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MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Construction of:

SPLIT BROOK BRIDGE
ANDOVER ROAD OVER SPLIT BROOK
RUMFORD, MAINE

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Soils Report No. 2014-27
Bridge No. 6487

December 9, 2014

Table of Contents

GEOTECHNICAL DESIGN SUMMARY **1**

1.0 INTRODUCTION **4**

2.0 GEOLOGIC SETTING **4**

3.0 SUBSURFACE INVESTIGATION **5**

4.0 LABORATORY TESTING **5**

5.0 SUBSURFACE CONDITIONS **6**

5.1 FILL SOILS 6
5.2 SAND LAYER 6
5.3 GROUNDWATER 6

6.0 FOUNDATION ALTERNATIVES **6**

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS **7**

7.1 PRECAST CONCRETE BOX CULVERT DESIGN AND CONSTRUCTION 7
7.2 PRECAST CONCRETE BOX CULVERT HEADWALLS 8
7.3 PRECAST CONCRETE OUTLET WALLS AND OUTLET AND INLET TOE WALLS 8
7.4 BEARING RESISTANCE 9
7.5 PRECAST CONCRETE MODULAR GRAVITY (PCMG) WALLS 9
7.6 SETTLEMENT 10
7.7 FROST PROTECTION 11
7.8 SCOUR AND RIPRAP 11
7.9 SEISMIC DESIGN CONSIDERATIONS 12
7.10 CONSTRUCTION CONSIDERATIONS 12

8.0 CLOSURE **13**

Sheets

Sheet 1 - Location Map
Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile
Sheet 3 - Boring Log

Appendices

Appendix A – Boring Log
Appendix B – Calculations
Appendix C – Special Provision

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of a steel plate pipe arch which carries Andover Road over Split Brook in Rumford, Maine with the new Split Brook Bridge. The existing 8-foot 11-inch by 6-foot steel pipe arch culvert is approximately 60 feet long and was built in 1949. The proposed replacement structure will be a 18-foot span by 6-foot rise, approximately 56-foot long, precast concrete box culvert on a 5 degree skew. The following design recommendations are discussed in detail in Section 7.0 of this report.

Precast Concrete Box Culvert Design and Construction - The precast concrete box culvert shall be supplier-designed in accordance with Standard Specification 534 - Precast Structural Concrete, and AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD). The loading specified for the structure should be Modified HL-93 Strength 1 in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations.

The proposed box culvert will have 1-foot tall precast headwalls. The box will be embedded approximately 2 feet into the streambed and 1-foot of special fill will be placed inside the bottom of the culvert to create a natural streambed. The proposed box culvert will be bedded on a 2-foot thick layer of ¾- inch crushed stone reinforced with geogrid and wrapped in geotextile fabric.

Precast Concrete Outlet Walls and Toe Walls – The box culvert's outlet walls will be slope-tapered to match the 2H:1V sideslopes. The bottom slab connecting the left and right outlet walls and the bottom slab of the upstream end of the box culvert shall include toe walls to prevent undermining. The toe walls should extend a minimum of 1 foot below the maximum depth of scour.

The sloped outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit state load combinations specified in LRFD. The walls shall be designed to resist lateral earth pressures, vehicular loads, creep and temperature and shrinkage deformations of the concrete box culvert. The walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil.

Bearing Resistance - For a precast concrete box culvert with a base width of 20 feet, the applied bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 7.5 kips per square foot (ksf). A factored bearing resistance of 5 ksf should be used to control settlement when analyzing the service limit state. In no instance shall be bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$.

Prefabricated Concrete Modular Gravity (PCMG) Walls - The upstream end of the precast concrete box culvert will have inlet walls consisting of PCMG walls founded on native soils. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for PCMG walls shall be investigated at the strength limit state using the factored bearing resistances provided in Figure 7-1 of this report. A factored bearing resistance of 5 ksf may be used to control settlement when analyzing the service limit state. The PCMG walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2 feet, per LRFD Article 3.11.6.4.

For the lowest PCMG unit on soil the location of the resultant of the reaction forces at the strength limit state should be within the middle one-half (1/2) of the footing width. A sliding resistance factor, φ_τ , of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on soil. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils.

Settlement – The native sands encountered at the site are loose to medium dense in consistency. Elastic settlements due to the proposed precast concrete box culvert were calculated and are estimated to be approximately 1.5 inches. This settlement will occur relatively quickly. All loose or soft soils encountered at the foundation elevation for the precast box culvert will be excavated in its entirety to allow for construction of the 2-foot thick geotextile-wrapped crushed stone mat. Furthermore, in order to minimize any long term distress to the proposed structure, contract documents shall specify that all silt or other soft soils within the box culvert footprint and PCMG wall footprint be removed to full depth from the excavation and replaces with crushed stone when encountered. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

Frost Protection – Foundations placed on soil should be founded a minimum of 6.4 feet below finished exterior grade for frost protection.

Scour and Riprap – The box culvert shall be constructed with integral concrete headwalls, slope-tapered outlet walls and PCMG inlet walls to retain crushed stone slope protection. The base slab of the culvert inlet section and the base slab at the outlet shall be fabricated with toe walls that extend a minimum of 1 foot below the maximum depth of scour. The PCMG walls shall be constructed no less than 2 feet below the maximum anticipated scour depth. Where required, the slopes shall be armored with a 3-foot thick layer of riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

Construction Considerations – The box culvert soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 3 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. To control future settlement, in no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

The proposed box culvert will be bedded on a 2-foot thick layer of ¾-inch crushed stone conforming to Standard Specification 703.13. The stone will be reinforced with geogrid (see Special Provision 620) and encapsulated with Standard Specification 722.03 - Erosion Control Geotextile with a bottom elevation of approximately 614 feet.

The Contractor shall minimize disturbance to any silty soils that may be encountered and all subgrade surfaces should be protected from any unnecessary construction traffic. Remove any cobbles or boulders (in excess of 6 inches) encountered at the bearing elevation and replace with compacted gravel borrow or ¾-inch stone.

Earthwork and excavations may result in the exposure of silt or other soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with ¾-inch stone or compacted MaineDOT Standard Specification 703.20, Gravel Borrow.

Exposed soils may become saturated and water seepage may be encountered during construction. Therefore, there may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater, surface water infiltration and soil erosion.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of a steel plate pipe arch culvert which carries Andover Road over Split Brook in Rumford, Maine. The replacement structure will be called Split Brook Bridge. A subsurface investigation has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the new box culvert construction. This report presents the subsurface information obtained at the site during the subsurface investigations, foundation design recommendations and geotechnical design parameters.

The existing structure consists of a 8-foot 11-inch span by 6-foot rise steel plate pipe arch built in 1949. The pipe arch is approximately 60-feet long. The pipe arch is heavily rusted with several holes at the waterline and below. The pipe arch is sagging and distorted.

The Maine Department of Transportation (MaineDOT) Highway Program considered the poor condition of the existing steel pipe arch culvert, and determined that replacement of the culvert was warranted. Due to the significantly larger structure required to provide adequate hydraulic capacity and satisfy full bankfull width requirements, this project was transferred to the MaineDOT Bridge Program for design.

The proposed replacement structure will be a 18-foot span by 6-foot rise, approximately 56-foot long, precast concrete box culvert on a 5 degree skew. The box culvert will have 1-foot tall precast headwalls. The proposed box will have inlet and outlet toe walls. The downstream end of the culvert will be slope-tapered to match the 2H:1V sideslopes and the upstream end will have in-line, prefabricated concrete modular gravity (PCMG) walls to avoid conflict with the Rumford waterline. The invert of the box culvert will be embedded approximately 2 feet into the streambed and 1 foot of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box will be supported on a 2-foot thick layer of crushed stone reinforced with geogrid and encapsulated in geotextile fabric.

The new box culvert will be located on the same horizontal alignment as the existing bridge. The finished grade over the proposed precast box culvert will match the existing. An approximate 21-day road closure with a detour route on Route 5 and Route 2 will be utilized for maintenance of traffic during replacement of the structure.

2.0 GEOLOGIC SETTING

The pipe arch to be replaced carries Andover Road over Split Brook approximately 1.4 miles north of Route 2, as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) has not conducted surficial geology mapping of the Rumford Quadrangle.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the bridge site as metamorphic interbedded pelite and sandstone of the Perry Mountain Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling one (1) cased wash boring. The cased wash boring, BB-RSB-101, was drilled in the westbound shoulder of Andover Road, approximately 5 feet outside of the existing pipe. The boring location is shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile.

Boring BB-RSB-101 was drilled to a depth of approximately 32 feet below the roadway surface. The boring was drilled on June 25, 2014 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring log provided in Appendix A – Boring Logs and on Sheet 3 - Boring Logs.

The boring was drilled using solid stem auger and cased wash boring techniques. Soil samples were typically obtained in at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT dill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633-05 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in July 2013 and was found to deliver approximately 43 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 of 0.867 to the raw field N-values. This hammer efficiency factor (0.867) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring log and identified field testing requirements. The boring was located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was not conducted on soil samples recovered from the test borings.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test boring generally consisted of granular fill and sand deposits. The boring log is provided in Appendix A – Boring Log and on Sheet 3 – Boring Log. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. A brief summary description of the strata encountered follows:

5.1 Fill Soils

A layer of granular fill was encountered in the boring. The encountered thickness was approximately 8.5 at the boring location. The fill soils encountered consisted of brown, moist, fine to coarse sand, some gravel, trace silt.

One (1) corrected SPT N-value in the fill unit was 20 blows per foot (bpf), indicating the fill is medium dense in consistency.

5.2 Sand Layer

A layer of native sand was encountered in boring BB-RSB-101 below the fill materials. The deposit consisted of grey or brown, wet, fine to coarse sand, some to trace silt, little to trace gravel. The thickness of the sand deposit penetrated ranged from approximately 23 feet. The full thickness of the sand deposit was not penetrated during the investigations.

Corrected SPT N-values in the sand deposit ranged from 9 to 22 bpf indicating the soil is loose to medium dense in consistency.

Boring BB-RSB-101 was terminated approximately 23 feet into the sand layer at a depth of 32 feet below the ground surface (bgs). No refusal surface was encountered in the boring.

5.3 Groundwater

Groundwater was observed in boring BB-RSB-101 at a depth of approximately 9.5 feet bgs. The water level is indicated on the boring log in Appendix A. Note that water was introduced into the borehole during drilling operations. It is likely that the water level measured does not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels and construction activities.

6.0 FOUNDATION ALTERNATIVES

Bridge structures with several hydraulic openings were considered for replacement of the existing steel pipe arch. The following alternatives were considered:

- Replacement with a 14-foot 1-inch span by 8-foot 9-inch rise steel plate pipe arch culvert, to provide bankfull width and a natural bottom;
- Replacement with a 18-foot span by 6-foot rise precast concrete box culvert, to provide 1.3 bankfull width and a natural bottom;
- Replacement with a 19-foot span by 12-foot rise steel ellipse culvert to provide bankfull width and a natural bottom.

It was determined that roadway grade would need to be raised approximately 2.0 to 3.0 feet to provide sufficient cover for both the steel plate pipe arch culvert and the steel ellipse culvert options. Due to the substantial amount of approach work and associated costs required to raise the road elevation, these two options were eliminated from further consideration. The precast concrete box culvert with a 18-foot span by 6-foot rise was found to meet the site hydraulic conditions and did not require the roadway elevation to be raised and was therefore the alternative selected. The length of the box will be approximately 56 feet with a 5 degree skew. The box will be embedded approximately 2 feet into the streambed and 1 foot of special fill will be placed inside to create a natural streambed.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 18-foot span by 6-foot rise precast concrete box culvert with a 5 degree skew. The proposed box culvert will have 1-foot tall precast headwalls. The downstream end of the culvert will be slope-tapered to match the 2:1 sideslopes and the upstream end will have in-line, prefabricated concrete modular gravity (PCMG) walls to avoid conflict with the waterline which crosses the brook just upstream of the proposed structure. The invert of the box culvert will be embedded approximately 2 feet into the streambed and 1 foot of special fill will be placed inside the bottom of the culvert to create a natural streambed.

The precast concrete box shall include accommodations for toe walls at both the inlet and outlet ends to prevent undermining per MaineDOT Bridge Design Guide (BDG) Section 8.3.1. The toe walls should extend a minimum of 1 foot below the maximum depth of scour.

Precast concrete box culverts are typically detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness and reinforcement. The precast concrete box culvert shall be supplier-designed in accordance with AASHTO LRFD Bridge Design Specifications 7th Edition 2014 (LRFD) and Standard Specification 534 - Precast Structural Concrete. The loading specified for the structure should be Modified HL-93 Strength 1 in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the

MaineDOT BDG Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: ϕ=32°, γ = 125 pcf.

The precast concrete box culvert shall be constructed in accordance with MaineDOT BDG Section 8 and Standard Specification 534.

The box culvert will be bedded on a 2-foot thick layer of ¾-inch crushed stone underlain with geogrid and wrapped in geotextile fabric. The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 3 inches. The crushed stone bedding should be placed in 12-inch maximum thick lifts and compacted with a minimum of four passes of a large walk behind compactor. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1 foot by 1 foot concrete headwalls are recommended.

7.3 Precast Concrete Outlet Walls and Outlet and Inlet Toe Walls

The precast concrete box culvert's outlet walls will be slope-tapered to match the 2H:1V sideslopes. The left and right outlet walls will share the same precast base slab or will be cast separately and joined at the site. The sloped outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The outlet walls shall be designed to resist lateral earth pressures, vehicular loads, creep and temperature and shrinkage deformations of the concrete box culvert. The outlet walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet per LRFD Article 3.11.6.4.

Outlet walls that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient, K_o, of 0.47 assuming a level backslope. The at-rest earth pressure coefficient will change if the backslope conditions are different. Wingwalls sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a, of 0.52 assuming a 2H:1V backslope. The active earth pressure coefficient will also change if the backslope conditions are different. See Appendix C – Calculations for supporting documentation.

The bottom slab connecting the left and right walls at the outlet of the box culvert and the upstream section of the box culvert shall include toe walls to prevent undermining per MaineDOT BDG Section 8.3.1. The outlet and inlet toe walls should extend a minimum of 1 foot below the maximum depth of scour.

7.4 Bearing Resistance

The proposed precast concrete box culvert will be bedded on a 2-foot thick layer of ¾-inch crushed stone underlain by a geogrid and encapsulated with geotextile fabric with a bottom elevation of approximately 614 feet. The native sands at this elevation are expected to be medium dense in consistency. These soils are characterized as having adequate bearing resistance.

For a precast concrete box culvert with a base width of 20 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 7.5 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a factored bearing resistance of 5.0 ksf. The service limit state bearing resistance may govern the design. In no instance shall be bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'c$. See Appendix B – Calculations for supporting calculations.

7.5 Precast Concrete Modular Gravity (PCMG) Walls

The upstream end of the precast concrete box culvert will have inlet walls consisting of Precast Concrete Modular Gravity (PCMG) walls founded on native soils. The PCMG walls will be constructed below the $Q_{1.1}$ elevation, therefore the wall base units shall be constructed no less than 2.0 feet below the maximum anticipated scour depth. In general, PCMG wingwalls should only be used in streams where the flow velocities are low, and the potential for scour is low. The PCMG walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item.

The bearing resistance for PCMG walls founded on a 6 by 12-inch leveling slab and embedded for frost and scour shall be investigated at the strength limit state using factored loads and a factored bearing resistance provided in Figure 7-1, below. Based on presumptive bearing resistance values, a factored bearing resistance of 5 ksf may be used to control settlement when analyzing the service limit state (see Appendix B – Calculations for supporting calculations.) The vertical stress may be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1.

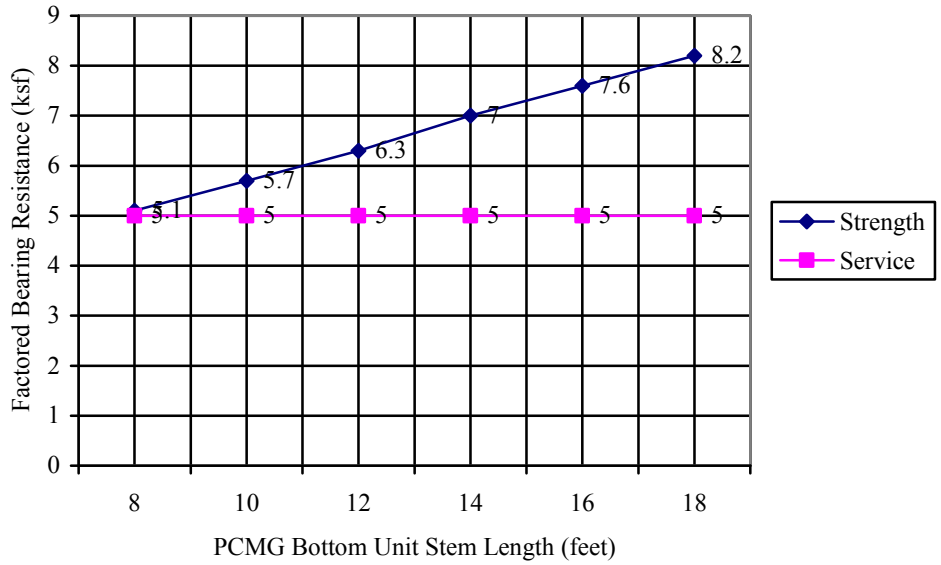


Figure 7-1 Factored Bearing Resistance for PCMG Walls

The PCMG walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2 feet, per LRFD Article 3.11.6.4.

For the lowest PCMG unit on soil the location of the resultant of the reaction forces at the strength limit state should be within the middle one-half (1/2) of the footing width.

The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Failure by sliding shall be investigated by the wall designer-supplier. A sliding resistance factor, φ_{τ}, of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on soil. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 = 0.80 x tan 30° at the foundation soil to concrete unit interfaces and a maximum frictional coefficient of 0.58 = tan 30° at foundation soil to soil-infill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4 and Table 10.5.5.2.2-1.

7.6 Settlement

The native sands encountered at the site are loose to medium dense in consistency. Elastic settlements due to the proposed precast concrete box culvert were calculated and are estimated to be approximately 1.5 inches. This settlement will occur relatively quickly. All loose or soft soils encountered at the foundation elevation for the precast box culvert will be

excavated in its entirety to allow for construction of the 2-foot thick geotextile-wrapped crushed stone mat. Furthermore, in order to minimize any long term distress to the proposed structure, contract documents shall specify that all silt or other soft soils within the box culvert footprint and PCMG wall footprint be removed to full depth from the excavation and replaces with crushed stone when encountered. With these provisions, post-construction settlement at the location of the replacement structure is anticipated to be minimal.

7.7 Frost Protection

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Rumford has a design freezing index (DFI) of approximately 1650 F-degree days. A water content of 20% was used for saturated, coarse-grained soils at the potential bearing elevation of PCMG inlet walls. These components correlate to a frost depth of 6 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Rumford has a DFI from the Modberg database of approximately 1631 F-degree days. A water content of 20% was used. These components correlate to a frost depth of approximately 6.9 feet.

Based on an averaged, calculated depth of frost, we recommend that foundations be designed with a minimum embedment of 6.4 feet for frost protection. See Appendix B – Calculations for supporting calculations.

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

7.8 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and wingwalls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1-foot below the maximum depth of scour. Inlet and outlet toe walls shall also be protected with riprap aprons. The PCMG walls shall be constructed no less than 2.0 feet below the maximum anticipated scour depth.

Where required, slopes shall be armored with a 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 - Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

7.9 Seismic Design Considerations

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

7.10 Construction Considerations

The soil envelope and backfill for the precast box shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 3 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. To control future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

The proposed box culvert will be bedded on a 2-foot thick layer of ¾-inch crushed stone, conforming to Standard Specification 703.13, reinforced with geogrid conforming to Special Provision 620 in Appendix C, and encapsulated in Class 1 non-woven erosion control geotextile fabric meeting MaineDOT Standard Specification 722.03 with a bottom elevation of approximately 614 feet. Based on the soils encountered in the borings, medium dense sands will be present at this elevation. The boring may not accurately represent soil conditions within the entire project limits. If silt or other soft soils are discovered within the box culvert footprint, or at the bearing elevation of the PCMG wall, all silt or soft soils shall be excavated to full depth and replaced with crushed stone.

Any silt soils encountered shall be excavated using a smooth edged backhoe bucket to minimize disturbance to the layer. The use of heavy equipment on any silt layer shall be prohibited.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. Any cobbles or boulders (in excess of 6 inches) encountered at the bearing elevation shall be removed and replaced with compacted granular borrow or ¾-inch stone.

Earthwork and excavations may result in the exposure of silt or other soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with ¾-inch stone or compacted MaineDOT Standard Specification 703.20, Gravel Borrow

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

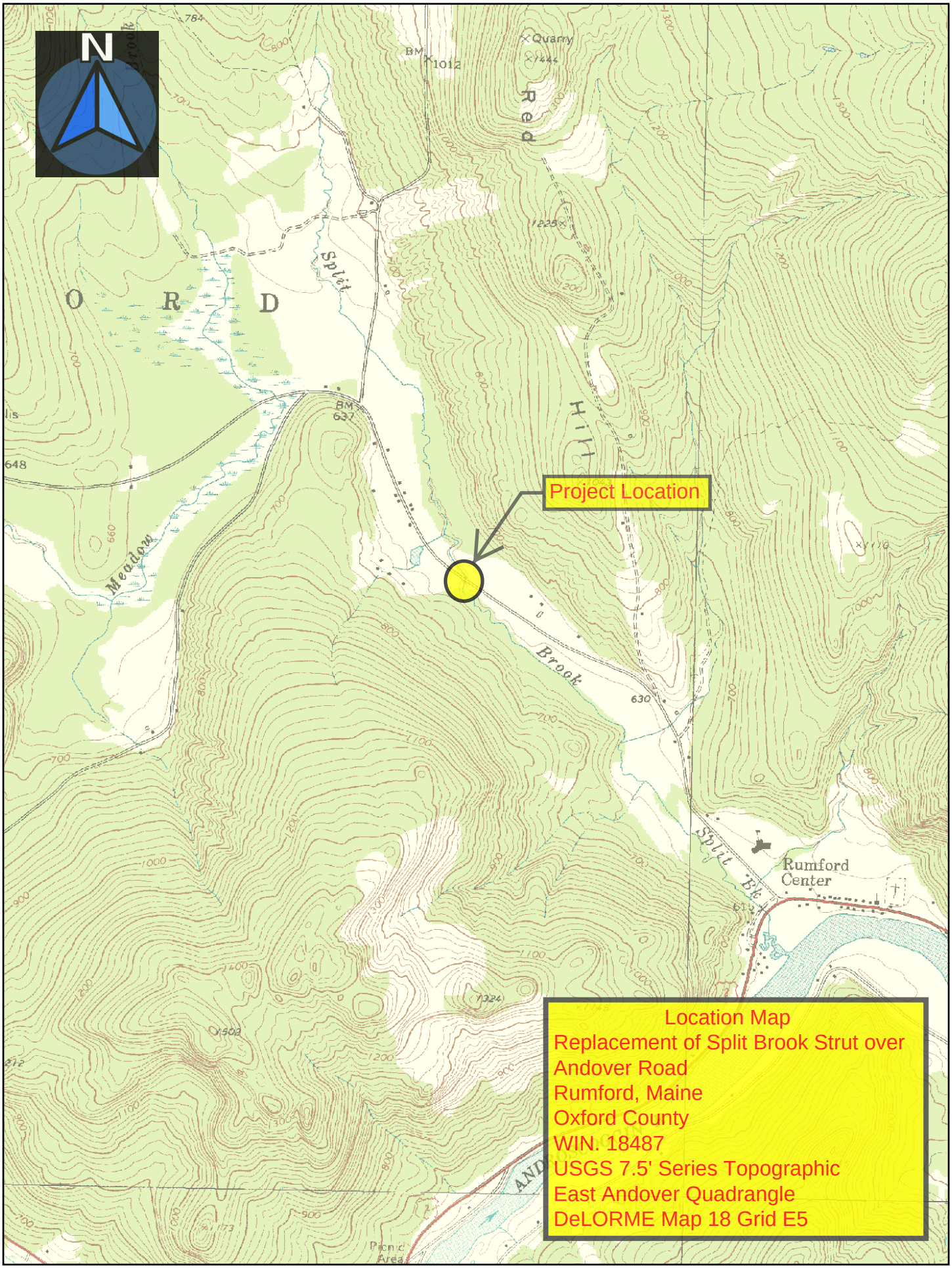
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed construction of Meadow Brook Bridge on Andover Road in Rumford, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

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

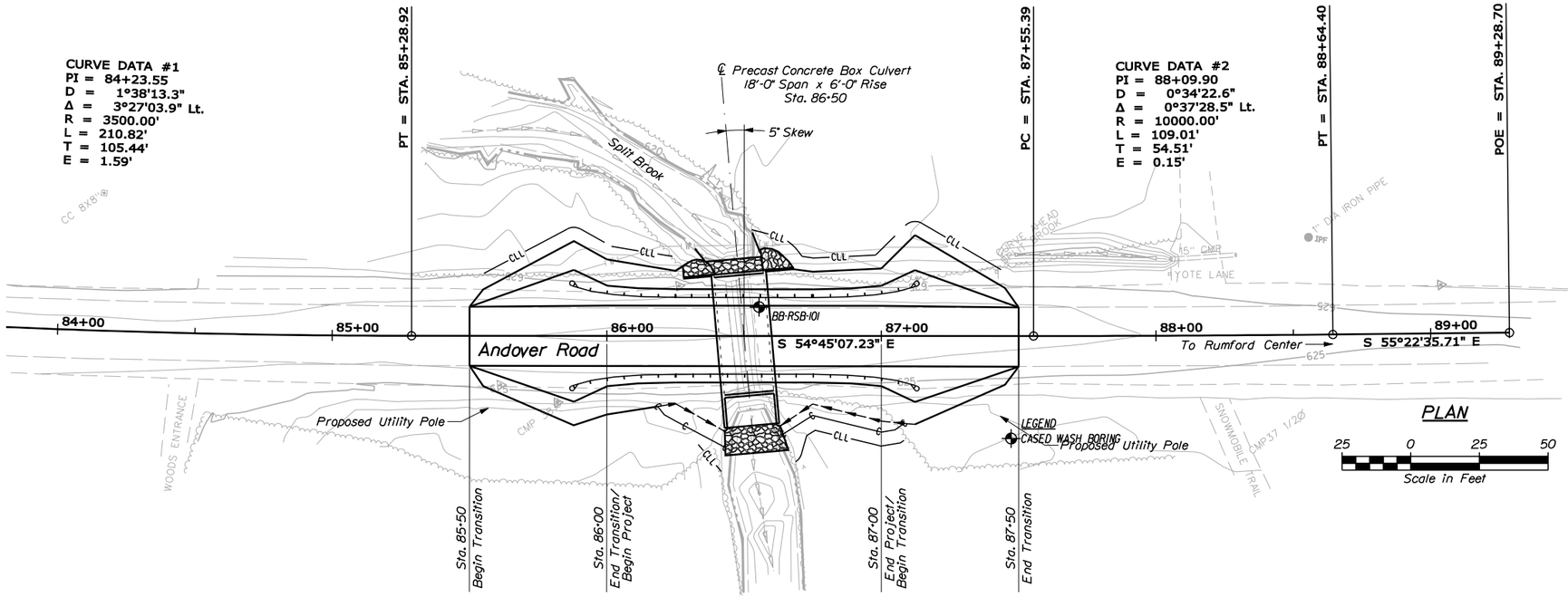
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

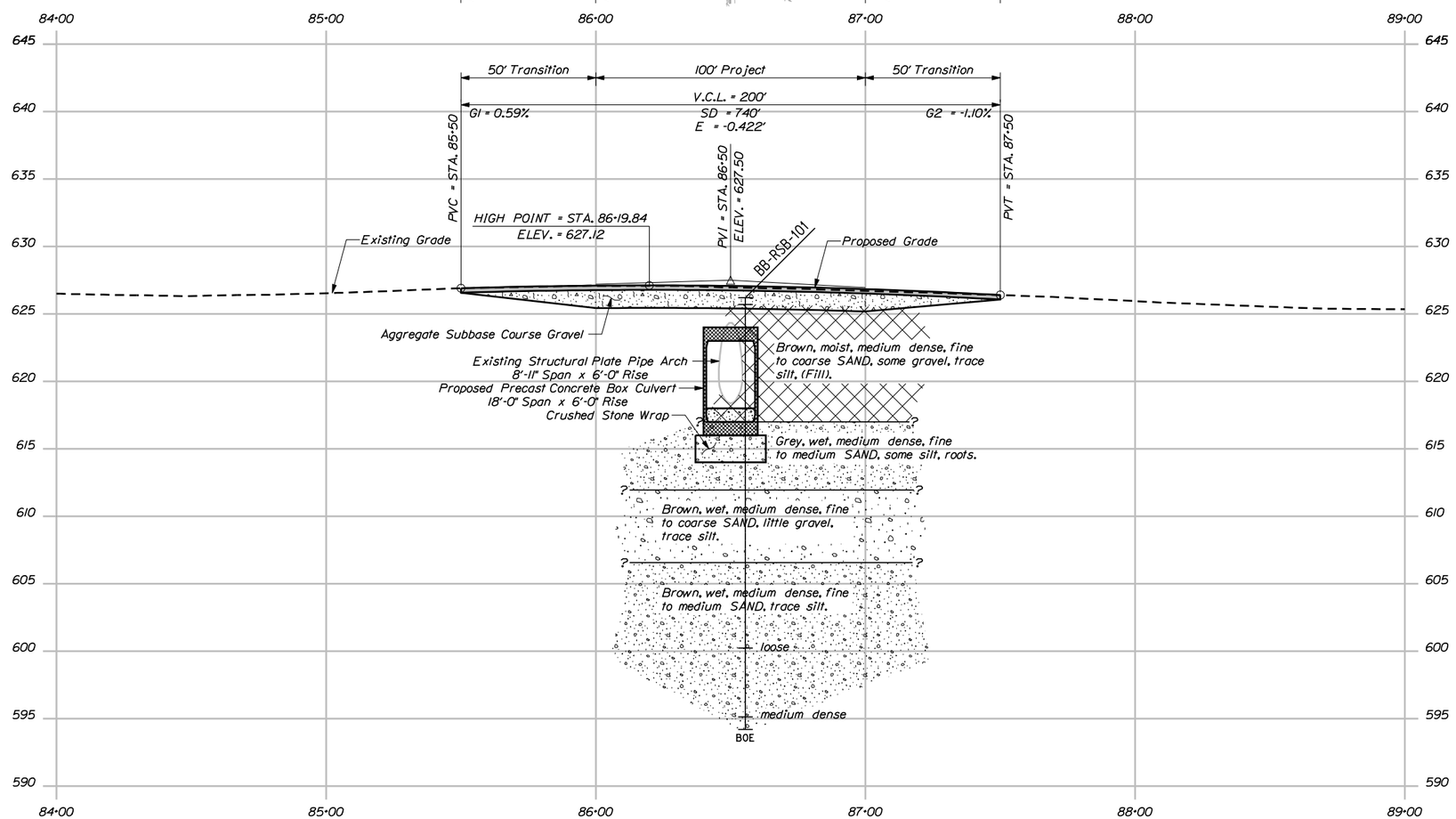
Sheets

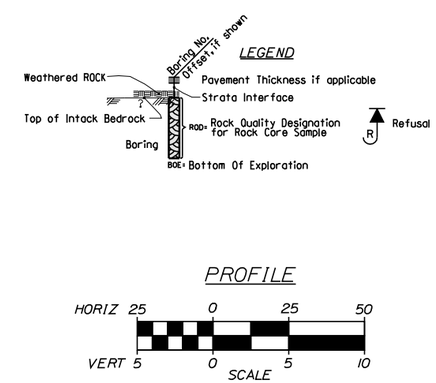


Map Scale 1:24000

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.







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

18487.00

WIN 18487.00

BRIDGE NO. 6487

BRIDGE PLANS

PROJ. MANAGER	S. BOOGE	BY	DATE
DESIGN-DETAILED	T. HUELM	D. SHAW	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	L.KRUSINSKI	T.WHITE	SEPT 2014
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

SPLIT BROOK BRIDGE
SPLIT BROOK
RUMFORD OXFORD COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\006_BLP&SP1.dgn Division: GEOTECH Username: terry.white Date:12/12/2014

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/12/2014

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Split Brook Bridge #6487 carries Andover Road over Split Brook
Location: Rumford, Maine

Boring No.: BB-RSB-101
WIN: 18487.00

Driller:	MaineDOT	Elevation (ft.)	626.2	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Daggett/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/25/2014; 07:00-10:30	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	86+55.5, 10.5 ft Lt.	Casing ID/OD:	NW	Water Level*:	9.5 ft bgs.
Hammer Efficiency Factor: 0.867		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
W = 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	625.70	6" PAVEMENT. (0.50)	
	51	24/24	1.00 - 3.00						Brown, moist, fine to coarse SAND, some gravel, trace silt. (Fill).	
5	10	24/8	5.00 - 7.00	9/7/7/9	14	20	33		Similar to above, medium dense.	
							36			
							46			
							51			
							69	617.20	(9.00)	
10	20	24/14	10.00 - 12.00	8/5/7/8	12	17	13		Grey, wet, medium dense, fine to medium SAND, some silt, roots.	
							19			
							43			
							25			
							46	612.20	(14.00)	
15	30	24/18	15.00 - 17.00	4/8/7/7	15	22	10		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	
							31			
							42			
							46			
							40	606.70	(19.50)	
20	40	24/18	20.00 - 22.00	5/4/4/6	8	12	19		Brown, wet, medium dense, fine to medium SAND, trace silt.	
							23			
							47			
							66			
							65			
25	50	24/19	25.00 - 27.00	2/3/3/5	6	9	16		Similar to above, except loose.	
							22			
							33			
							41			
							45			
30	60	24/20	30.00 - 32.00	3/4/4/4	8	12			Similar to above, except medium dense.	
								594.20	(32.00) Bottom of Exploration at 32.00 feet below ground surface. NO REFUSAL	
35										
40										
45										
50										

Remarks:

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

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

Boring No.: BB-RSB-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18487.00

WIN 18487.00
BRIDGE NO. 6487

BRIDGE PLANS

PROJ. MANAGER	S. BOOGE	BY	DATE	
DESIGN-DETAILED	T. HUELM	D. SHAW		SIGNATURE
CHECKED-REVIEWED				
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	DEC 2014	P.E. NUMBER
DESIGN3-DETAILED3				
REVISIONS 1				DATE
REVISIONS 2				
REVISIONS 3				
REVISIONS 4				
FIELD CHANGES				

SPLIT BROOK BRIDGE
SPLIT BROOK
RUMFORD OXFORD COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Log

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Split Brook Bridge carries Andover Road over Split Brook
Location: Rumford, Maine

Boring No.: BB-RSB-101
PIN: 18487.00

Driller:	MaineDOT	**Elevation (ft.)**	626.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/25/2014; 07:00-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	86+55.5, 10.5 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.867	**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	625.70	6" PAVEMENT. 0.50	
	S1	24/24	1.00 - 3.00						Brown, moist, fine to coarse SAND, some gravel, trace silt. (Fill).	
5	1D	24/8	5.00 - 7.00	9/7/7/9	14	20	33		Similar to above, medium dense.	
							36			
							46			
							51			
							69	617.20	9.00	
10	2D	24/14	10.00 - 12.00	8/5/7/8	12	17	13		Grey, wet, medium dense, fine to medium SAND, some silt, roots.	
							19			
							43			
							25			
							46	612.20	14.00	
15	3D	24/18	15.00 - 17.00	4/8/7/7	15	22	10		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	
							31			
							42			
							46			
							40	606.70	19.50	
20	4D	24/18	20.00 - 22.00	5/4/4/6	8	12	19		Brown, wet, medium dense, fine to medium SAND, trace silt.	
							23			
							47			
							66			
25							65			

Remarks:

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

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

Boring No.: BB-RSB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Split Brook Bridge carries Andover Road over Split Brook
Location: Rumford, Maine

Boring No.: BB-RSB-101
PIN: 18487.00

Driller:	MaineDOT	**Elevation (ft.)**	626.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Daggett/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/25/2014; 07:00-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	86+55.5, 10.5 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.5 ft bgs.
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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	5D	24/19	25.00 - 27.00	2/3/3/5	6	9	16			Similar to above, except loose.	
							22				
							33				
							41				
							45				
30	6D	24/20	30.00 - 32.00	3/4/4/4	8	12				Similar to above, except medium dense.	
								594.20		32.00	
										Bottom of Exploration at 32.00 feet below ground surface. NO REFUSAL	
35											
40											
45											
50											

Remarks:

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

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

Appendix B

Calculations

Earth Pressure:

Backfill engineering strength parameters

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

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi_1 := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Frame Stem Walls

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

Reference: Das, Principles of Foundation Engineering, 4th Edition, pg 336

$$K_o := 1 - \sin(\phi_1)$$

$$K_o = 0.47$$

Active Earth Pressure - Rankine Theory
Independent walls free to rotate

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

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot deg - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.31$$

- For a sloped backfill

β = Angle of fill slope to the horizontal (1V:2H slope case)

$$\beta := 26.56 \cdot deg$$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.52$$

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

Analysis : Bearing Resistance of Precast Box - Foundation on Fine to Coarse Sand

Assumptions

1. Box bottom is embedded 2 feet below the streambed

2. Assumed parameters for undrained medium dense fine to coarse sand
   No soil sample available for testing. Visual inspection from field boring logs place the sample in SM to SW range with little to no variation in design values between these classifications.
   Wet unit weight = 125 pcf
   Dry unit weight = 121 pcf
   ϕ = 32 degrees, undrained
   Su = undrained shear strength, c = 0 psf

3. Method used: Terzaghi, use strip equations since L>B

Foundation Widths and Depth

$B := 20ft$

$D_f := 2.0 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

Foundation soils:

Samples of the fine to coarse-grained sands

$\gamma 1_{sat} := 125 \cdot pcf$

$\gamma 1_d := 121 \cdot pcf$

$\phi := 32 \cdot deg$

$c := 0 \cdot psf$

Nominal Bearing Resistance - For Service Limit State

Method**:** LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Samples of the medium dense, fine to coarse grained sand

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Fine to medium sand, silty or clayey med. to coarse sand (SW, SM, SC)	medium dense	4-8 ksf	5 ksf

Fine to coarse sand deposit is medium dense in consistency. Recommend 5 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

$s_\gamma := 1.0$ $\quad$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for undrained silt ϕ = 32 degrees

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

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

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $\quad$ $q = 0.125 \cdot ksf$

$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$

$q_n = 16.7 \cdot ksf$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$q_r = 7.5 \cdot ksf$ for $B = 20 \cdot ft$

7.5 ksf for strength limit state design for a 20-foot wide precast box foundation.

Analysis : Bearing Resistance of PCMG Walls Fine to Coarse Sand

Bearing resistance of PCMG Walls. Assume bottom unit is placed at the bottom elevation of the crushed stone mat supporting the precast concrete box. This is approx. 4' below the streambed at EL. 614.00 ft. Q1.1 is approximated at 620.5 ft. PCMG walls should be above Q1.1

Assumptions

1. Box bottom is embedded 4 feet below the streambed

2. Assumed parameters for undrained medium dense fine to coarse sand
   No soil sample available for testing. Visual inspection from field boring logs place the sample in SM to SW range with little to no variation in design values between these classifications.
   Wet unit weight = 125 pcf
   Dry unit weight = 121 pcf
   ϕ = 32 degrees, undrained
   Su = undrained shear strength, c = 0 psf

3. Method used: Terzaghi, use strip equations since L>B

Foundation Widths and Depth

$D_f := 4.0 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

$$B := \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} ft$$

Foundation soils:

Samples of the fine to coarse-grained sands

$\gamma 1_{sat} := 125 \cdot pcf$

$\gamma 1_d := 121 \cdot pcf$

$\phi := 32 \cdot deg$

$c := 0 \cdot psf$

Nominal Bearing Resistance - For Service Limit State

Method: LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Samples of the medium dense, fine to coarse grained sand

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Fine to medium sand, silty or clayey med. to coarse sand (SW, SM, SC)	medium dense	4-8 ksf	5 ksf

Fine to coarse sand deposit is medium dense in consistency. Recommend 5 ksf to limit settlement to 1.0 inch for Service Limit State Loads

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

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

$s_\gamma := 1.0$ $\qquad$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for undrained silt ϕ = 32 degrees

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

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

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $\qquad$ $q = 0.25 \cdot ksf$

$$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = \begin{pmatrix} 11.3 \\ 12.7 \\ 14.1 \\ 15.4 \\ 16.8 \\ 18.2 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$$q_r = \begin{pmatrix} 5.1 \\ 5.7 \\ 6.3 \\ 7 \\ 7.6 \\ 8.2 \end{pmatrix} \cdot ksf \quad \textbf{for} \quad B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

Factored Bearing Resistance for extreme limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 1.0$

$q_r := q_n \cdot \varphi_b$

$$q_r = \begin{pmatrix} 11.3 \\ 12.7 \\ 14.1 \\ 15.4 \\ 16.8 \\ 18.2 \end{pmatrix} \cdot ksf \quad \textbf{for} \quad B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

Calculation of Elastic Settlement at Station 86+50

Select Soil Profile based on N-values from BB-RSB-101

Reference: FHWA Soils and Foundation Reference Manual - Volume 1 (FHWA-NHI-06-088)

Native Sands - fine to coarse sand, some gravel, trace silt (fill) - medium dense

$H_0 := 9.5 \cdot ft$ $\gamma_{sand0} := 125 \cdot pcf$

Assumed water Elev. 3' above crushed stone (conservative)

Bottom of crushed stone wrap

Native Sands - fine to medium sand, some silt - medium dense

$H_1 := 2 \cdot ft$ $\gamma_{sand1} := 125 \cdot pcf$ $\gamma_w := 62.4 \cdot pcf$ $N_1 := 17$

Native Sands - fine to course sand, little gravel, trace silt - medium dense

$H_2 := 6 \cdot ft$ $\gamma_{sand2} := 120 \cdot pcf$ $N_2 := 22$

Native Sands - fine to coarse sand - medium dense

$H_3 := 5 \cdot ft$ $\gamma_{sand3} := 125 \cdot pcf$ $N_3 := 12$

Native Sands - fine to coarse sand - loose

$H_4 := 5 \cdot ft$ $\gamma_{sand4} := 115 \cdot pcf$ $N_4 := 9$

Native Sands - fine to medium sand - loose

$H_5 := 8 \cdot ft$ $\gamma_{sand5} := 125 \cdot pcf$ $N_5 := 12$

Calculate Change in Stress due to New Precast Concrete Box

RECTANGULAR LOADS
UNIFORM VERTICAL LOADS

Project Name: Rumford Split Brook
Client: MaineDOT
Project Number: 18487.00
Project Manager: S. Bodge
Date: 11/18/2014
Computed by: BS

Corner Point P1. Coordinates (0.0, 0.0)
Corner Point P2. Coordinates (57.0, 20.0)

p load/unit area = 570.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 27.5 (ft) Y= 10.0 (ft)

Z (ft)	Change in Vertical Stress (psf)		
0.00	570.00		
1.00	569.76	at 1.0 ft	$\Delta\sigma_{z1} := 569.76 \cdot psf$
2.00	568.12		
3.00	564.00		
4.00	556.80		
5.00	546.41	at 5.0 ft	$\Delta\sigma_{z2} := 546.41 \cdot psf$
6.00	533.15		
7.00	517.57		
8.00	500.30		
9.00	481.94		
10.00	463.02		
11.00	443.98	at 11.0 ft	$\Delta\sigma_{z3} := 443.98 \cdot psf$
12.00	425.11		
13.00	406.66		
14.00	388.78		
15.00	371.56		
16.00	355.06		
17.00	339.31	at 17.0 ft	$\Delta\sigma_{z4} := 339.31 \cdot psf$
18.00	324.30		
19.00	310.03		
20.00	296.47		
21.00	283.61		
22.00	271.41		
23.00	259.84		
24.00	248.87	at 24.0 ft	$\Delta\sigma_{z5} := 248.87 \cdot psf$
25.00	238.46		
26.00	228.60		
27.00	219.24		

Layer 1

Existing effective overburden stress at SPT interval Layer 1 (11 ft bgs)

$$\sigma_1 := H_0 \cdot (\gamma_{sand0}) + 1.5ft \cdot (\gamma_{sand1} - \gamma_w)$$

$$\sigma_1 = 1.281 \times 10^3 \cdot psf$$

Corrected SPT N-value for overburden per LRFD 10.4.6.2.4

$$N_1 = 17$$

$$CN_1 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_1}\right)$$

$$CN_1 = 1.151$$ Should not exceed 2.0

$$Ncor1 := CN_1 \cdot N_1$$

$$Ncor1 = 19.561$$

FHWA NHI-06-088, Figure 7-7, pg 7-17 curve for "Well graded silty sand and gravel"

Bearing Capacity Index $C_1 := 76$

Effective overburden stress at midpoint of layer

$$\sigma_1 := \frac{H_1}{2} \cdot (\gamma_{sand1} - \gamma_w)$$

$$\sigma_1 = 62.6 \cdot psf$$

Per FHWA-NHI-06-088, for shallow surface deposits, a min. value of 200 psf should be used to prevent unrealistic settlement predictions (pg 7-16)

$$\sigma_1 := 200 \cdot psf$$

Settlement

$$\Delta H_1 := \overrightarrow{\left[H_1 \cdot \frac{1}{C_1} \cdot \log\left[\frac{(\sigma_1) + \Delta\sigma_{z1}}{\sigma_1}\right]\right]}$$

$$\Delta H_1 = 0.185 \cdot in$$

Layer 2

Existing effective overburden stress at SPT interval Layer 2 (16 ft bgs)

$$\sigma_2 := H_0 \cdot (\gamma_{sand0}) + H_1 \cdot (\gamma_{sand1} - \gamma_w) + 4.5ft \cdot (\gamma_{sand2} - \gamma_w)$$

$$\sigma_2 = 1.572 \times 10^3 \cdot psf$$

Corrected SPT N-value for overburden per LRFD 10.4.6.2.4

$$N_2 = 22$$

$$CN_2 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_2}\right)$$

$$CN_2 = 1.082$$ Should not exceed 2.0

$$Ncor2 := CN_2 \cdot N_2$$

$$Ncor2 = 23.811$$

FHWA NHI-06-088, Figure 7-7, pg 7-17 curve for "Clean, well graded fine to coarse SAND"

Bearing Capacity Index $C_2 := 77$

Effective overburden stress at midpoint of layer

$$\sigma_2 := H_1 \cdot (\gamma_{sand1} - \gamma_w) + \frac{H_2}{2} \cdot (\gamma_{sand2} - \gamma_w) \qquad \sigma_2 = 298 \cdot psf$$

Settlement

$$\Delta H_2 := \overrightarrow{\left[H_2 \cdot \frac{1}{C_2} \cdot \log\left[\frac{(\sigma_2) + \Delta\sigma_{z2}}{\sigma_2}\right]\right]}$$

$$\Delta H_2 = 0.423 \cdot in$$

Layer 3

Existing effective overburden stress at SPT interval in Layer 3 (21 ft bgs)

$$\sigma_3 := H_0 \cdot (\gamma_{sand0}) + H_1 \cdot (\gamma_{sand1} - \gamma_w) + H_2 \cdot (\gamma_{sand2} - \gamma_w) + 3.5ft \cdot (\gamma_{sand3} - \gamma_w)$$

$$\sigma_3 = 1877.4 \cdot psf$$

Corrected SPT N-value for overburden per LRFD 10.4.6.2.4

$$N_3 = 12$$

$$CN_3 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_3}\right)$$

$$CN_3 = 1.023$$ Should not exceed 2.0

$$Ncor3 := CN_3 \cdot N_3$$

$$Ncor3 = 12.275$$

FHWA NHI-06-088, Figure 7-7, pg 7-17 curve for "Well graded fine to medium silty sand"

Bearing Capacity Index $C_3 := 48$

Effective overburden stress at midpoint of layer

$$\sigma_3 := H_1 \cdot (\gamma_{sand1} - \gamma_w) + H_2 \cdot (\gamma_{sand2} - \gamma_w) + \frac{H_3}{2} \cdot (\gamma_{sand3} - \gamma_w)$$

$$\sigma_3 = 627.3 \cdot psf$$

Settlement

$$\Delta H_3 := \overrightarrow{\left[H_3 \cdot \frac{1}{C_3} \cdot \log\left[\frac{(\sigma_3) + \Delta\sigma_{z3}}{\sigma_3}\right]\right]}$$

$$\Delta H_3 = 0.291 \cdot in$$

Layer 4

Existing effective overburden stress at SPT Interval (26 ft bgs) (Layer 4)

$$\sigma_4 := H_0\cdot(\gamma_{sand0}) + H_1\cdot(\gamma_{sand1} - \gamma_w) + H_2\cdot(\gamma_{sand2} - \gamma_w) + H_3\cdot(\gamma_{sand3} - \gamma_w) + 3.5ft\cdot(\gamma_{sand4} - \gamma_w)$$

$$\sigma_4 = 2155.4\cdot psf$$

Corrected SPT N-value for overburden per LRFD 10.4.6.2.4

$$N_4 = 9$$

$$CN_4 := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_4}\right)$$

$$CN_4 = 0.977$$ Should not exceed 2.0

$$Ncor4 := CN_4\cdot N_4$$

$$Ncor4 = 8.791$$

FHWA NHI-06-088, Figure 7-7, pg 7-17 curve for "Well graded fine to medium silty SAND"

Bearing Capacity Index $C_4 := 40$

Effective overburden stress at midpoint of layer

$$\sigma_4 := H_1\cdot(\gamma_{sand1} - \gamma_w) + H_2\cdot(\gamma_{sand2} - \gamma_w) + H_3\cdot(\gamma_{sand3} - \gamma_w) + \frac{H_4}{2}\cdot(\gamma_{sand4} - \gamma_w)$$

$$\sigma_4 = 915.3\cdot psf$$

Settlement

$$\Delta H_4 := \overrightarrow{\left[H_4\cdot\frac{1}{C_4}\cdot\log\left[\frac{(\sigma_4) + \Delta\sigma_{z4}}{\sigma_4}\right]\right]}$$

$$\Delta H_4 = 0.205\cdot in$$

Layer 5

Existing effective overburden stress at SPT Interval (17 ft bgs) (Layer 4)

$$\sigma_5 := H_0\cdot(\gamma_{sand0}) + H_1\cdot(\gamma_{sand1} - \gamma_w) + H_2\cdot(\gamma_{sand2} - \gamma_w) + H_3\cdot(\gamma_{sand3} - \gamma_w) \ldots + H_4\cdot(\gamma_{sand4} - \gamma_w) + 3.5ft\cdot(\gamma_{sand5} - \gamma_w)$$

$$\sigma_5 = 2453.4\cdot psf$$

Corrected SPT N-value for overburden per LRFD 10.4.6.2.4

$$N_5 = 12$$

$$CN_5 := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_5}\right)$$

$$CN_5 = 0.933$$ Should not exceed 2.0

$$Ncor5 := CN_4\cdot N_4$$

$$Ncor5 = 8.791$$

FHWA NHI-06-088, Figure 7-7, pg 7-17 curve for "Well graded fine to medium silty SAND"

Bearing Capacity Index $C_5 := 40$

Effective overburden stress at midpoint of layer

$$\sigma_5 := H_1\cdot(\gamma_{sand1} - \gamma_w) + H_2\cdot(\gamma_{sand2} - \gamma_w) + H_3\cdot(\gamma_{sand3} - \gamma_w) + H_4\cdot(\gamma_{sand4} - \gamma_w) + \frac{H_5}{2}\cdot(\gamma_{sand5} - \gamma_w)$$

$$\sigma_5 = 1297.2\cdot psf$$

Settlement

$$\Delta H_5 := \overrightarrow{\left[H_5\cdot\frac{1}{C_5}\cdot\log\left[\frac{(\sigma_5) + \Delta\sigma_{z5}}{\sigma_5}\right]\right]}$$

$$\Delta H_5 = 0.183\cdot in$$

$$\Delta H_T := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5$$

$$\boxed{\Delta H_T = 1.287\cdot in}$$

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

From Design Freezing Index Map: Rumford, ME
DFI = 1650 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1700 $d1 := 72.4$

For DFI = 1600 $d2 := 70.2$

$$d := in \cdot \left(\frac{d2 - d1}{10} \cdot 2 + d1 \right)$$

Depth of Frost Penetration $d = 72 \cdot in$ $d = 6 \cdot ft$

Method 2 - ModBerg Software

Examine frost depth for foundations placed on coarse grained soils.; use ModBerg weather database information for Rumford

--- ModBerg Results ---

Project Location: Rumford, Maine
Air Design Freezing Index = 1631 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1305 F-days
Mean Annual Temperature = 43.5 deg F
Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	82.3	20.0	125.0	34	46	3.8	1.9	3,600

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

Total Depth of Frost Penetration = 6.86 ft = 82.3 in.
****

Recommendation: 6.4 feet for design of foundations constructed on coarse grained soils

Appendix C

Special Provision

SPECIAL PROVISION
SECTION 620
REINFORCEMENT GEOGRID

Description This work shall consist of furnishing and installing reinforcement geogrid in accordance with these specifications and in reasonably close conformity with the lines, grades, and dimensions shown on the plans or as directed by the Resident.

Material Geogrids shall consist of a regular network of integrally connected polymeric tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil, aggregate or other material. The geogrid structure shall be dimensionally stable to retain its geometry under construction stresses and shall have high resistance to damage during construction, ultraviolet degradation, and all forms of chemical and biological degradation encountered in the soil being reinforced. Woven geogrids are not acceptable for this application.

The reinforcement geogrid shall meet or exceed the Minimum Average Roll Values (MARV) of the properties in Table 1.

Acceptable manufacturers for reinforcement geogrids must be approved by the Resident.

Table 1 - Physical Property Requirements
(Non-Woven Biaxial Reinforcement Geogrid)

Reinforcement Geogrid Mechanical Property	Test Method	Minimum Average Roll Value (MARV)[1]
Tensile Strength at 5% Strain MD	ASTM D-6637	600 lb/ft
Tensile Strength at 5% Strain XD	ASTM D-6637	1,200 lb/ft
Rib Junction Strength	GRI-GG2	1,000 lb/ft in both directions
Aperture Openings		Between 0.75 and 3 inches
Percent Open Area		50 to 80%

Certification Prior to construction the Contractor shall submit to the Resident the Manufacturer's certification that the geogrid supplied has been evaluated in full compliance with this Specification and is fit for long-term, critical soil reinforcement applications. The Contractor's submittal package shall include, but not be limited to, actual tests for tension/creep, durability/aging, construction damage, and quality control tensile testing.

Delivery, Storage and Handling The Contractor shall check the reinforcing geogrid upon delivery to ensure that the proper material has been received. Each geogrid roll shall be shipped in a protective bag and clearly marked with roll number, lot number, geogrid style and principle strength direction. During all periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F and all deleterious materials that might otherwise become affixed to the geogrid and affect its performance. The manufacturer's recommendations shall be

[1] Values are minimum average roll values determined in accordance with ASTM D-4759.

followed with regard to protection from direct sunlight. The geogrid shall be stored off the ground in a clean, dry environment out of the pathway of construction equipment.

Construction Requirements Reinforcement geogrid shall be installed, in accordance with the manufacturer's recommendations, to the proper elevation and alignment, as shown on the plans or as directed by the Resident.

1. The geogrid shall be laid at the proper elevation and alignment as shown on the plans. The Contractor shall verify correct orientation of the geogrid. Geogrid may be temporarily secured in-place with staples, pins, sand bags or backfill as required by fill properties, fill placement procedures, or weather conditions, or as directed by the Resident.

2. Reinforcement geogrid shall be oriented such that the roll length runs parallel to the construction centerline.

3. Adjacent rolls of reinforcement geogrid shall be overlapped a minimum of 1 foot.

4. Lengths of reinforcement geogrid shall be continuous, splicing along the length will not be allowed.

5. Seams along adjacent lengths of reinforcement geogrid shall be tied together with hog rings or cable ties every 3 to 6 feet.

6. The reinforcement geogrid shall be anchored at each end, and pulled taut, to reduce any considerable slack, as directed by the Resident.

7. Fill shall not be dumped directly onto the Reinforcement Geogrid or Reinforcement Geotextile. It shall be dumped at the edge of Reinforcement Geogrid/Reinforcement Geotextile or on a previous course of fill with a minimum compacted depth of 8 inches.

8. The geogrid shall be covered with fill materials within 14 days of placement to protect against unnecessary exposure.

9. Fill may then be pushed onto the reinforcement geogrid using a track mounted bulldozer. At no time shall construction equipment be allowed directly onto the reinforcement geogrid. Track mounted equipment shall be allowed on previous courses of fill with a minimum compacted depth of 8 inches. Smooth drum roller compaction equipment shall be allowed on previous courses of fill with a minimum compacted depth of 8 inches and spread fill with a minimum depth of 12 inches, loose measure. At no time shall rubber tired or sheeps-foot rollers be allowed onto the reinforced fill. Turning of vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the geogrid. Sudden breaking and sharp turning shall be avoided. Equipment speeds over 15 MPH shall not be allowed.

10. Placement, spreading, and compaction of soil on top of the reinforcement geogrid shall advance from one end of the reinforcement geogrid and move towards the other. Care shall be taken to minimize the development of wrinkles and to ensure that the reinforcement geogrid doesn't move from its position during fill placement. Limited stacking may the permitted, as directed by the Resident.

11. Fill shall be compacted as specified in (1) the Standard Specifications or (2) to at least 90 percent of the maximum dry density determined in accordance with AASHTO T-180, whichever is greater. Density testing shall be made at a minimum frequency of one (1) test per lift or as otherwise specified in the Standard Specifications.

12. During construction the surface of the fill shall be kept approximately horizontal. Fill shall be graded away from the slope crest and rolled at the end of each work day to prevent ponding of water on the surface of the reinforced soil mass.

13. Any geogrid damage shall be repaired or replaced in accordance with the manufacturer's recommendations. The Contractor shall replace any geogrid damaged during installation at no additional cost to the Department.

14. Rutting may develop within the initial granular lift but rut depths should not exceed 3 inches. It may be necessary to decrease the size and/or weight of the construction equipment or increase the thickness of the granular lift if rut depths of 3 inches or less cannot be maintained.

15. All rutting formed during construction shall be filled with new base material. In no case shall rutting be filled by blading down.

Method of Measurement Reinforcement Geogrid measurement will be by the square yard of material installed. Incidental overlaps for connections, splices, etc. are not included in the pay item.

Basis of Payment Reinforcement geogrid placement will be paid for at the Contract unit price per square yard which shall be full compensation for all off-loading, inspection, storage, labor, materials, equipment, tools and any incidentals to complete the installation.

Pay Item	Description	Pay Unit
620.65	Reinforcement Geogrid	Square Yard