111.00 | Su=1161/312 psf | | | | 273.00 | | V11: 26.0/7.0 ft-lbs | |
| | MV | | | Would not push | | | 16 | | | 111.00 Grey, wet, very loose, fine SAND, trace silt, (Lower Marine Sands). Failed 55x110 mm vane attempt. | |
| | | | | | | | 14 | | | | |
| | | | | | | | 16 | | | | |
| | | | | | | | 16 | | | | |
| 115 | 20D | 24/22 | 115.00 - 117.00 | WOR/WOH/WOH/ WOH | --- | | 23 | | | Grey, wet, very loose, Silty fine to medium SAND, (Lower Marine Sands). | |
| | | | | | | | 32 | | | | |
| | | | | | | | 35 | | | | |
| | | | | | | | 30 | | | | |
| | | | | | | | 21 | | | | |
| 120 | 21D | 24/21 | 120.00 - 122.00 | 1/3/4/6 | 7 | 10 | 59 | | | Grey, wet, loose, fine to coarse SAND, little silt, trace gravel, (Lower Marine Sands). | G#267505 A-2-4, SM WC=12.0% |
| | | | | | | | 55 | | | | |
| | | | | | | | 62 | | | | |
| | | | | | | | 48 | | | | |
| 125 | | | | | | | 37 | | | | |

**Remarks:**

100-200# down pressure on core barrel.

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# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream
**Location:** Lexington Township, Maine

**Boring No.:** BB-LSS-102
**WIN:** 19291.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

Definitions:
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$N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 125 | 22D | 24/22 | 125.00 - 127.00 | 1/5/11/15 | 16 | 22 | 57 | | | Similar to above, except medium dense. | |
| | | | | | | | 121 | | | | |
| | | | | | | | 236 | 257.00 | | 127.00 | |
| | | | | | | | 279 | | | Roller Coned ahead from 127.5-130.0 ft bgs. | |
| | | | | | | | 299 | | | | |
| 130 | 23D | 14.4/7 | 130.00 - 131.20 | 15/14/40(2.4") | --- | | 283 | | | Grey, wet, very dense, Gravelly, fine to coarse SAND, little silt, with cobbles. (Glacial Till). | |
| | | | | | | | 150 | | | | |
| | | | | | | | 191 | | | | |
| | | | | | | | 195 | | | Roller Coned ahead from 133.0-135.5 ft bgs. | |
| | | | | | | | 186 | | | | |
| 135 | MD | 24/0 | 135.50 - 137.50 | 50/36/40/58 | 76 | 106 | 182 | | | Roller Coned ahead to 140.2 ft bgs. | |
| | | | | | | | 184 | | | | |
| | | | | | | | 271 | | | | |
| | | | | | | | a134 | 245.60 | | a134 blows for 0.4 ft. 138.40 | |
| | | | | | | | | | | Top of Bedrock at Elev. 245.6 ft. | |
| 140 | R1 | 60/58 | 140.20 - 145.20 | RQD = 93% | | | NQ-2 | | | R1:Bedrock: Light grey, medium grained, muscovite GRANITE, hard, fresh, massive.<br>Rock Mass Quality = Excellent.<br>R1:Core Times (min:sec)<br>140.2-141.2 ft (2:00)<br>141.2-142.2 ft (2:00)<br>142.2-143.2 ft (2:00)<br>143.2-144.2 ft (1:55)<br>144.2-145.2 ft (2:10) 97% Recovery | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 145 | R2 | 60/59 | 145.20 - 150.20 | RQD = 93% | | | | | | R2:Bedrock: Same as R1.<br>Rock Mass Quality = Excellent.<br>R2:Core Times (min:sec)<br>145.2-146.2 ft (2:00)<br>146.2-147.2 ft (2:00)<br>147.2-148.2 ft (2:00)<br>148.2-149.2 ft (1:55)<br>149.2-150.2 ft (2:10) 98% Recovery | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 150 | | | | | | | | | | | |

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| Maine Department of Transportation<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | Project: Lower Sandy Stream Bridge #5128 carries Long Falls Dam Rd. over Sandy Stream<br>Location: Lexington Township, Maine | Boring No.: BB-LSS-102<br>WIN: 19291.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 384.0 | **Auger ID/OD:** | 5" Solid Stem |
| **Operator:** | Giguere/Giles/Daggett | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 140#/30" |
| **Date Start/Finish:** | 5/14,16,17,21/2012 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ-2" |
| **Boring Location:** | 4+65.4, 6.2 ft Lt. | **Casing ID/OD:** | HW & NW | **Water Level*:** | 14.0 ft bgs. |
| **Hammer Efficiency Factor:** | 0.84 | **Hammer Type:** | Automatic ☒ Hydraulic ☐ Rope & Cathead ☐ | | |

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| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-uncorrected | $N_{60}$ | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 150 | | | | | | | | 233.80 | | 150.20<br>**Bottom of Exploration at 150.20 feet below ground surface.** | |
| 155 | | | | | | | | | | | |
| 160 | | | | | | | | | | | |
| 165 | | | | | | | | | | | |
| 170 | | | | | | | | | | | |
| 175 | | | | | | | | | | | |

**Remarks:**

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# **Appendix B**

Laboratory Data

State of Maine - Department of Transportation

## Laboratory Testing Summary Sheet

# Town(s): Lexington Work Number: 19291.00

| 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-LSS-101, 3D | 3+56.2 | 7.4 Rt. | 10.0-12.0 | 261888 | 1 | 31.1 | | | SM | A-2-4 | II |
| BB-LSS-101, 6D | 3+56.2 | 7.4 Rt. | 25.0-27.0 | 261889 | 1 | 19.9 | | | SP-SC | A-2-4 | II |
| BB-LSS-101, 7D | 3+56.2 | 7.4 Rt. | 30.0-32.0 | 261890 | 1 | 18.4 | | | SP-SM | A-3 | 0 |
| BB-LSS-101, 8D | 3+56.2 | 7.4 Rt. | 35.0-37.0 | 261891 | 1 | 20.2 | | | SM | A-2-4 | II |
| BB-LSS-101, 11D | 3+56.2 | 7.4 Rt. | 50.0-52.0 | 261892 | 1 | 23.2 | | | SC-SM | A-2-4 | II |
| BB-LSS-101, 15D | 3+56.2 | 7.4 Rt. | 70.5-72.5 | 261893 | 2 | 21.3 | | | ML | A-4 | IV |
| BB-LSS-101, 17D | 3+56.2 | 7.4 Rt. | 80.0-82.0 | 261894 | 2 | 28.0 | 26 | 7 | CL-ML | A-4 | IV |
| BB-LSS-101, 21D | 3+56.2 | 7.4 Rt. | 100.0-102.0 | 261895 | 2 | 27.7 | 29 | 14 | CL | A-6 | III |
| BB-LSS-101, 22D | 3+56.2 | 7.4 Rt. | 105.0-107.0 | 261896 | 2 | 26.5 | 27 | 6 | CL-ML | A-4 | IV |
| BB-LSS-101, 26D | 3+56.2 | 7.4 Rt. | 125.0-127.0 | 261897 | 2 | 14.7 | | | SM | A-2-4 | II |
| BB-LSS-102, 2D | 4+65.4 | 6.2 Lt. | 10.0-12.0 | 261898 | 3 | 40.4 | | | ML | A-4 | IV |
| BB-LSS-102, 6D | 4+65.4 | 6.2 Lt. | 30.0-32.0 | 261899 | 3 | 22.6 | | | SM | A-2-4 | II |
| BB-LSS-102, 8D | 4+65.4 | 6.2 Lt. | 40.0-42.0 | 261900 | 3 | 18.8 | | | SP | A-3 | 0 |
| BB-LSS-102, 12D | 4+65.4 | 6.2 Lt. | 60.0-62.0 | 267501 | 3 | 21.2 | | | ML | A-4 | IV |
| BB-LSS-102, 14D | 4+65.4 | 6.2 Lt. | 70.5-72.5 | 267502 | 4 | 22.3 | | | SC-SM | A-4 | IV |
| BB-LSS-102, 17D | 4+65.4 | 6.2 Lt. | 90.0-92.0 | 267504 | 4 | 27.9 | 25 | 2 | ML | A-4 | IV |
| BB-LSS-102, 18D | 4+65.4 | 6.2 Lt. | 101.0-102.0 | 267503 | 4 | 27.1 | 28 | 7 | CL-ML | A-4 | IV |
| BB-LSS-102, 21D | 4+65.4 | 6.2 Lt. | 120.0-122.0 | 267505 | 4 | 12.0 | | | SM | A-2-4 | II |
| Grab Sample #1 | 3+59.1 | 42.9 Lt. | 0.0-0.4 | 261886 | 5 | 35.7 | | | SC-SM | A-2-4 | II |
| Grab Sample #2 | 3+59.1 | 42.9 Lt. | 0.4-0.8 | 261887 | 5 | 36.2 | | | SC-SM | A-2-4 | II |

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

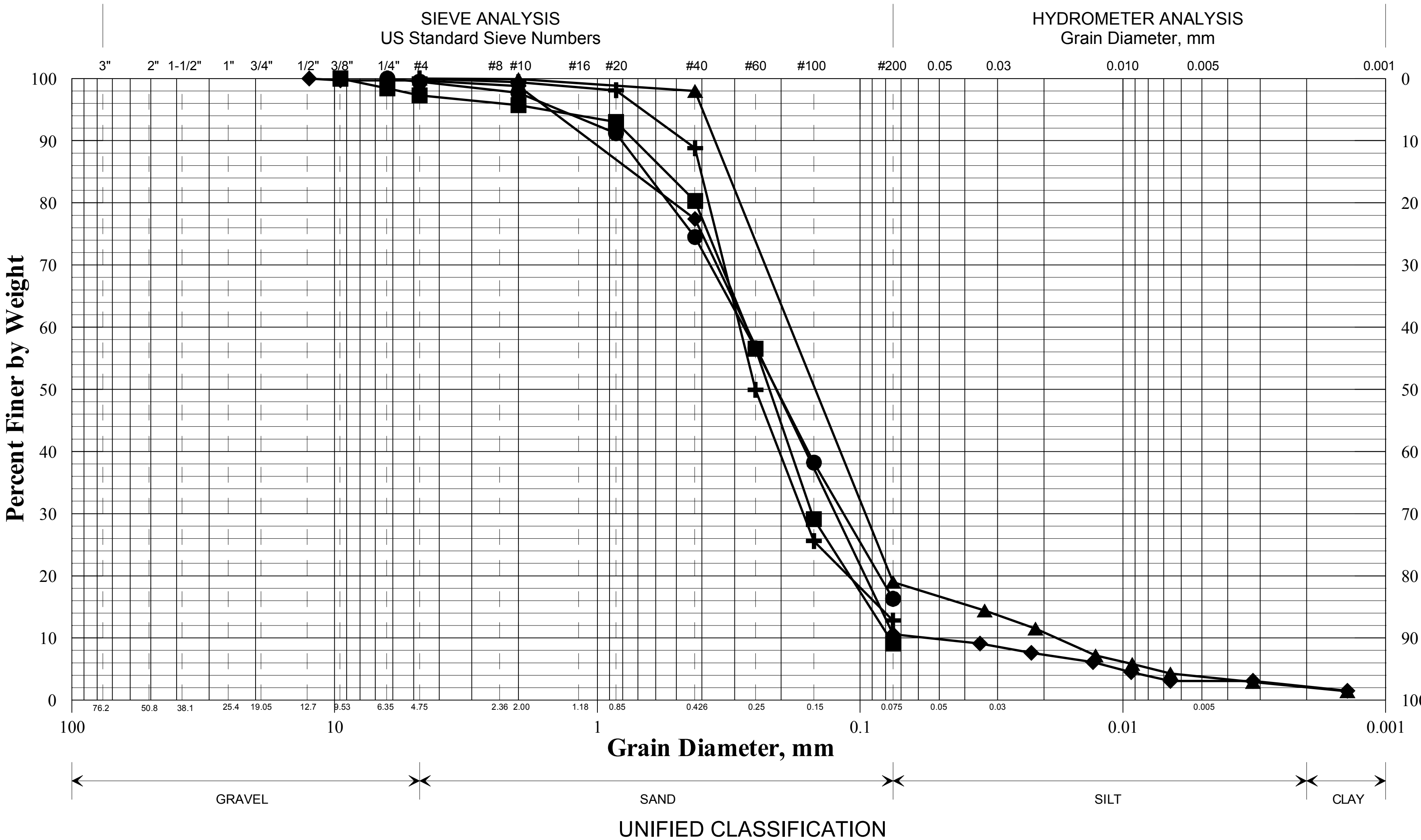
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

# *State of Maine Department of Transportation*
## GRAIN SIZE DISTRIBUTION CURVE



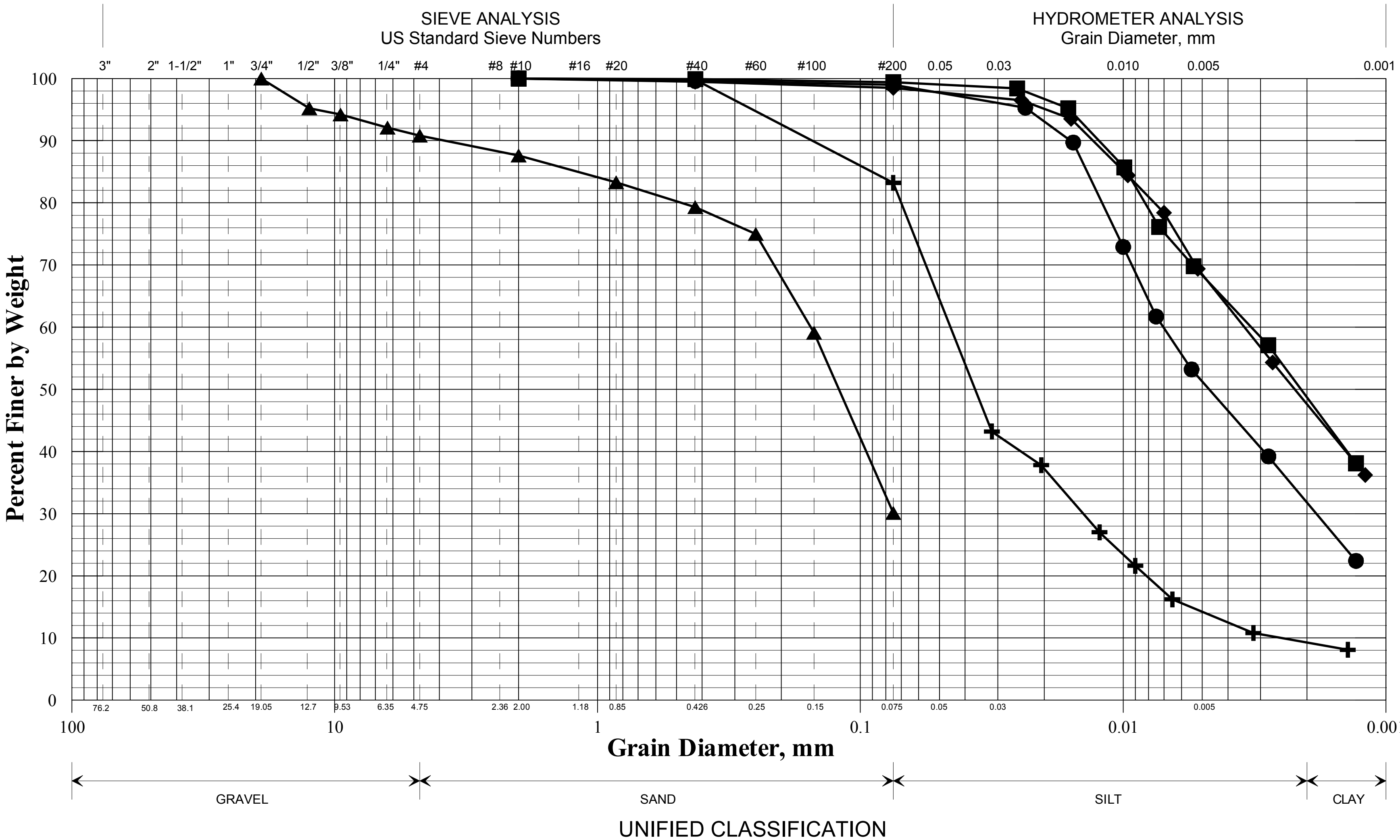

| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | BB-LSS-101/3D | 3+56.2 | 7.4 RT | 10.0-12.0 | SAND, little silt. | 31.1 | | | |
| ◆ | BB-LSS-101/6D | 3+56.2 | 7.4 RT | 25.0-27.0 | SAND, trace silt, trace clay, trace gravel. | 19.9 | | | |
| ■ | BB-LSS-101/7D | 3+56.2 | 7.4 RT | 30.0-32.0 | SAND, trace silt, trace gravel. | 18.4 | | | |
| ● | BB-LSS-101/8D | 3+56.2 | 7.4 RT | 35.0-37.0 | SAND, little silt, trace gravel. | 20.2 | | | |
| ▲ | BB-LSS-101/11D | 3+56.2 | 7.4 RT | 50.0-52.0 | SAND, little silt, trace gravel, trace clay. | 23.2 | | | |
| ✖ | | | | | | | | | |

| WIN |
|---|
| 019291.00 |
| **Town** |
| Lexington Twp |
| **Reported by/Date** |
| WHITE, TERRY A 6/8/2012 |

**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-LSS-101/15D | 3+56.2 | 7.4 RT | 70.5-72.5 | SILT, little sand, trace clay. | 21.3 | | | |
| ◆ | BB-LSS-101/17D | 3+56.2 | 7.4 RT | 80.0-82.0 | Clayey SILT, trace sand. | 28.0 | 26 | 19 | 7 |
| ■ | BB-LSS-101/21D | 3+56.2 | 7.4 RT | 100.0-102.0 | Clayey SILT, trace sand. | 27.7 | 29 | 15 | 14 |
| ● | BB-LSS-101/22D | 3+56.2 | 7.4 RT | 105.0-107.0 | SILT, some clay, trace sand. | 26.5 | 27 | 21 | 6 |
| ▲ | BB-LSS-101/26D | 3+56.2 | 7.4 RT | 125.0-127.0 | SAND, some silt, trace gravel. | 14.7 | | | |
| ✖ | | | | | | | | | |

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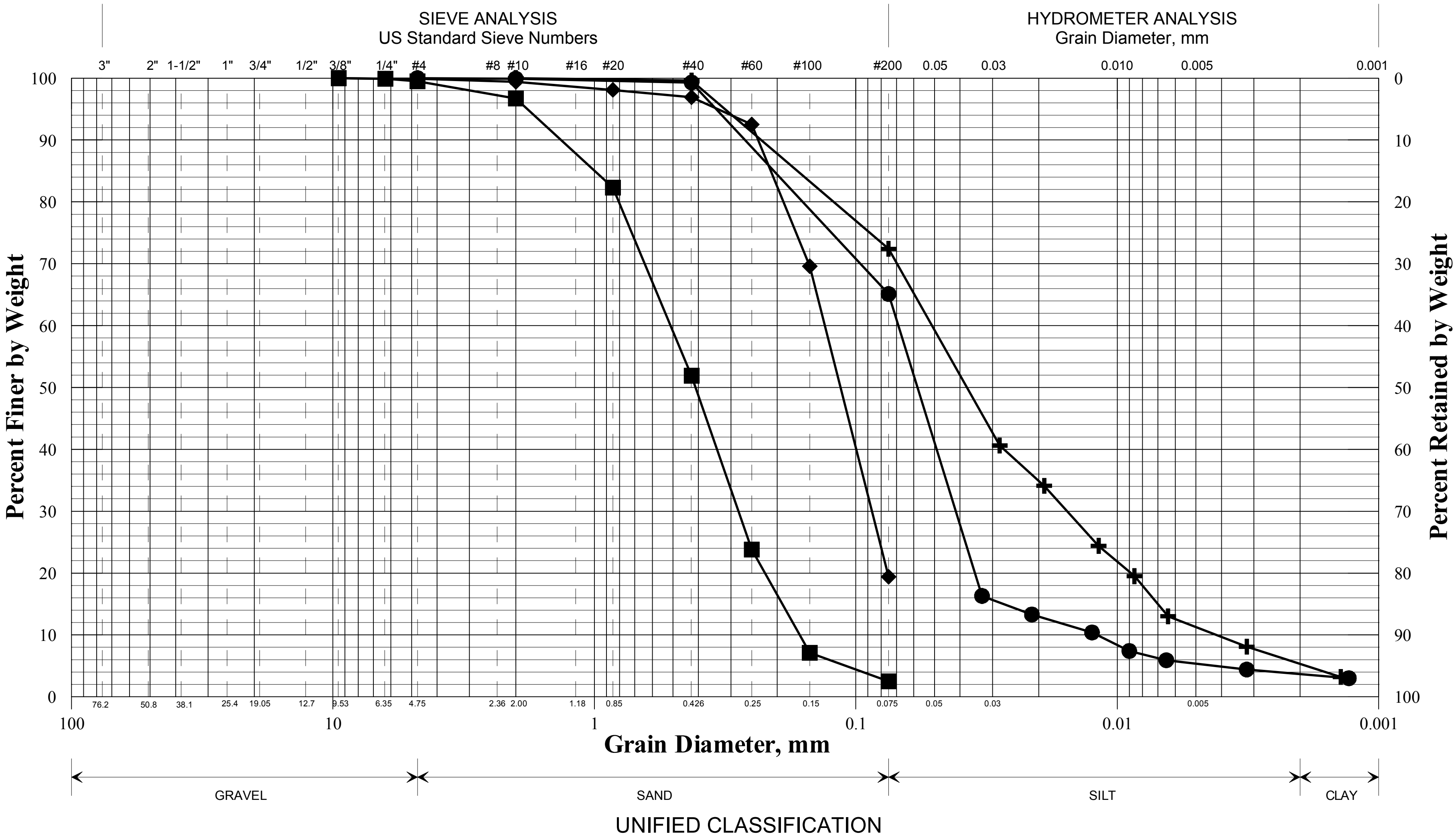
| 019291.00 |
| **Town** |
| Lexington Twp |
| **Reported by/Date** |
| WHITE, TERRY A 6/8/2012 |

**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-LSS-102/2D | 4+65.4 | 6.2 LT | 10.0-12.0 | SILT, some sand, trace clay. | 40.4 | | | |
| ◆ | BB-LSS-102/6D | 4+65.4 | 6.2 LT | 30.0-32.0 | SAND, little silt, trace gravel. | 22.6 | | | |
| ■ | BB-LSS-102/8D | 4+65.4 | 6.2 LT | 40.0-42.0 | SAND, trace silt, trace gravel. | 18.8 | | | |
| ● | BB-LSS-102/12D | 4+65.4 | 6.2 LT | 60.0-62.0 | SILT, some sand, trace clay. | 21.2 | | | |
| ▲ | | | | | | | | | |
| ✖ | | | | | | | | | |

| WIN |
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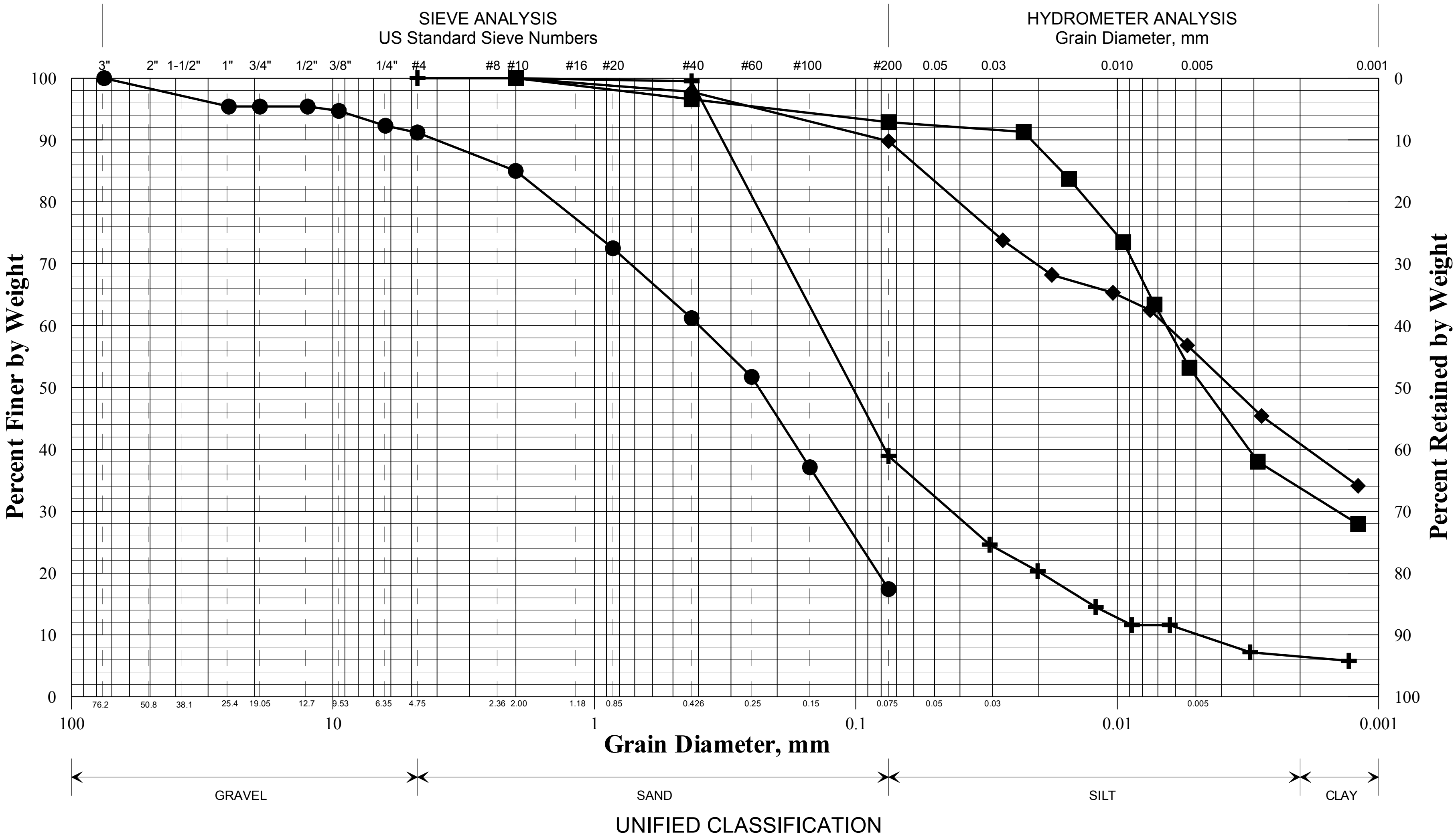
| 019291.00 |
| **Town** |
| Lexington Twp |
| **Reported by/Date** |
| WHITE, TERRY A 6/8/2012 |

**SHEET 3**

## *State of Maine Department of Transportation*
## GRAIN SIZE DISTRIBUTION CURVE




| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | BB-LSS-102/14D | 4+65.4 | 6.2 LT | 70.5-72.5 | SAND, some silt, trace clay. | 22.3 | | | |
| ◆ | BB-LSS-102/17D | 4+65.4 | 6.2 LT | 90.0-92.0 | Clayey SILT, trace sand. | 27.9 | 25 | 23 | 2 |
| ■ | BB-LSS-102/18D | 4+65.4 | 6.2 LT | 101.0-102.0 | SILT, some clay, trace sand. | 27.1 | 28 | 21 | 7 |
| ● | BB-LSS-102/21D | 4+65.4 | 6.2 LT | 120.0-122.0 | SAND, little silt, trace gravel. | 12.0 | | | |
| ▲ | | | | | | | | | |
| ✖ | | | | | | | | | |

| WIN |
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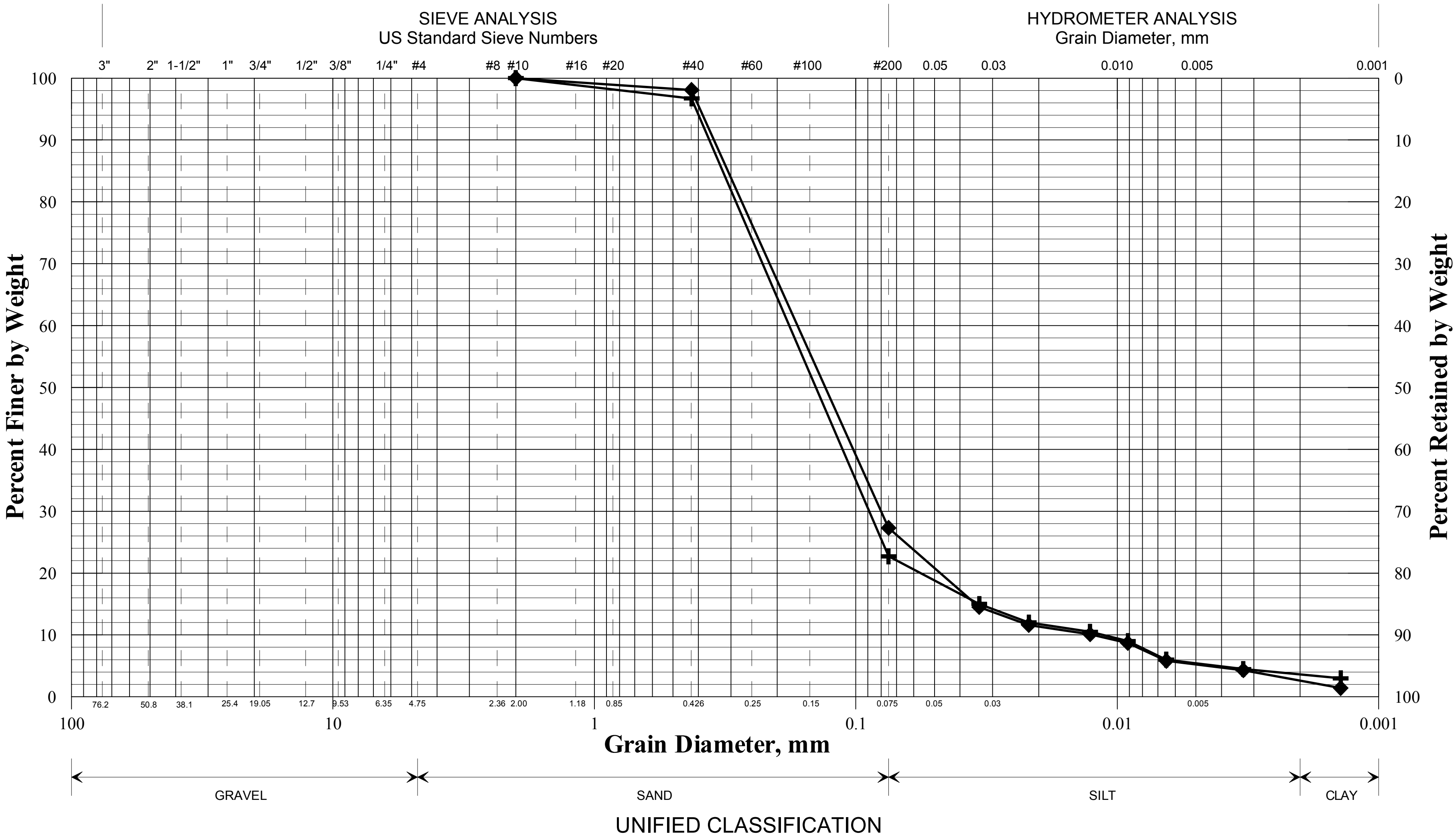
| 019291.00 |
| Town |
| Lexington Twp |
| Reported by/Date |
| WHITE, TERRY A 6/8/2012 |

## *State of Maine Department of Transportation*
## GRAIN SIZE DISTRIBUTION CURVE




| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | GRAB #1 | 3+59.1 | 42.9 LT | 0.0-0.4 | SAND, little silt, trace clay. | 35.7 | | | |
| ◆ | GRAB #2 | 3+59.1 | 42.9 LT | 0.4-0.8 | SAND, some silt, trace clay. | 36.2 | | | |
| ■ | | | | | | | | | |
| ● | | | | | | | | | |
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| WIN |
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| 019291.00 |
| Town |
| Lexington Twp |
| Reported by/Date |
| WHITE, TERRY A 6/7/2012 |

# Appendix C

Calculations

# LIQUIDITY INDEX (LI):

$$\text{Liquidity Index} = \frac{\text{natural water content - Plastic Limit}}{\text{Liquid Limit -Plastic Limit}}$$

| | |
|---|---|
| wc is close to LL | Soil is normally consolidated |
| wc is close to PL | Soil is some-to-heavily over consolidated |
| wc is intermediate | Soil is over consolidated |
| wc is greater than LL | Soil is on the verge of being a viscous liquid when remolded |

| Sample | WC | LL | PL | PI | Plasticity | LI | |
|---|---|---|---|---|---|---|---|
| BB-LSS-101/17D | 28.0 | 26 | 19 | 7 | low plasticity | 1.29 | viscous liquid when remolded |
| BB-LSS-101/21D | 27.7 | 29 | 15 | 14 | medium plasticity | 0.91 | normally consolidated |
| BB-LSS-101/22D | 26.5 | 27 | 21 | 6 | low plasticity | 0.92 | normally consolidated |
| BB-LSS-102/17D | 27.9 | 25 | 23 | 2 | slightly plastic | 2.45 | viscous liquid when remolded |
| BB-LSS-102/18D | 27.1 | 28 | 21 | 7 | low plasticity | 0.87 | normally consolidated |

# Abutment Foundations: Integral Driven H-piles

**Axial Structural Resistance of H-piles**

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

Look at the following piles:

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

yield strength: $F_y := 50 \cdot ksi$

Determine equivalent yield resistance $P_o = QF_yA_s$ LRFD Article 6.9.4.1.1

$Q := 1.0$ LRFD Article 6.9.4.2 $F_y = 50 \cdot ksi$

$P_o := Q \cdot F_y \cdot A_s$

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Determine elastic critical buckling resistance: $Pe = \pi^2 EA_s/(Kl/r_s)^2$ LRFD eq. 6.9.4.1.2-1

E = steel modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 1.2$ LRFD Table C4.6.2.5-1 Design value: ideal conditions, rotation fixed, translation free at head; rotation fixed, translation fixed at tip

l = unbraced length $l_{unbraced} := 12 \cdot in$ Assume 1 foot unbraced - scour (unlikely)

$r_s$ = radius of gyration $r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

LRFD Article C6.9.4.1.2 states that the critical flexural buckling resistances be calculated about the x- and y-axes with the smaller value taken as Pe.

Use y-axis as this results in the smaller value.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s}\right)^2} \cdot A_s\right]}$$

$$P_e = \begin{pmatrix} 174999 \\ 256564 \\ 359780 \\ 448914 \\ 611956 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 226 \\ 235 \\ 336 \\ 344 \\ 356 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

$$P_n := \overrightarrow{\left[\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o\right]}$$

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

# STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "good" based on borings.

**Strength Limit State** Axial Resistance factor for piles in compression under good driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$

**Factored** Compressive Resistance: eq. 6.9.2.1-1

$$P_r := \phi_c \cdot P_n$$

$$P_r = \begin{pmatrix} 464 \\ 653 \\ 641 \\ 782 \\ 1031 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Strength Limit State

# SERVICE/EXTREME LIMIT STATES:

**Service and Extreme Limit States** Axial Resistance

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.5.3

$$\phi := 1.0$$

**Factored** Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_r := \phi \cdot P_n$$

$$P_r = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Service/Extreme Limit States

## Geotechnical Resistance - by Canadian Geotechnical Method

Assume abutment piles will be end bearing on bedrock driven through overlying sand and silt.

Bedrock Type:

Granite RQD 90%

Use RQD = 90% and $\phi$ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

**Axial Geotechnical Resistance of H-piles**

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

Look at these piles:

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Note: All matrices set up in this order

Steel area:

$$A_s = \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$$

Pile depth:

$$d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$$

Pile width:

$$b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

End bearing resistance of piles in bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

$q_u$ for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 20000 \cdot psi$

Determine $K_{sp}$: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 48 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b:

$$b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.6667 \\ 0.6614 \\ 0.6005 \\ 0.5981 \\ 0.5941 \end{pmatrix}$$

$K_{sp}$ includes a factor of safety of 3

Length of rock socket, $L_s$: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, $B_s$: $B_s := 1 \cdot ft$

depth factor, $d_f$: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 1920 \\ 1905 \\ 1729 \\ 1723 \\ 1711 \end{pmatrix} \cdot ksf$$

**Nominal** Geotechnical Tip Resistance, $R_p$:

Multiply by 3 to take out FS=3 on $K_{sp}$

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

# STRENGTH LIMIT STATE:

**Factored** Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing in rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, $\phi_{stat}$ $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 279 \\ 389 \\ 347 \\ 421 \\ 552 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Strength Limit State

# SERVICE/EXTREME LIMIT STATES:

**Factored** Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States $\phi$ = 1.0 LRFD 10.5.5.1 and 10.5.5.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Service/Extreme
Limit States

## Axial Geotechnical Resistance Piles Driven to Hard Rock per LRFD Article 10.7.3.2.3

LRFD Article 10.7.3.2.3 states: "The nominal resistance of piles driven to point bearing on hard rock where pile penetratic into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exce the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions."

Nominal Structural Resistance:
previously calculated

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot \text{kip}$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Determine *Factored* Axial Geotechnical Resistance at the **Strength Limit State**

Apply resistance factor for severe driving from LRFD Article 6.5.4.2

$$\phi_{csevere} := 0.5$$

Factored Axial Geotechnical Resistance
**Strength Limit State**

$$P_{strength} := \phi_{csevere} \cdot P_n$$

$$P_{strength} = \begin{pmatrix} 387 \\ 544 \\ 534 \\ 652 \\ 859 \end{pmatrix} \cdot \text{kip}$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

Determine *Factored* Axial Geotechnical Resistance at the **Service and Extreme Limit States**:

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.5.3

$$\phi = 1.0$$

Factored Axial Geotechnical Resistance -
**Service and Extreme Limit States**

$$P_{serv\_ext} := \phi \cdot P_n$$

$$P_{serv\_ext} = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot \text{kip}$$

**HP 12 x 53**
**HP 12 x 74**
**HP 14 x 73**
**HP 14 x 89**
**HP 14 x 117**

## DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1 Pile Drivability Analysis, Steel piles and 6.5.4.2 resistance during pile driving

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile can not exceed 45 ksi

**Compute Resistance that can be achieved in a drivability analysis:**

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-45 gives resistance factor for dynamic test, $\phi_{dyn}$:

$\phi_{dyn} := 0.65$

# Pile Size = 12 x 53

**Assume Contractor will use a Delmag 36-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation — 19-Dec-2012
19291 Lexington 12x53 Drivability — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 525.0 | 44.85 | 7.48 | 11.7 | 6.46 | 38.49 |
| 526.0 | 44.92 | 7.49 | 11.7 | 6.47 | 38.57 |
| 527.0 | 44.96 | 7.49 | 11.8 | 6.47 | 38.59 |
| 528.0 | 44.98 | 7.49 | 12.0 | 6.47 | 38.61 |
| 529.0 | 45.01 | 7.50 | 12.1 | 6.48 | 38.62 |
| 530.0 | 45.05 | 7.50 | 12.2 | 6.48 | 38.64 |
| 531.0 | 45.07 | 7.50 | 12.4 | 6.49 | 38.65 |
| 532.0 | 45.11 | 7.51 | 12.5 | 6.49 | 38.67 |
| 533.0 | 45.16 | 7.51 | 12.6 | 6.49 | 38.69 |
| 534.0 | 45.17 | 7.51 | 12.8 | 6.50 | 38.71 |

Limit driving stress to ~45 ksi - blow count limited to 12 bpi as >12 bpi exceeds 45 ksi

$R_{dr\_12x53} := 528 \cdot kip$

Strength Limit State:

$R_{dr\_12x53\_strength} := R_{dr\_12x53} \cdot \phi_{dyn}$

$R_{dr\_12x53\_strength} = 343 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_12x53\_servext} := R_{dr\_12x53} \cdot \phi$

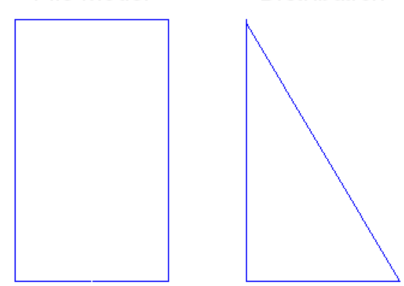
$R_{dr\_12x53\_servext} = 528 \cdot kip$

DELMAG D 36-32

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 3.20 | kips |
| Hammer Cushion | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.040 | in |
| Skin Damping | 0.050 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 130.00 | ft |
| Pile Penetration | 128.00 | ft |
| Pile Top Area | 15.50 | in2 |

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

# Pile Size = 12 x 74

**Assume Contractor will use a Delmag 36-32 hammer on second fuel setting**

State of Maine Dept. Of Transportation 19-Dec-2012
19291 Lexington 12x74 Drivability GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 662.0 | 44.58 | 7.25 | 11.7 | 7.05 | 40.98 |
| 664.0 | 44.65 | 7.25 | 11.9 | 7.06 | 40.98 |
| 665.0 | 44.70 | 7.25 | 12.0 | 7.06 | 41.01 |
| 666.0 | 44.71 | 7.24 | 12.1 | 7.06 | 40.99 |
| 668.0 | 44.80 | 7.24 | 12.3 | 7.08 | 41.05 |
| 670.0 | 44.84 | 7.26 | 12.3 | 7.08 | 41.13 |
| 672.0 | 44.90 | 7.26 | 12.5 | 7.09 | 41.14 |
| 674.0 | 44.97 | 7.26 | 12.7 | 7.09 | 41.15 |
| 676.0 | 45.01 | 7.26 | 12.9 | 7.10 | 41.15 |
| 678.0 | 45.05 | 7.26 | 13.1 | 7.10 | 41.17 |

Limit driving stress to ~45 ksi - blow count limited to 12 bpi as >12 bpi exceeds 45 ksi

$R_{dr\_12x74} := 665 \cdot kip$

Strength Limit State:

$R_{dr\_12x74\_strength} := R_{dr\_12x74} \cdot \phi_{dyn}$

$R_{dr\_12x74\_strength} = 432 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_12x74\_servext} := R_{dr\_12x74} \cdot \phi$

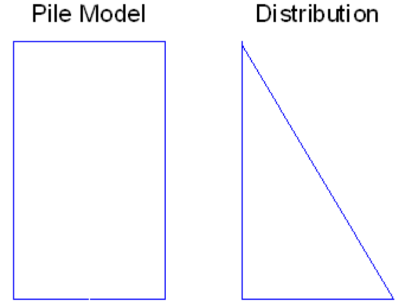
$R_{dr\_12x74\_servext} = 665 \cdot kip$

DELMAG D 36-32

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 3.20 | kips |
| Hammer Cushion | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.040 | in |
| Skin Damping | 0.050 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 130.00 | ft |
| Pile Penetration | 128.00 | ft |
| Pile Top Area | 21.80 | in2 |

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

## Pile Size = 14 x 73

**Assume Contractor will use a APE D36-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation — 19-Dec-2012
19291 Lexington 14x73 Drivability — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 620.0 | 43.24 | 6.26 | 14.6 | 6.64 | 34.99 |
| 622.0 | 43.32 | 6.27 | 14.8 | 6.65 | 35.05 |
| 624.0 | 43.39 | 6.28 | 15.0 | 6.66 | 35.08 |
| 626.0 | 43.43 | 6.30 | 15.3 | 6.66 | 35.09 |
| 628.0 | 43.51 | 6.30 | 15.5 | 6.67 | 35.11 |
| 630.0 | 43.54 | 6.32 | 15.8 | 6.67 | 35.13 |
| 632.0 | 43.60 | 6.32 | 16.1 | 6.68 | 35.14 |
| 634.0 | 43.67 | 6.33 | 16.4 | 6.68 | 35.16 |
| 636.0 | 43.76 | 6.35 | 16.6 | 6.69 | 35.24 |
| 638.0 | 43.80 | 6.36 | 16.9 | 6.69 | 35.26 |

Limit blow count to 15 bpi with driving stress < 45 ksi

$R_{dr\_14x73} := 624 \cdot kip$

Strength Limit State:

$R_{dr\_14x73\_strength} := R_{dr\_14x73} \cdot \phi_{dyn}$

$R_{dr\_14x73\_strength} = 406 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

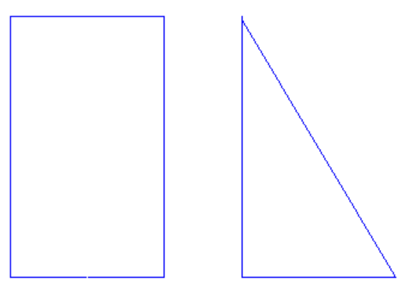
$R_{dr\_14x73\_servext} := R_{dr\_14x73} \cdot \phi$

$R_{dr\_14x73\_servext} = 624 \cdot kip$

APE D 36-32

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 2.56 | kips |
| Hammer Cushion | 52988 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.040 | in |
| Skin Damping | 0.050 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 130.00 | ft |
| Pile Penetration | 128.00 | ft |
| Pile Top Area | 21.40 | in2 |

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

## Pile Size = 14 x 89

**Assume Contractor will use a Delmag 46-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation — 19-Dec-2012
19291 Lexington 14x89 Drivability — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 710.0 | 44.62 | 5.32 | 7.6 | 6.97 | 46.27 |
| 715.0 | 44.76 | 5.37 | 7.7 | 6.98 | 46.35 |
| 720.0 | 44.92 | 5.42 | 7.8 | 6.99 | 46.47 |
| 721.0 | 44.65 | 5.43 | 8.1 | 6.93 | 46.01 |
| 722.0 | 44.66 | 5.43 | 8.1 | 6.93 | 45.98 |
| 723.0 | 44.72 | 5.45 | 8.1 | 6.93 | 46.05 |
| 725.0 | 44.76 | 5.47 | 8.2 | 6.94 | 46.07 |
| 730.0 | 44.90 | 5.50 | 8.4 | 6.95 | 46.09 |
| 735.0 | 45.03 | 5.54 | 8.6 | 6.96 | 46.19 |
| 740.0 | 45.16 | 5.58 | 8.8 | 6.97 | 46.21 |

Limit driving stress to 45 ksi - blow count limited to 8 bpi as >8 bpi exceeds 45 ksi

$R_{dr\_14x89} := 720 \cdot kip$

Strength Limit State:

$R_{dr\_14x89\_strength} := R_{dr\_14x89} \cdot \phi_{dyn}$

$R_{dr\_14x89\_strength} = 468 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

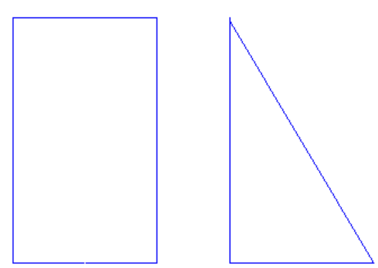
$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \phi$

$R_{dr\_14x89\_servext} = 720 \cdot kip$

DELMAG D 46-32

| | | |
|---|---|---|
| Efficiency | 0.800 | |
| Helmet | 3.20 | kips |
| Hammer Cushion | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.040 | in |
| Skin Damping | 0.050 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 130.00 | ft |
| Pile Penetration | 128.00 | ft |
| Pile Top Area | 26.10 | in2 |

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

## Pile Size = 14 x 117

**Assume Contractor will use a Delmag 46-32 hammer on lowest fuel setting**

State of Maine Dept. Of Transportation — 16-Aug-2012
19291 Lexington 14x117 Drivability — GRLWEAP (TM) Version 2003

| Ultimate Capacity kips | Maximum Compression Stress ksi | Maximum Tension Stress ksi | Blow Count blows/in | Stroke feet | Energy kips-ft |
|---|---|---|---|---|---|
| 970.0 | 43.39 | 4.50 | 14.9 | 7.02 | 42.81 |
| 971.0 | 43.41 | 4.50 | 15.1 | 7.02 | 42.76 |
| 972.0 | 43.45 | 4.52 | 15.0 | 7.02 | 42.85 |
| 973.0 | 43.45 | 4.51 | 15.2 | 7.02 | 42.80 |
| 974.0 | 43.46 | 4.52 | 15.2 | 7.02 | 42.82 |
| 975.0 | 43.48 | 4.53 | 15.2 | 7.02 | 42.85 |
| 976.0 | 43.50 | 4.54 | 15.3 | 7.02 | 42.87 |
| 977.0 | 43.51 | 4.53 | 15.4 | 7.02 | 42.81 |
| 978.0 | 43.53 | 4.54 | 15.4 | 7.02 | 42.82 |
| 979.0 | 43.54 | 4.54 | 15.5 | 7.02 | 42.86 |

Limit blow count to 15 bpi with driving stress < 45 ksi

$R_{dr\_14x117} := 972 \cdot kip$

Strength Limit State:

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \phi_{dyn}$

$R_{dr\_14x117\_strength} = 632 \cdot kip$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \phi$

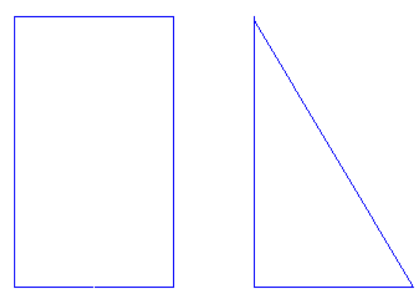
$R_{dr\_14x117\_servext} = 972 \cdot kip$

DELMAG D 46-32

| | | |
|---|---|---|
| Stroke | 7.02 | feet |
| Efficiency | 0.800 | |
| Helmet | 3.20 | kips |
| Hammer Cushion | 109975 | kips/in |
| Skin Quake | 0.100 | in |
| Toe Quake | 0.040 | in |
| Skin Damping | 0.050 | sec/ft |
| Toe Damping | 0.150 | sec/ft |
| Pile Length | 130.00 | ft |
| Pile Penetration | 128.00 | ft |
| Pile Top Area | 34.40 | in2 |

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

## Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

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

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

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

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

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

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

$K_p = 6.89$

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

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

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

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

$K_{p\_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for $K_p$ when $\beta$>0.

# Bearing Resistance - PCMG Wall:

**Part 1 - Service Limit State**

**Nominal and factored Bearing Resistance - spread footing on sand fill**

**Presumptive Bearing Resistance for Service Limit State ONLY**

Reference: AASHTO LRFD Bridge Design Specifications 6th Edition
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Sand fill

Consistency In Place: medium dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use: 6 ksf

$$tsf := g \cdot \left(\frac{ton}{ft^2}\right)$$

Recommended Value: $6 \cdot ksf = 3 \cdot tsf$

Therefore: $q_{nom} := 3 \cdot tsf$

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

$q_{factored\_bc} := 3 \cdot tsf$ or $q_{factored\_bc} = 6 \cdot ksf$

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

**Part 2 - Strength Limit State**

**Nominal and factored Bearing Resistance - PCMG Wall on sand fill**

**Reference: Foundation Engineering and Design by JE Bowles Fifth Edition**

Assumptions:
1. PCMG Wall will be on sand fill $D_f := 6 \cdot ft$
2. Assumed parameters for foundation soils: (Ref: Bowles 5th Ed Table 3-4)
   - Saturated unit weight: $\gamma_s := 125 \cdot pcf$
   - Dry unit weight: $\gamma_d := 120 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 32 \cdot deg$
   - Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on $\phi$-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Look at several stem lengths

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

Terzaghi Shape factors from Table 4-1

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

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

For $\phi$=32 deg $N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

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

$q := D_f \cdot (\gamma_s)$ $q = 0.375 \cdot tsf$

$q_{nominal} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s)B \cdot N_\gamma \cdot s_\gamma$

$$q_{nominal} = \begin{pmatrix} 12.8 \\ 14.2 \\ 15.6 \\ 17 \\ 18.3 \end{pmatrix} \cdot tsf$$

**At Strength Limit State**:

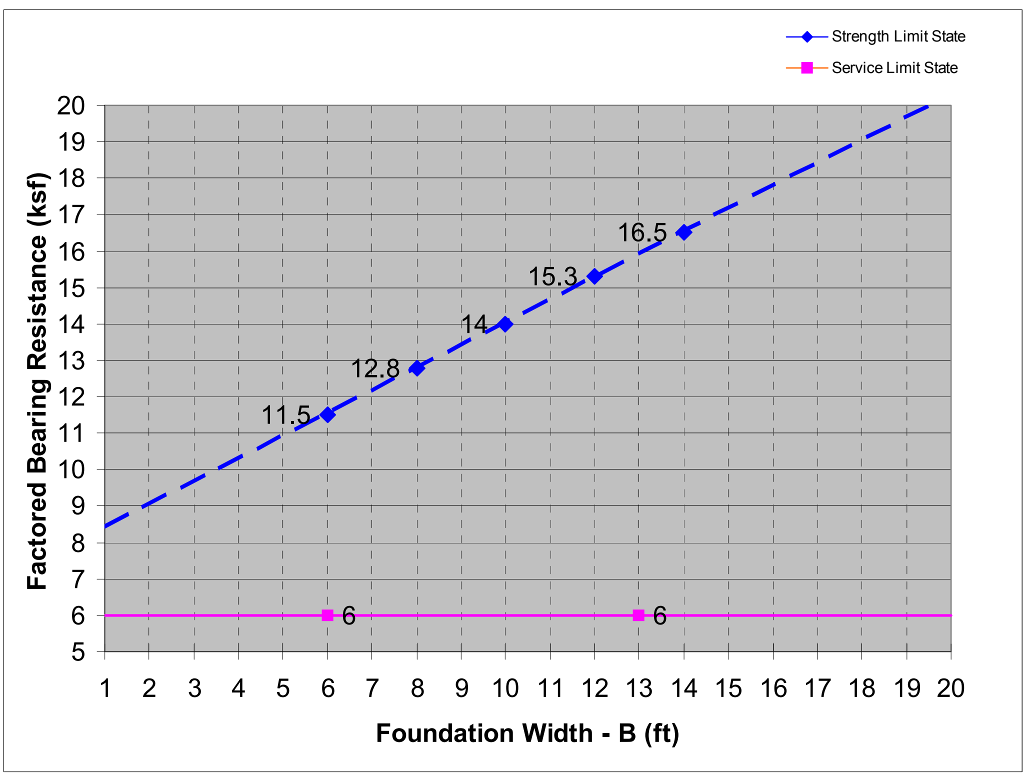
Resistance Factor: $\phi_b := 0.45$

AASHTO LRFD Table 10.5.5.2.2-1

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

Based on these footing widths

$$q_{factored} = \begin{pmatrix} 5.8 \\ 6.4 \\ 7 \\ 7.6 \\ 8.2 \end{pmatrix} \cdot tsf \qquad q_{factored} = \begin{pmatrix} 11.5 \\ 12.8 \\ 14 \\ 15.3 \\ 16.5 \end{pmatrix} \cdot ksf \qquad B = \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$



## Settlement Analysis:

Reference: FHWA Soils and Foundations Reference Manual - Volume 1 (FHWA NHI-06-088) Hough pg 7-16 and AASHTO LRFD Bridge Design Specifications 5th Edition 2010

The roadway grade at centerline may be raised by as much as 1.2 feet.
Look at a simplified soil profile based on BB-LSS-101

Finished Grade

Proposed Fill - Look at 1.2 feet of fill
N = 25 bpf (medium dense)
$\gamma$ = 125 pcf

Existing Grade

Existing Fill - fine to coarse sand $H_{1fill} := 14.0 \cdot ft$ $\gamma_{fill} := 125 \cdot pcf$ $N_{fill} := 8$

Groundwater at 14.0 ft bgs

$\gamma_w := 62.4pcf$

Sand Alluvium - fine to coarse sand $H_{2sand} := 6 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{2sand} := 9$

Marine Delta Deposits - sand, silt, silty sand
Total Layer height: H = 56.0 ft - divide into 6 layers

$H_3 := 56.0 \cdot ft$

$H_{3mdd1} := 8.0 \cdot ft$ $\gamma_{mdd} := 115 \cdot pcf$ $N_{3mdd1} := 5$

$H_{3mdd2} := 8.0 \cdot ft$ $N_{3mdd2} := 6$

$H_{3mdd3} := 10.0 \cdot ft$ $N_{3mdd3} := 9$

$H_{3mdd4} := 10.0 \cdot ft$ $N_{3mdd4} := 10$

$H_{3mdd5} := 10.0 \cdot ft$ $N_{3mdd5} := 14$

$H_{3mdd6} := 10.0 \cdot ft$ $N_{3mdd6} := 11$

Glaciomarine Deposits - clayey silt and silt
Total Layer height: H = 48.0 ft - divide into 5 layers

$H_4 := 48.0 \cdot ft$

$H_{4gmd1} := 8.0 \cdot ft$ $\gamma_{gmd} := 125 \cdot pcf$ $C_{c\_gmd1} := 0.4$ $C_{r\_gmd1} := 0.03$ $e_{ogmd1} := 0.77$

$H_{4gmd2} := 10.0 \cdot ft$ $C_{c\_gmd2} := 0.4$ $C_{r\_gmd2} := 0.03$ $e_{ogmd2} := 0.77$

$H_{4gmd3} := 10.0 \cdot ft$ $C_{c\_gmd3} := 0.4$ $C_{r\_gmd3} := 0.03$ $e_{ogmd3} := 0.77$

$H_{4gmd4} := 10.0 \cdot ft$ $C_{c\_gmd4} := 0.4$ $C_{r\_gmd4} := 0.03$ $e_{ogmd4} := 0.73$

$H_{4gmd5} := 10.0 \cdot ft$ $C_{c\_gmd5} := 0.4$ $C_{r\_gmd5} := 0.03$ $e_{ogmd5} := 0.73$

Assumed Values based on Lab Data and "A Summary of Geotechnical Engineering Information on the Presumpscot Formation Silty Clay" 1986 by David W. Andrews:

Sand - Till (?)
Total Layer height: H = 7.4 ft

$H_5 := 7.4 \cdot ft$

$H_{5sand} := 7.4 \cdot ft$ $\gamma_{sand} := 115 \cdot pcf$ $N_{5sand} := 28$

Bedrock - granite

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

| | | | |
|---|---|---|---|
| Project Name: | Lower Sandy Stream | Client: | Lexington |
| Project Number: | 19291.00 | Project Manager: | SBodge |
| Date: | 08/23/12 | Computed by: | km |

Embank. slope a = 10.00(ft)
Embank. width b = 30.00(ft)
p load/unit area = 150.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 20.00(ft)

| Z (ft) | Vert. Δz (psf) |
|---|---|
| 0.00 | 150.00 |
| 1.00 | 149.96 |
| 5.00 | 145.70 |
| 7.00 | 140.28 |
| 8.00 | 136.96 |
| 9.00 | 133.36 |
| 16.00 | 107.03 |
| 17.00 | 103.59 |
| 19.00 | 97.13 |
| 23.00 | 85.81 |
| 24.00 | 83.29 |
| 25.00 | 80.90 |
| 31.00 | 68.66 |
| 32.00 | 66.94 |
| 33.00 | 65.28 |
| 40.00 | 55.51 |
| 41.00 | 54.33 |
| 42.00 | 53.20 |
| 50.00 | 45.52 |
| 51.00 | 44.70 |
| 52.00 | 43.92 |
| 60.00 | 38.47 |
| 61.00 | 37.88 |
| 62.00 | 37.30 |
| 70.00 | 33.26 |
| 71.00 | 32.81 |
| 72.00 | 32.38 |
| 79.00 | 29.63 |
| 80.00 | 29.27 |
| 81.00 | 28.92 |
| 88.00 | 26.70 |
| 89.00 | 26.41 |
| 90.00 | 26.12 |
| 98.00 | 24.05 |
| 99.00 | 23.81 |
| 100.00 | 23.58 |
| 108.00 | 21.87 |
| 109.00 | 21.67 |
| 110.00 | 21.48 |
| 118.00 | 20.05 |
| 119.00 | 19.88 |
| 120.00 | 19.72 |
| 126.00 | 18.80 |
| 127.00 | 18.65 |
| 128.00 | 18.51 |
| 130.00 | 18.23 |

at 7.0 ft $\Delta\sigma_{z1fill} := 140.28 \cdot psf$

at 17.0 ft $\Delta\sigma_{z2sand} := 103.59 \cdot psf$

at 24.0ft $\Delta\sigma_{z3mdd1} := 83.29 \cdot psf$

at 32.0 ft $\Delta\sigma_{z3mdd2} := 66.94 \cdot psf$

at 41.0 ft $\Delta\sigma_{z3mdd3} := 54.33 \cdot psf$

at 51.0 ft $\Delta\sigma_{z3mdd4} := 44.7 \cdot psf$

at 61.0 ft $\Delta\sigma_{z3mdd5} := 37.88 \cdot psf$

at 71.0 ft $\Delta\sigma_{z3mdd6} := 32.81 \cdot psf$

at 80.0 ft $\Delta\sigma_{z4gmd1} := 29.27 \cdot psf$

at 89.0 ft $\Delta\sigma_{z4gmd2} := 26.41 \cdot psf$

at 99.0 ft $\Delta\sigma_{z4gmd3} := 23.81 \cdot psf$

at 109.0 ft $\Delta\sigma_{z4gmd4} := 21.67 \cdot psf$

at 119.0 ft $\Delta\sigma_{z4gmd5} := 19.88 \cdot psf$

at 127.7 ft $\Delta\sigma_{z5sand} := 18.55 \cdot psf$

**Existing Fill**

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

Calculate vertical stress at mid point: $\sigma_{1fill\_o} := \frac{H_{1fill}}{2} \cdot \gamma_{fill}$ $\quad \sigma_{1fill\_o} = 0.875 \cdot ksf$

Corrected SPT $N_{60}$-value (bpf) $\quad N_{fill} = 8$

$C_{N\_1fill} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{1fill\_o}}\right)$ $\quad C_{N\_1fill} = 1.2782$ $\quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_1fill} \cdot N_{fill}$ $\quad N1_{60} = 10$
From LRFD Eq 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $\quad C_{1fill} := 47$

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

$\Delta\sigma_{z1fill} = 140.28 \cdot psf$

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

Calculate vertical stress at mid point:

$\sigma_{2sand\_o} := \left[\frac{H_{2sand}}{2} \cdot (\gamma_{sand} - \gamma_w)\right] + H_{1fill} \cdot (\gamma_{fill})$ $\quad \sigma_{2sand\_o} = 1.9078 \cdot ksf$

Corrected SPT $N_{60}$-value (bpf) $\quad N_{2sand} = 9$

$C_{N\_2sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{2sand\_o}}\right)$ $\quad C_{N\_2sand} = 1.0176$ $\quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_2sand} \cdot N_{2sand}$ $\quad N1_{60} = 9$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $\quad C_{2sand} := 45$

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

$\Delta\sigma_{z2sand} = 103.59 \cdot psf$

**Marine Delta Deposits - 6 layers**

Layer 1: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd1\_o} := \left[\frac{H_{3mdd1}}{2}\cdot(\gamma_{mdd}-\gamma_{w})\right] + H_{2sand}\cdot(\gamma_{sand}-\gamma_{w}) + H_{1fill}\cdot(\gamma_{fill}) \qquad \sigma_{3mdd1\_o} = 2.276\cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd1} = 5$

$$C_{N\_3mdd1} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{3mdd1\_o}}\right) \qquad C_{N\_3mdd1} = 0.9586$$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd1}\cdot N_{3mdd1}$ $N1_{60} = 5$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd1} := 42$

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

$$\Delta\sigma_{z3mdd1} = 83.29\cdot psf$$

Layer 2: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd2\_o} := \left[\frac{H_{3mdd2}}{2}\cdot(\gamma_{mdd}-\gamma_{w})\right] + H_{3mdd1}\cdot(\gamma_{mdd}-\gamma_{w}) + H_{2sand}\cdot(\gamma_{sand}-\gamma_{w}) + H_{1fill}\cdot(\gamma_{fill})$$

$$\sigma_{3mdd2\_o} = 2.6968\cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd2} = 6$

$$C_{N\_3mdd2} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{3mdd2\_o}}\right) \qquad C_{N\_3mdd2} = 0.9018$$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd2}\cdot N_{3mdd2}$ $N1_{60} = 5$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd2} := 42$

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

$$\Delta\sigma_{z3mdd2} = 66.94\cdot psf$$

Layer 3: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd3\_o} := \left[\frac{H_{3mdd3}}{2} \cdot (\gamma_{mdd} - \gamma_w)\right] + (H_{3mdd1} + H_{3mdd2}) \cdot (\gamma_{mdd} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + H_{1fill} \cdot (\gamma_{fill})$$

$$\sigma_{3mdd3\_o} = 3.1702 \cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd3} = 9$

$$C_{N\_3mdd3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{3mdd3\_o}}\right) \qquad C_{N\_3mdd3} = 0.8477$$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd3} \cdot N_{3mdd3}$ $N1_{60} = 8$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd3} := 38$

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

$$\Delta\sigma_{z3mdd3} = 54.33 \cdot psf$$

Layer 4: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd4\_o} := \left[\frac{H_{3mdd4}}{2} \cdot (\gamma_{mdd} - \gamma_w)\right] + (H_{3mdd1} + H_{3mdd2} + H_{3mdd3}) \cdot (\gamma_{mdd} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + H_{1fill} \cdot (\gamma_{fill})$$

$$\sigma_{3mdd4\_o} = 3.6962 \cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd4} = 10$

$$C_{N\_3mdd4} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{3mdd4\_o}}\right) \qquad C_{N\_3mdd4} = 0.7964$$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd4} \cdot N_{3mdd4}$ $N1_{60} = 8$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd4} := 38$

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

$$\Delta\sigma_{z3mdd4} = 44.7 \cdot psf$$

Layer 5: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd5\_o} := \left[\frac{H_{3mdd5}}{2} \cdot (\gamma_{mdd} - \gamma_w)\right] + (36 \cdot ft) \cdot (\gamma_{mdd} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + H_{1fill} \cdot (\gamma_{fill})$$

$$\sigma_{3mdd5\_o} = 4.2222 \cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd5} = 14$

$C_{N\_3mdd5} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{3mdd5\_o}}\right)$ $C_{N\_3mdd5} = 0.7519$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd5} \cdot N_{3mdd5}$ $N1_{60} = 11$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd5} := 43$

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

$$\Delta\sigma_{z3mdd5} = 37.88 \cdot psf$$

Layer 6: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3mdd6\_o} := \left[\frac{H_{3mdd6}}{2} \cdot (\gamma_{mdd} - \gamma_w)\right] + (46 \cdot ft) \cdot (\gamma_{mdd} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + H_{1fill} \cdot (\gamma_{fill})$$

$$\sigma_{3mdd6\_o} = 4.7482 \cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $N_{3mdd6} = 11$

$C_{N\_3mdd6} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{3mdd6\_o}}\right)$ $C_{N\_3mdd6} = 0.7127$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3mdd6} \cdot N_{3mdd6}$ $N1_{60} = 8$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{3mdd6} := 38$

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

$$\Delta\sigma_{z3mdd6} = 32.81 \cdot psf$$

Calculate vertical stress at bottom of Marine Delta Deposits layer (76.0 ft bgs):

$$\sigma_{76ft} := H_3 \cdot (\gamma_{mdd} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + H_{1fill} \cdot \gamma_{fill}$$

$$\sigma_{76ft} = 2.51 \cdot tsf$$

**Glaciomarine Deposits - 5 layers**

Assumed Values based on Lab Data and "A Summary of Geotechnical Engineering Information on the Presumpscot Formation Silty Clay" 1986 by David W. Andrews:

Layer 1: Assumed Values: $e_{ogmd1} = 0.77$ $C_{r\_gmd1} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{4gmd1\_o} := \left[\frac{H_{4gmd1}}{2} \cdot \left(\gamma_{gmd} - \gamma_w\right)\right] + \sigma_{76ft} \qquad \sigma_{4gmd1\_o} = 5.2616 \cdot ksf$$

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

$$\Delta\sigma_{z4gmd1} = 29.27 \cdot psf$$

Layer 2: Assumed Values: $e_{ogmd2} = 0.77$ $C_{r\_gmd2} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{4gmd2\_o} := \left[\frac{H_{4gmd2}}{2} \cdot \left(\gamma_{gmd} - \gamma_w\right)\right] + H_{4gmd1} \cdot \left(\gamma_{gmd} - \gamma_w\right) + \sigma_{76ft} \qquad \sigma_{4gmd2\_o} = 5.825 \cdot ksf$$

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

$$\Delta\sigma_{z4gmd2} = 26.41 \cdot psf$$

Layer 3: Assumed Values: $e_{ogmd3} = 0.77$ $C_{r\_gmd3} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{4gmd3\_o} := \left[\frac{H_{4gmd3}}{2} \cdot \left(\gamma_{gmd} - \gamma_w\right)\right] + \left(H_{4gmd1} + H_{4gmd2}\right) \cdot \left(\gamma_{gmd} - \gamma_w\right) + \sigma_{76ft} \qquad \sigma_{4gmd3\_o} = 6.451 \cdot ksf$$

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

$$\Delta\sigma_{z4gmd3} = 23.81 \cdot psf$$

Layer 4: Assumed Values: $e_{ogmd4} = 0.73$ $C_{r\_gmd4} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{4gmd4\_o} := \left[\frac{H_{4gmd4}}{2} \cdot \left(\gamma_{gmd} - \gamma_w\right)\right] + \left(H_{4gmd1} + H_{4gmd2} + H_{4gmd3}\right) \cdot \left(\gamma_{gmd} - \gamma_w\right) + \sigma_{76ft} \qquad \sigma_{4gmd4\_o} = 7.077 \cdot ksf$$

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

$$\Delta\sigma_{z4gmd4} = 21.67 \cdot psf$$

Layer 5: Assumed Values: $e_{ogmd5} = 0.73$ $C_{r\_gmd5} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{4gmd5\_o} := \left[\frac{H_{4gmd4}}{2} \cdot \left(\gamma_{gmd} - \gamma_w\right)\right] + \left(H_{4gmd1} + H_{4gmd2} + H_{4gmd3} + H_{4gmd4}\right) \cdot \left(\gamma_{gmd} - \gamma_w\right) + \sigma_{76ft} \qquad \sigma_{4gmd5\_o} = 7.703 \cdot ksf$$

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

$$\Delta\sigma_{z4gmd5} = 19.88 \cdot psf$$

**Sand/Till**

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

Calculate vertical stress at mid point:

$$\sigma_{5sand\_o} := \frac{H_5}{2} \cdot (\gamma_{sand} - \gamma_w) + H_4 \cdot (\gamma_{gmd} - \gamma_w) + \sigma_{76ft} \qquad \sigma_{5sand\_o} = 8.2106 \cdot ksf$$

Corrected SPT $N_{60}$-value (bpf) $\quad N_{5sand} = 28$

$$C_{N\_5sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{5sand\_o}}\right) \qquad C_{N\_5sand} = 0.5295$$

LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $\quad N1_{60} := C_{N\_5sand} \cdot N_{5sand} \qquad N1_{60} = 15$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded silty sand and gravel" curve

Bearing Capacity Index: $\quad C_{5sand} := 65$

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

$$\Delta\sigma_{z5sand} = 18.55 \cdot psf$$

**Calculate Settlement:**

Existing Fill: $\quad \Delta H_{1fill} := H_{1fill} \cdot \frac{1}{C_{1fill}} \cdot \log\left(\frac{\sigma_{1fill\_o} + \Delta\sigma_{z1fill}}{\sigma_{1fill\_o}}\right) \qquad \Delta H_{1fill} = 0.2308 \cdot in$

Native Sand: $\quad \Delta H_{2sand} := H_{2sand} \cdot \frac{1}{C_{2sand}} \cdot \log\left(\frac{\sigma_{2sand\_o} + \Delta\sigma_{z2sand}}{\sigma_{2sand\_o}}\right) \qquad \Delta H_{2sand} = 0.0367 \cdot in$

Marine Delta Layer 1: $\quad \Delta H_{3mdd1} := H_{3mdd1} \cdot \frac{1}{C_{3mdd1}} \cdot \log\left(\frac{\sigma_{3mdd1\_o} + \Delta\sigma_{z3mdd1}}{\sigma_{3mdd1\_o}}\right) \qquad \Delta H_{3mdd1} = 0.0357 \cdot in$

Marine Delta Layer 2: $\quad \Delta H_{3mdd2} := H_{3mdd2} \cdot \frac{1}{C_{3mdd2}} \cdot \log\left(\frac{\sigma_{3mdd2\_o} + \Delta\sigma_{z3mdd2}}{\sigma_{3mdd2\_o}}\right) \qquad \Delta H_{3mdd2} = 0.0243 \cdot in$

Marine Delta Layer 3: $\quad \Delta H_{3mdd3} := H_{3mdd3} \cdot \frac{1}{C_{3mdd3}} \cdot \log\left(\frac{\sigma_{3mdd3\_o} + \Delta\sigma_{z3mdd3}}{\sigma_{3mdd3\_o}}\right) \qquad \Delta H_{3mdd3} = 0.0233 \cdot in$

Marine Delta Layer 4: $\quad \Delta H_{3mdd4} := H_{3mdd4} \cdot \frac{1}{C_{3mdd4}} \cdot \log\left(\frac{\sigma_{3mdd4\_o} + \Delta\sigma_{z3mdd4}}{\sigma_{3mdd4\_o}}\right) \qquad \Delta H_{3mdd4} = 0.0165 \cdot in$

Marine Delta Layer 5: $\quad \Delta H_{3mdd5} := H_{3mdd5} \cdot \frac{1}{C_{3mdd5}} \cdot \log\left(\frac{\sigma_{3mdd5\_o} + \Delta\sigma_{z3mdd5}}{\sigma_{3mdd5\_o}}\right) \qquad \Delta H_{3mdd5} = 0.0108 \cdot in$

Marine Delta Layer 6: $\quad \Delta H_{3mdd6} := H_{3mdd6} \cdot \frac{1}{C_{3mdd6}} \cdot \log\left(\frac{\sigma_{3mdd6\_o} + \Delta\sigma_{z3mdd6}}{\sigma_{3mdd6\_o}}\right) \qquad \Delta H_{3mdd6} = 0.0094 \cdot in$

$$\Delta H_{3mdd} := \Delta H_{3mdd1} + \Delta H_{3mdd2} + \Delta H_{3mdd3} + \Delta H_{3mdd4} + \Delta H_{3mdd5} + \Delta H_{3mdd6} \qquad \Delta H_{3mdd} = 0.1201 \cdot in$$

Glaciomarine Layer 1: $\Delta H_{4gmd1} := H_{4gmd1} \cdot \left(\frac{C_{r\_gmd1}}{1 + e_{ogmd1}}\right) \cdot \log\left(\frac{\sigma_{4gmd1\_o} + \Delta\sigma_{z4gmd1}}{\sigma_{4gmd1\_o}}\right)$ $\Delta H_{4gmd1} = 0.0039 \cdot in$

Glaciomarine Layer 2: $\Delta H_{4gmd2} := H_{4gmd2} \cdot \left(\frac{C_{r\_gmd2}}{1 + e_{ogmd2}}\right) \cdot \log\left(\frac{\sigma_{4gmd2\_o} + \Delta\sigma_{z4gmd2}}{\sigma_{4gmd2\_o}}\right)$ $\Delta H_{4gmd2} = 0.004 \cdot in$

Glaciomarine Layer 3: $\Delta H_{4gmd3} := H_{4gmd3} \cdot \left(\frac{C_{r\_gmd3}}{1 + e_{ogmd3}}\right) \cdot \log\left(\frac{\sigma_{4gmd3\_o} + \Delta\sigma_{z4gmd3}}{\sigma_{4gmd3\_o}}\right)$ $\Delta H_{4gmd3} = 0.0033 \cdot in$

Glaciomarine Layer 4: $\Delta H_{4gmd4} := H_{4gmd4} \cdot \left(\frac{C_{r\_gmd4}}{1 + e_{ogmd4}}\right) \cdot \log\left(\frac{\sigma_{4gmd4\_o} + \Delta\sigma_{z4gmd4}}{\sigma_{4gmd4\_o}}\right)$ $\Delta H_{4gmd4} = 0.0028 \cdot in$

Glaciomarine Layer 5: $\Delta H_{4gmd5} := H_{4gmd5} \cdot \left(\frac{C_{r\_gmd5}}{1 + e_{ogmd5}}\right) \cdot \log\left(\frac{\sigma_{4gmd5\_o} + \Delta\sigma_{z4gmd5}}{\sigma_{4gmd5\_o}}\right)$ $\Delta H_{4gmd5} = 0.0023 \cdot in$

$$\Delta H_{4gmd} := \Delta H_{4gmd1} + \Delta H_{4gmd2} + \Delta H_{4gmd3} + \Delta H_{4gmd4} + \Delta H_{4gmd5} \qquad \Delta H_{4gmd} = 0.0163 \cdot in$$

Sand/Till: $\Delta H_{5sand} := H_5 \cdot \frac{1}{C_{5sand}} \cdot \log\left(\frac{\sigma_{5sand\_o} + \Delta\sigma_{z5sand}}{\sigma_{5sand\_o}}\right)$ $\Delta H_{5sand} = 0.0013 \cdot in$

**TOTAL SETTLEMENT:**

$$\Delta H_T := \Delta H_{1fill} + \Delta H_{2sand} + \Delta H_{3mdd} + \Delta H_{4gmd} + \Delta H_{5sand} \qquad \Delta H_T = 0.4053 \cdot in$$

Say approximately 1 inch of settlement will occur during construction

**LK Check with FOSSA Software Indicates ~ 1 inch of settlement immediate (elastic) and consolidation ρ. LM 12/03/2012**

# Frost Protection:

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

From the Design Freezing Index Map:
Lexington TWP Maine
DFI = 2000 degree-days

From the lab testing: soils are coarse grained assume a water content = ~30%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 2000 frost penetration = 67.5 inches

$\text{Frost\_depth} := 67.5\text{in}$ $\quad$ $\text{Frost\_depth} = 5.625 \cdot \text{ft}$

*Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.*

**Method 2 - Check Frost Depth using Modberg Software**

Closest Station is Farmington

ModBerg Results
Project Location: Farmington, Maine

| | | |
|---|---|---|
| Air Design Freezing Index | = | 2023 F-days |
| N-Factor | = | 0.80 |
| Surface Design Freezing Index | = | 1618 F-days |
| Mean Annual Temperature | = | 41.2 deg F |
| Design Length of Freezing Season | = | 145 days |

| Layer #:Type | t | w% | d | Cf | Cu | Kf | Ku | L |
|---|---|---|---|---|---|---|---|---|
| 1-Coarse | 90.0 | 30.0 | 120.0 | 38 | 56 | 4.7 | 1.9 | 5,184 |

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 = 7.50 ft = 90.0 in.

$\text{Frost\_depth}_{\text{modberg}} := 67.5 \cdot \text{in}$ $\quad$ $\text{Frost\_depth}_{\text{modberg}} = 5.625\,\text{ft}$

Use Frost Depth = 6.0 feet for design

## Seismic:

Seismic Site Classification
Ref: LRFD Table C3.10.3.1-1
Method B: Average N for the top 100 feet of soil

| BB-LSS-101 | | | | | BB-LSS-102 | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | |
| **Depth** | **SPT N** | | **di** | **di/N** | **Depth** | **SPT N** | | **di** | **di/N** |
| 2 | 14 | | 3 | 0.214286 | 6 | 7 | | 7 | 1 |
| 6 | 8 | | 5 | 0.625 | 11 | 1 | | 5 | 5 |
| 11 | 4 | | 5 | 1.25 | 16 | 8 | | 5 | 0.625 |
| 16 | 10 | | 5 | 0.5 | 21 | 11 | | 5 | 0.454545 |
| 21 | 4 | | 5 | 1.25 | 26 | 7 | | 5 | 0.714286 |
| 26 | 6 | | 5 | 0.833333 | 31 | 7 | | 5 | 0.714286 |
| 31 | 6 | | 5 | 0.833333 | 36 | 7 | | 5 | 0.714286 |
| 36 | 7 | | 5 | 0.714286 | 41 | 8 | | 5 | 0.625 |
| 41 | 11 | | 5 | 0.454545 | 46 | 6 | | 5 | 0.833333 |
| 46 | 7 | | 5 | 0.714286 | 51 | 6 | | 5 | 0.833333 |
| 51 | 11 | | 5 | 0.454545 | 56 | 13 | | 5 | 0.384615 |
| 56 | 11 | | 5 | 0.454545 | 61 | 13 | | 5 | 0.384615 |
| 61 | 15 | | 5 | 0.333333 | 66 | 7 | | 5 | 0.714286 |
| 66 | 11 | | 5 | 0.454545 | 71 | 18 | | 5 | 0.277778 |
| 71 | 11 | | 5 | 0.454545 | 76 | 4 | | 5 | 1.25 |
| 76 | 3 | | 5 | 1.666667 | 81 | 1 | WOR | 12 | 12 |
| 81 | 1 | WOR | 5 | 5 | 91 | 1 | WOR | 11 | 11 |
| 86 | 1 | WOR | 5 | 5 | | | | | |
| 91 | 1 | WOR | 5 | 5 | | | | | |
| 96 | 1 | WOR | 7 | 7 | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |
| **SUM** | | | **100** | **33.20725** | | | | **100** | **37.52536** |
| | | | | | | | | | |

**di/di/N** **3.01139**

**di/di/N** **2.664864**

| SUM | Nav. | 2.838127 |
|---|---|---|

Nav <15 bpf; Site Class E

Note: Weight of rod (WOR) and weight of hammer (WOH) values are taken as N=1.

19291.00 Lexington Township Lower Sandy Stream Bridge

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04961
Zip Code Latitude = 45.012500
Zip Code Longitude = -070.084500
Site Class B
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.073 | PGA - Site Class B |
| 0.2 | 0.160 | Ss - Site Class B |
| 1.0 | 0.049 | S1 - Site Class B |

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04961
Zip Code Latitude = 45.012500
Zip Code Longitude = -070.084500
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

| Period (sec) | Sa (g) | |
|---|---|---|
| 0.0 | 0.182 | As - Site Class E |
| 0.2 | 0.399 | SDs - Site Class E |
| 1.0 | 0.171 | SD1 - Site Class E |

# **Appendix D**

Special Provisions

# SPECIAL PROVISION
# SECTION 635
# PREFABRICATED CONCRETE MODULAR GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are**:** all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessity to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

| | |
|---|---|
| Gravel Borrow | 703.20 |
| Preformed Expansion Joint Material | 705.01 |
| Reinforcing Steel | 709.01 |
| Structural Pre-cast Concrete Units | 712.061 |
| Drainage Geotextile | 722.02 |

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) ±3/16 inch.
2. Squareness. The length differences between the two diagonals shall not exceed 5/16 inch.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 inch in 4 feet will be rejected. For textured surfaces, any surface defects in excess of 5/16 inch in 5 feet shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 inches wide, by 0.5 inch preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 inches wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 inches, minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 inch in 10 feet. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: backfill and bedding material shall only contain particles that will pass the 3-inch square mesh sieve and the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 inches in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a licensed Professional Engineer registered in accordance with the laws of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength, Service and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. 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 by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:
   1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.
   2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$
      Where: $R_R$ = Factored Sliding Resistance
      $\gamma_{p(max)}$ = Maximum Load Factor
      EH = Horizontal Earth Pressure
      ES = Earth Surcharge (as applicable)
   3. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure
      Where: $q_R$ = Factored Bearing Resistance, as shown on the plans
      Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
   4. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than or equal to 1.5.

   Live load surcharge on Prefabricated Concrete Modular Gravity walls shall be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($h_{eq}$) taken from LRFD Table 3.11.6.4-2 with consideration for the distance from the wall pressure surface to the edge of traffic. Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Section 11, where Article 11.10.10.2 is modified such that the upper 3.5 feet of concrete modular units shall be designed for an additional horizontal load of $\gamma P_{H1}$, where $\gamma P_{H1}$=300 lbs per linear foot of wall.

B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For overturning, the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikus Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 feet. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.

D. Safety Against Structural Failure. Prefabricated units shall be designed for all strength and reinforcement requirements in accordance with LRFD Section 5 and LRFD Article 11.11.5.

E. 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.

F. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.

G. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.

H. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

I. Design Life. Design life shall be in accordance with AASHTO requirements or 75 years; the more stringent requirements apply.

J. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

C. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.

E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.

F. Four weeks prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.05 Construction Requirements

Excavation. The excavation and use as fill or disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity Wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any pre-cast units that are damaged due to handling will be replaced at the Contractor's expense.

Installation of Wall Units. A field representative from the 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 Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 inch. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 inch per 10 feet of wall height. The prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content 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 uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square foot of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier 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 Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

| Pay Item | Pay Unit |
|---|---|
| 635.14 Prefabricated Concrete Modular Gravity Wall | Square Foot |