MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

PRINCETON BRIDGE
ROUTE 1 OVER GRAND FALLS FLOWAGE
PRINCETON, MAINE

Prepared by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Reviewed by:
Michael J. Moreau, P.E.
Geotechnical Engineer

Washington County
PIN 12662.00

Soils Report No. 2011-01
Bridge No. 2688

Fed No. AC-BH-1266(200)X
January 12, 2011

Table of Contents

Tables

Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Princeton Bridge which carries State Route 1 over Grand Falls Flowage located on the town line of Princeton and Indian Township, Maine. The bridge is a two span, painted steel structure and a total length of 137 feet. The proposed replacement bridge will be a 136-foot span, simply supported, precast, prestressed concrete butted box beam superstructure. The proposed abutments and return wingwalls are full height, cantilever-type walls on spread footings founded directly on bedrock or seals cast on bedrock. The proposed pier is a mass pier on a spread footing founded on seal concrete cast directly on bedrock. The following design recommendations are discussed in detail in this report:

Spread Footings Foundations -General - The proposed abutment and mass pier foundations shall be supported on spread footings founded on bedrock. The abutment borings indicate that bedrock with Rock Quality Designations (RQD) corresponding to rock that is very highly fractured to slightly fractured, will be encountered at the bedrock surface, therefore, the bedrock surface shall be cleared of all loose bedrock and loose, decomposed bedrock. RQD values at the Pier borings correlated to very highly fractured and very poor quality bedrock. Bedrock subgrade preparation at the Pier may require more extensive removal of highly fractured portions of bedrock.

Cantilever-type Abutments and Wingwalls - Abutments and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. They shall be designed for all relevant strength, service and extreme limit states in accordance with AASHTO LRFD Bridge Design Specifications, Fifth Edition, 2010, (herein referred to as LRFD).

The design of project abutments founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. A maximum frictional coefficient of 0.70 at the bedrock-concrete interface should be assumed.

For abutment and wingwall footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8) of the footing dimensions, in either direction.

The overall global stability of foundations are typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever wingwalls. The Designer may assume soil properties for the structural backfill of ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of surcharge loads is permitted.

The contractor should maintain the abutment and wingwall excavations so that the foundations can be constructed in the dry. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil.

Mass Pier – Strength and extreme limit state design of the mass pier foundation shall consider bearing resistance, eccentricity (overturning), failure by sliding and structural failure. Extreme event load combinations are those relating to ice load, vessel collision, and certain hydraulic events. Service limit state design checks shall be used to assess pier footing settlement, horizontal movement, bearing resistance, sliding and eccentricity.

For pier footings or concrete seals on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions, in either direction.

For sliding analyses at the strength limit state, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of piers founded on spread footings on bedrock. Sliding computations for resistance of the pier footing to lateral loads shall assume a maximum frictional coefficient of 0.60 at the bedrock-concrete interface.

The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the pier foundation, and therefore a global stability evaluation may be waived.

Test borings drilled at the pier location indicate that very highly fractured and very poor quality bedrock will be encountered at the bedrock surface. Therefore, bedrock subgrade preparation at the pier may require excavation and removal of up to approximately 2 to 4 feet of very highly fractured rock in some areas, to approximate Elevation 188.0 feet.

Site conditions may warrant that the nose of the pier be designed to effectively break up or deflect floating ice or debris. Facing the nose with a steel plate/angle or facing the pier with granite should be considered.

Factored Bearing Resistance – The factored bearing pressure at the strength limit state for abutment spread footings on sound bedrock should not exceed the factored bearing resistance of 20 kips per square foot (ksf). Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used when analyzing the service limit state and for preliminary footing sizing and to control settlement.

The factored bearing pressure at the strength limit state for pier spread footings on prepared bedrock should not exceed the factored bearing resistance of 10 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used when analyzing the service limit state and for preliminary footing sizing and to control settlement.

No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Scour and Riprap - For the scour protection of abutment, pier and wingwall footings, place the bottom of seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible/scourable rock. Bridge approach slopes should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1-foot thick layer of bedding material.

Settlement - The existing approach embankments at both bridge approaches will be raised with up to 1 to 2 feet of additional fill and will result in negligible densification of the underlying soils. Post-construction settlement will be minimal. Any settlement of bridge abutments will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

Frost Protection - Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock. Any foundations placed on granular soils should be founded a minimum of 6.5 feet below finished exterior grade for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

Seismic Design Considerations – Seismic analysis is not required for multiple-span bridges in Seismic Zone 1, however superstructure connections and bridge seat dimensions shall be designed in accordance with LRFD requirements.

Construction Considerations –
Excavation.

- Cofferdams and temporary lateral earth support systems will be required to permit abutment, wingwall and pier construction.
- Remove old abutments, wingwalls and pier in their entirety.
- Preparation of the bedrock subgrade for all foundations may require excavation of bedrock to create level benches or flatten bedrock surfaces with slopes steeper than 4 horizontal to 1 vertical (4H:1V). All loose bedrock and soil debris should be removed from bearing surfaces and the final bedrock surface washed with high-pressure water and air before concrete is placed for the abutment and wingwall foundations.
- Excavation of bedrock may be conducted using conventional equipment, but may require drilling and blasting methods.

Blasting.

- Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre- and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

Dewatering.

- Control groundwater and surface water infiltration to permit construction in the dry at abutments and wingwalls.

Exposed Natives Soils

- Do not use excavated existing fill or glacial till soils for fill anywhere beneath the new pavement structure, dressing slopes or for new backfill. Use these soils to dress slopes only below the bottom elevation of the shoulder subbase gravel.
- Glacial till is generally considered moisture-sensitive due to the high fines content. If encountered, the soil is susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the contractor should remove and replace the disturbed materials and replace with compacted granular borrow.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Princeton Bridge which carries State Route 1 over Grand Falls Flowage, between Princeton and Indian Township, Maine. This report presents the soils information obtained at the site during the subsurface investigations, foundation recommendations and geotechnical design parameters for bridge replacement.

Princeton Bridge was built in 1939 and is a 137-foot, 2-span, painted steel girder bridge. The superstructure is supported on concrete gravity abutments and a mass concrete pier. The pier and abutments are founded on spread footings bearing on bedrock.

Year 2009 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports assign the substructures a condition rating of 5 – fair, and indicate a Bridge Sufficiency Rating of 49. The bridge is considered to be in fair condition and in need of complete replacement due to insufficient bridge width and deterioration of the abutments and the river pier.

The MaineDOT Bridge Program identified the preferred bridge structure alternative to be a 136-foot, two-span, precast prestressed concrete box beam superstructure, with foundations consisting of cantilever-type abutments and a mass pier. All proposed foundations consist of spread footings founded directly on bedrock or on seal concrete cast on bedrock. The superstructure curb-to-curb width will be increased from 25 feet to 31 feet and will be centered on the existing alignment.

2.0 GEOLOGIC SETTING

Princeton Bridge on State Route 1 on the town line of Princeton and Indian Township, Maine, crosses Grand Falls Flowage as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey (MGS) Surficial Geology of Big Lake Quadrangle, Maine, Open-file No. 86-61 (1986) indicates that the surficial soil unit at the Princeton Bridge site is glacial till.

Glacial till is a heterogeneous mixture of sand, silt, clay and stones, and includes two varieties: basal till and ablation till. Basal till is fine grained and very compact, often bonded or cemented. Ablation till is less dense, at times loose, and sandy and stoney. The till unit generally overlies bedrock, and was deposited directly by glacial ice. Till deposits typically conform to the bedrock surface, and were deposited directly by the glacial ice.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the Princeton Bridge site as the Flume Ridge Formation and describes the Flume Ridge Formation as consisting of calcareous sandstone, interbedded sandstone and impure limestone. Bedrock cores obtained during the project subsurface investigation consist of interbedded calcareous and non-calcareous metamorphic Siltstones and non-calcareous Slates.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling nine (9) test borings. Six (6) of the nine (9) borings were advanced to bedrock and were terminated with bedrock cores. Test borings BB-PIT-101 and BB-PIT-201 were drilled at the east and west quadrants of the proposed Abutment No. 1, respectively Test borings BB-PIT-102 and BB-PIT-203 were drilled at the location of the proposed Pier. Test borings BB-PIT-103, BB-PIT-103A, BB-PIT-103B, BB-PIT-103C and BB-PIT-202 were drilled at the location of proposed Abutment No. 2. The preliminary, Series-100 borings were drilled on October 13 and 14, 2004 by Maine Test Boring (MTB), Inc. of Brewer, Maine. The three 200-Series borings were drilled to determine approximate bedrock elevations at the westerly portions of Abutments No. 1 and No. 2 and the Pier. The 200-Series borings were drilled on October 28, 2010 using the MaineDOT drill rig. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were typically obtained 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 two drill rigs used at the site were equipped with either a rope and cathead or an automatic hammer to drive the split spoon. The MTB rope and cathead hammer used to complete the 2004 borings is considered to deliver 60 percent of its total theoretical energy; therefore the N-values presented on those boring logs do not require correction. The MaineDOT drill rig is equipped with a Central Mine Equipment (CME) automatic hammer. The hammer was calibrated by MaineDOT in February of 2009 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. The N-values presented for borings drilled with the MaineDOT hammer are corrected values computed by applying average energy transfer factors of 0.84 to the raw field N-values. The hammer efficiency factor of 0.84 and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in five (5) borings using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Subsurface Inspector certified by the New England Transportation Technician Certification Program (NETTCP) logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling programs.

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 3 and 4 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site.

Laboratory testing consisted of three (3) standard grain size analyses, two (2) grain size analyses with hydrometer, and five (5) natural water content tests. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs, on Sheet 3- Boring Logs and on Sheet 4 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at all of the test borings generally consisted of granular fill, reworked glacial till and weathered bedrock, all underlain by metasedimentary bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. The boring logs are provided in Appendix A – Boring Logs and on Sheets 3 and 4 – Boring Logs. A brief summary description of the strata encountered follows:

5.1 Fill

A layer of fill was encountered in the seven (7) test borings drilled in the fill extensions directly behind the bridge abutments. The encountered fill layer is approximately 11 to 15 feet thick. The fill soils generally consisted of: light brown, damp, silty, sand, some gravel; brown or grey, damp to wet, sand, some gravel, little to some silt, trace clay; dark brown, wet, sand, little gravel, trace silt and asphalt, with petroleum odor; brown, dry to damp, gravelly sand, some to trace silt, trace wood fragments; or brown, damp to wet, sandy gravel, trace silt, with occasional cobbles.

Three solid stem auger explorations drilled in the fill extensions leading to the westerly abutment refused on cobbles or boulders encountered at depths of 3.7 to 8.1 feet below ground surface (bgs).

Corrected SPT N-values in fill unit ranged from 10 to 48 blows per foot (bpf) indicating that the fill is loose to dense in consistency.

Four (4) grain size analysis resulted in the fill soils being classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SM and SC-SM under the Unified Soil Classification System (USCS). The measured water contents of the samples tested ranged from approximately 5 to 31 percent.

5.2 Reworked Glacial Till

A shallow and discontinuous layer of reworked glacial till soils was encountered in borings BB-PIT-101 and BB-PIT-103C. The encountered thickness ranged from approximately 2.4 to 2.9 feet thick at the boring locations. The reworked soils consisted of brown and grey, damp to moist, sand, some to trace gravel, some silt, trace clay.

Corrected SPT N-values in unit were > 50 bpf, indicating a soil of very dense consistency.

Laboratory testing of samples of the deposit indicates USCS soil classifications of CL-ML and SC-SM. The AASHTO classifications for the samples tested are A-4 and A-2-4. The measured water contents of the tested samples ranged from approximately 19 to 31 percent.

5.3 Bedrock

Below the north and south bridge approach fills, bedrock was encountered and cored at depths ranging from approximately 17.4 feet bgs and approximate Elevation 192.1 feet in boring BB-PIT-101 to a depth of approximately 13.1 feet bgs and approximate Elevation 196.40 feet in boring BB-PIT-201. In the river channel, bedrock was encountered at depths of approximately 2.6 to 7.3 feet below the streambed, corresponding to Elevation 190.6 feet to 192.4 feet in boring BB-PIT-102 and BB-PIT-203. In the river channel, the test borings encountered approximately 2.7 to 3.9 feet of very highly fractured bedrock overlying more intact bedrock.

The bedrock at the site is identified as grey, fine grained, metamorphic Siltstone with interbeds of Slate, hard, slightly weathered to fresh, steeply dipping to irregular foliation in the Slate, close bedding, surfaces fresh; Siltstone beds were generally massive. The RQD of the bedrock was determined to range from 0 to 92 percent, correlating to a Rock Mass Quality of ‘very poor’ to ‘excellent’.

Table 1 below summarizes approximate top of bedrock elevations at the proposed bridge abutments and pier:

Proposed Substructure	Boring	Station	Offset	Approx. Depth to Bedrock (feet)	**Approx. Elevation of Bedrock Surface (feet)**
Abutment No. 1 (Right)	BB-PIT-101	17+80.3	14.1 Rt.	17.4	**192.1**
Abutment No. 1 (Left)	BB-PIT-201	17+85	8.0 Lt	13.1	**196.4**
Pier (Right)	BB-PIT-102	18+53.3	9.0 Rt.	2.6	**192.4**
Pier (Left)	BB-PIT-203	18+63	7.5 Lt.	7.3	**190.6**
Abutment No. 2 (Right)	BB-PIT-103C	19+43.2	10.0 Rt	13.9	**196.4**
Abutment No. 2 (Left)	BB-PIT-202	19+40	8.0 Lt	14.8	**195.2**

Table 1. Summary of Approximate Bedrock Elevations

5.4 Groundwater

The groundwater levels observed in three borings drilled in the bridge approach fills ranged from approximately 5 to 9.5 feet bgs. Groundwater levels will fluctuate with precipitation, seasonal changes, runoff, and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

Our assessment of subsurface conditions at the site indicate the most effective foundation type for this site to be cantilever-type abutments, wingwalls and mass piers on spread footings founded directly on bedrock or on seals constructed on bedrock. Design recommendations for these foundation alternatives are discussed in detail in Section 7.0 - Geotechnical Design Recommendations.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 General - Spread Footings on Bedrock

Bedrock was encountered at depths approximately 13 to 17 feet below the existing roadway surface at the proposed Abutment No. 1 and Abutment No. 2 and approximately 3 to 8 feet below the riverbed at the proposed Pier. It is therefore considered feasible that spread footings, or seals if required, could be practically and economically constructed to bear on bedrock within moderately shallow excavations requiring cofferdams and temporary soil support systems.

The abutment borings indicate that suitable bedrock with an average RQD of approximately 40 percent will be encountered at the bedrock surface, however, the bedrock surface shall be cleared of all loose bedrock and highly fractured bedrock. The Pier borings indicate that bedrock with an RQD of 0 percent, which correlates to very highly fractured and very poor quality bedrock, will be encountered at the bedrock surface. Therefore, bedrock subgrade preparation at the Pier may require more extensive removal (approximately 2 to 4 feet) of highly fractured portions of bedrock that might be loose.

Based on borings conducted at the site, top of bedrock elevations encountered in those borings and potential for rock excavation, the approximate bottom of footing (BOF) or bottom of seal elevations are estimated to be:

- Elevation 192 to 196 feet at Abutment No. 1,
- Elevation 195 to196 feet at Abutment No. 2 and
- Elevation 188 to 190 feet at the Pier.

7.2 Abutment and Wingwall Design

Abutments and wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, extreme and service limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity (overturning),
- failure by sliding
- reinforced concrete structural failure.

For the scour protection of abutment and wingwall footings, construct footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible rock. As such, strength and extreme event limit state designs do not need to consider foundation resistance after the design or check floods for scour.

Extreme limit state design checks for abutments shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load combinations relating to certain hydraulic events and ice (if warranted by ice history or stream constriction by the abutment). Resistance factors, φ, for the extreme event limit state shall be taken as 1.0.

For the service limit state, a resistance factor, φ,of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of foundations are typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

For footings or concrete seals on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions, in either direction. This eccentricity corresponds to the resultant of reaction forces falling within the middle three-fourths (3/4) of the footing.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

Anchorage of footings to seals or of seals to bedrock may be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Cantilever-type abutments should be designed for active earth pressure over the abutment height. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.31 is recommended. Earth loads for wingwalls shall also be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory.

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

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 2 below:

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

Table 2. Equivalent Height of Soil for Estimating Live Load Surcharge

Abutment and wingwall designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the 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 above the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 Mass Pier Foundation

Mass pier foundations shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, extreme and service limit states. The design of mass piers supported on spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity (overturning),
- failure by sliding
- reinforced concrete structural failure.

For scour protection of the pier, construct the seal and footing directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible rock. As such, strength and extreme event limit state designs do not need to consider foundation resistance after scour due to the design and check floods for scour.

A modified Strength Limit State analysis should be performed that includes the ice pressures specified in BDG Section 3.9 – Ice Loads.

Extreme limit state design checks for piers shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load combinations related to ice loads, vessel collision and certain hydraulic events. Resistance factors, φ, for the extreme event limit state shall be taken as 1.0. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For the service limit state, a resistance factor (φ)of 1.0 shall be used to assess spread footing design for: settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of foundations are typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

For pier footings or concrete seals on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions, in either direction. This corresponds to the resultant of the reaction forces falling within the middle three-fourths (3/4) of the footing dimensions.

For sliding analyses at the strength limit state, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of piers founded on spread footings on bedrock. Sliding computations for resistance of the pier footing to lateral loads shall assume a maximum frictional coefficient of 0.60 at the bedrock-concrete interface.

Anchorage of the pier footing to seals or of the seal to bedrock may be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Design parameters for the design of pier footings for bearing resistance are provided in Section 7.3, above.

It is recommended that the proposed center pier foundations be a spread footing supported on bedrock with a minimum RQD of approximately 30 percent. Based on the test borings drilled at the proposed center pier, bedrock meeting this requirement will be encountered with excavation of up to approximately 2 to 4 feet of very highly fractured rock, to approximate Elevation 188.0 feet

Site conditions may warrant that the nose of the pier be designed to effectively break up or deflect floating ice or debris. Facing the nose with a steel plate/angle or facing the pier with granite should be considered.

7.4 Bearing Resistance

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2.

The bearing resistance for ***abutment and wingwall footings*** founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 20 ksf. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 16 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination.

The bearing resistance for the ***pier footing*** founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 10 ksf. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of

16 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination.

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

7.5 Scour and Riprap

For the scour protection of abutment, pier and wingwall footings, place the bottom of seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible rock.

Bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Stone riprap shall conform to item number 703.26 Plain and Hand Laid Riprap of the Standard Specification and 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 or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material conforming to item number 703.19, of the Standard Specification. Riprap may be placed at the toes of abutments, wingwalls and retaining walls, as required.

7.6 Settlement

The existing approach embankments at both bridge approaches will be raised with up to 1 to 2 feet of additional fill. Placing 2 feet of earth fill over approximately 20 feet of granular fill soils will result in negligible densification of the underlying soils and subsequent settlement of the embankments. Any settlement will occur during and immediately after construction of the embankments. Post-construction settlement will be minimal.

Any settlement of bridge abutments will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

7.7 Frost Protection

We recommend that project spread footings for abutment and wingwalls be constructed to bear directly on bedrock. Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock.

Any foundations placed on granular fill should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Princeton has a design freezing index of approximately 1600 F-degree days. An assumed water content of 10% was used for granular soils above the water table. These components correlate to a frost depth of 7.0 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Princeton was assigned a design freezing index of approximately 1588 F-degree days. An assumed water content of 10% was used for granular soils above the water table. These components correlate to a frost depth of 6.5 feet. We recommend that foundations constructed within granular fill soils be founded a minimum of 6.5 feet below finished exterior grade for frost protection.

7.8 Seismic Design Considerations

In conformance with LRFD Table 4.7.4.3.1-1, seismic analysis is not required for multiple-span bridges in Seismic Zone 1. While Princeton Bridge is not on the National Highway System, and is therefore not classified as functionally important. Furthermore, the bridge is not classified as a major structure, since the bridge construction costs will not exceed $10 million. These criteria eliminate the BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and bridge seat dimensions shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak ground acceleration coefficient (PGA) = 0.081g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.260g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.104g
- Site Class D (based on an average shear wave velocity (v_s), between 600 ft/s and 1,200 ft/sec, for the upper 100 ft of the soil profile)
- Seismic Zone 1, based on a $S_{D1} < 0.15g$

7.9 Construction Considerations

Construction activities will include construction of cofferdams and earth support systems to support the approach fills and control stream flow during construction of seals and footings for abutments, wingwalls and piers. Construction activities will also include common earth and rock excavation.

Glacial till is generally considered moisture-sensitive due to the high fines content. If encountered, the soil is susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the contractor should remove and replace the disturbed materials and replace with compacted granular borrow.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The final bearing surface shall be solid. The bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it shall be benched in level steps or excavated to be completely level. Anchoring, doweling or other means of improving sliding resistance may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction.

The contractor should maintain the abutment and wingwall excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete. It is anticipated that the pier foundation will not be constructed in the dry, therefore, the condition of the bedrock surface prior to placing tremie seal concrete should be inspected with the use of remote underwater cameras or tactile methods. The pier foundation subgrade should be confirmed to be relatively level or the surface should be benched to create a near level, stepped subgrade for the seal placement.

Where foundations are constructed in the dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. In the dry or underwater excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

7.0 CLOSURE

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

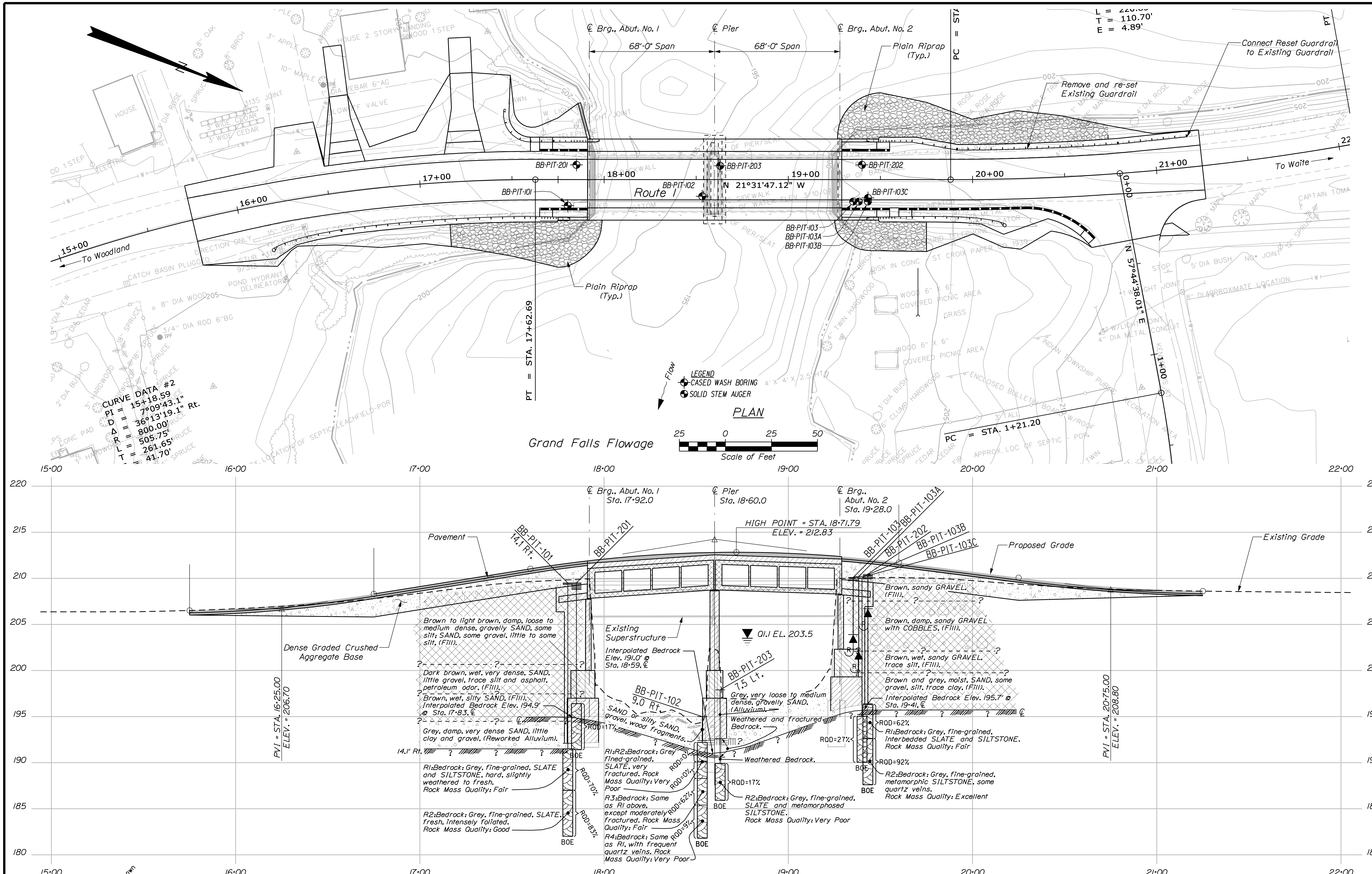
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map

Map Generated on Wednesday, January 12, 2011 10:04:47 AM **Map Scale 1:24000**

Filename: ...\GEOTECH\MSTA\006_BLP&SP1.dgn   Division: GEOTECH   Username: terry.white   Date:1/12/2011



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

Top of Intact Bedrock

Weathered and or Fractured Bedrock

PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

Notes: 1. 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.

2. Soil and bedrock statigraphy illustrated only at BB-PIT-101, BB-PIT-102, BB-PIT-203 and BB-PIT-103C for clarity. Refer to Boring Log Sheets for specific subsurface conditions encountered at all borings.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BH-1266(200)X

PIN 12662.00

BRIDGE PLANS

BRIDGE NO. 2688

	BY	DATE
PROJ. MANAGER	D. ANDERSON	
DESIGN-DETAILED	L. KRUSINSKI	
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

PRINCETON BRIDGE
GRAND FALLS FLOWAGE
PRINCETON WASHINGTON COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 4

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:1/12/2011

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-101
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	209.5	Auger ID/OD:	5" SSA
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/13/04-10/13/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	17+80.3, 14.1 ft Rt.	Casing ID/OD:	HW	Water Level*:	9.0 ft bgs

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
					SSA
1D	24/7	5.00 - 7.00	25/24/24/21	48	
2D	16/12	10.00 - 11.33	54/87/75(4")	---	143
					114
					26
					21
					47
3D	24/13	15.00 - 17.00	32/44/43/33	87	71
					77
					a100
R1	54/52	18.00 - 22.50	RQD = 70%		NQ
R2	60/60	22.50 - 27.50	RQD = 83%		

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

209.35 — PAVEMENT. — 0.15
Brown, damp, dense, SAND, some gravel, little silt, (Fill). — Laboratory: G#182506 A-1-b, SM WC=5.1%
200.50 — 9.00 — Dark brown, wet, medium to coarse SAND, little gravel, trace silt, old pavement, (Fill), (Strong Petroleum Odor).
Brown, wet, silty fine to coarse SAND.
194.50 — 15.00 — Grey, damp, very dense, SAND, some silt, little clay, little gravel, (Reworked Till). — Laboratory: G#182505 A-4, CL-ML WC=9.3%
a100 blows for 0.4'.
192.10 — 17.40 — Top of Bedrock at Elev. 192.1'.
Washed ahead to 18.0' bgs.
R1:Bedrock: Grey, fine grained, SLATE and SILTSTONE, hard, slightly weathered to fresh, joint set along bedding at steep to vertical angles, unweathered surfaces. Flume Ridge Formation. Rock Mass Quality: Fair.
R1:Core Times (min:sec)
18.0-19.0' (2:30)
19.0-20.0' (2:40)
20.0-21.0' (2:45)
21.0-22.0' (2:38)
22.0-22.5' (1:50) 96% Recovery
R2: Bedrock: Grey, fine-grained, SLATE, fresh, intensely foliated. Joint sets along foliation/bedding and horizontal. Flume Ridge Formation. Rock Mass Quality: Good.
R2:Core Times (min:sec)
22.5-23.5' (2:52)
23.5-24.5' (2:50)
24.5-25.5' (3:08)
25.5-26.5' (3:08)
26.5-27.5' (3:15) 100% Recovery
182.00 — 27.50 — Bottom of Exploration at 27.50 feet below ground surface.

Remarks:
Existing bridge deck elevation at Abutment 1.
Boring locations pointed on pavement for possible future survey.

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

Page 1 of 1 — Boring No.: BB-PIT-101

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-102
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	195.0	Auger ID/OD:	N/A
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/14/04-10/14/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	18+53.3, 9.0 ft Rt.	Casing ID/OD:	HW	Water Level*:	At ground surface

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
1D/AB	24/4	0.00 - 2.00	24/41/32/13	73	22
					33
					a35 NQ
R1	25.2/21.8	2.80 - 4.90	RQD = 0%		
R2	21.6/21.6	4.90 - 6.70	RQD = 0%		
R3	34.8/31	6.70 - 9.60	RQD = 62%		
R4	43.2/42	9.60 - 13.20	RQD = 9%		

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

(1D/A) 0.0-1.5' bgs.
Brown, wet, fine to coarse SAND, wood, rock fragments, trace gravel. Density probably loose due to higher blow counts caused by wood.
193.50 — 1.50 — (1D/B) 1.5-2.0' bgs.
Brown, wet, silty fine to coarse SAND, trace gravel.
a35 blows for 0.6'.
192.40 — 2.60 — Top of Bedrock at Elev. 192.4'.
Roller coned ahead from 2.0-2.8' bgs.
R1:Bedrock: Grey, fine grained, SLATE, hard, slightly weathered, very fractured along steeply dipping bedding, surfaces fresh with no residue. Flume Ridge Formation. Rock Mass Quality: Very poor.
R1:Core Times (min:sec)
2.8-3.8' (3:17)
3.8-4.8' (2:55)
4.8-4.9' (0:45) 81% Recovery
R2:Bedrock: Same as R1, except highly fractured. Rock Mass Quality: Very Poor.
R2:Core Times (min:sec)
4.9-5.9' (2:50)
5.9-6.7' (2:38) 100% Recovery
R3:Bedrock: Same as R1, except moderately fractured. Rock Mass Quality: Fair.
R3:Core Times (min:sec)
6.7-7.7' (2:03)
7.7-8.7' (3:11)
8.7-9.6' (3:05) 91% Recovery
R4:Bedrock: Same as R1, with frequent quartz veins. Rock Mass Quality: Very Poor.
R4:Core Times (min:sec)
9.6-10.6' (2:35)
10.6-11.6' (2:50)
11.6-12.6' (3:20)
12.6-13.2' (2:10) 98% Recovery
181.80 — 13.20 — Bottom of Exploration at 13.20 feet below ground surface.

Remarks:
Concrete Deck 0.85' thick.
Top of Deck to water 8.7'.
Top of Deck to ground surface 16.4'.

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

Page 1 of 1 — Boring No.: BB-PIT-102

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-103
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	210.1	Auger ID/OD:	5"
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/13/04-10/13/04	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	19+35.2, 12.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
					SSA
1D	14.4/4	5.00 - 6.20	11/19/50(2.4")	---	

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

209.75 — PAVEMENT. — 0.35
Brown, damp, sandy GRAVEL, trace silt, (Fill).
207.60 — 2.50 — Similar to above, but with cobbles, (Fill).
203.90 — 6.20 — Bottom of Exploration at 6.20 feet below ground surface.
AUGER REFUSAL

Remarks:
1. Existing Bridge Deck elevation at Abutment 2.
2. Soil Descriptions based on visual observations and drill attitude.

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

Page 1 of 1 — Boring No.: BB-PIT-103

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-103A
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	210.2	Auger ID/OD:	5"
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	N/A
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/13/04-10/13/04	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	19+38.2, 12.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
					SSA

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

209.85 — PAVEMENT. — 0.35
Brown, damp, sandy GRAVEL, trace silt, (Fill).
207.70 — 2.50 — Similar to above, but with cobbles, (Fill).
202.10 — 8.10 — Bottom of Exploration at 8.10 feet below ground surface.
REFUSAL on Boulder

Remarks:
1. xisting Bridge Deck elevation at Abutment 2.
2. Soil descriptions based on visual observations and drill attitude.

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

Page 1 of 1 — Boring No.: BB-PIT-103A

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-103B
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	210.4	Auger ID/OD:	5"
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	N/A
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/13/04-10/13/04	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	19+43.2, 12.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
					SSA

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

210.05 — PAVEMENT. — 0.35
Brown, damp, sandy GRAVEL, trace silt, (Fill).
208.90 — 1.50 — Similar to above, but with cobbles, (Fill).
206.70 — 3.70 — Bottom of Exploration at 3.70 feet below ground surface.
REFUSAL on Boulder

Remarks:
1. Existing Bridge Deck elevation at Abutment 2.
2. Soil descriptions based on visual observations and drill attitude.

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

Page 1 of 1 — Boring No.: BB-PIT-103B

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine
Boring No.: BB-PIT-103C
PIN: 12662.00

Driller:	Maine Testboring, Inc.	Elevation (ft.)	210.3	Auger ID/OD:	5"
Operator:	B. Enos/C. Wormley	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	Mobile B-47 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/13/04-10/14/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	19+43.2, 10.0 ft Rt.	Casing ID/OD:	HW	Water Level*:	8.0 ft bgs

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-value	Casing Blows
					SSA
1D/AB	24/13	10.00 - 12.00	28/67/26/43	93	68
					90
					130
					a200
R1	44.4/43	14.00 - 17.70	RQD = 62%		NQ
R2	60/56	17.70 - 22.70	RQD = 92%		

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

209.95 — PAVEMENT. — 0.35
Brown, damp, sandy GRAVEL, trace silt, (Fill).
207.90 — 2.40 — Similar to above, but with cobbles, (Fill).
202.30 — 8.00 — Brown, wet, sandy GRAVEL, trace silt, (Fill).
(1D/A) 10.0-11.0' bgs.
199.30 — 11.00 — (1D/B) 11.0-12.0' bgs.
Brown and grey, moist, SAND, some gravel, some silt, trace clay, (Reworked Till). — Laboratory: G#182507 A-2-4, SC-SM WC=31.3%
a200 blows for 0.9'.
196.40 — 13.90 — Top of Bedrock at Elev. 196.4'.
Roller coned ahead fron 13.9-14.0' bgs.
196.30 — 14.00 — R1:Bedrock: Grey, fine grained, interbedded metamorphic SILTSTONE and SLATE, moderately hard to hard, slightly weathered in the upper 16 inches to fresh, highly foliated. Flume Ridge Formation. Rock Mass Quality: Fair.
R1: Core Times (min:sec)
14.0-15.0' (3:05)
15.0-16.0' (2:38)
16.0-17.0' (2:30)
17.0-17.7' (2:18) 96% Recovery
R2:Bedrock: Grey, fine-grained, Metamorphic SILTSTONE, fresh, some quartz veins. Flume Ridge Formation. Rock Mass Quality: Excellent
R2:Core Times (min:sec)
17.7-18.7' (2:15)
18.7-19.7' (2:32)
19.7-20.7' (2:40)
20.7-21.7' (2:55)
21.7-22.7' (3:07) 93% Recovery
187.60 — 22.70 — Bottom of Exploration at 22.70 feet below ground surface.

Remarks:
Existing Bridge Deck elevation at Abutment 2.

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

Page 1 of 1 — Boring No.: BB-PIT-103C

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-BH-1266(200)X
BRIDGE PLANS
PIN 12662.00
BRIDGE NO. 2688

		BY	DATE
PROJ. MANAGER	D. ANDERSON		
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

PRINCETON BRIDGE
GRAND FALLS FLOWAGE
PRINCETON WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
3
OF 4

Filename: ...\msta\008_BORING LOGS2.dgn   Division: GEOTECH   Username: terry.white   Date:1/12/2011

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-201
PIN: 12662.00

Driller:	MaineDOT	Elevation (ft.)	209.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/28/10; 07:30-10:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	17+85, 8.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	5.0 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	208.80	PAVEMENT. 0.70	
	1D	24/12	1.00 - 3.00	10/7/6/5	13	18			Brown, damp, medium dense, gravelly, fine to coarse SAND, some silt. (Fill)	
								206.50	3.00	
5	2D	24/17	5.00 - 7.00	4/3/4/6	7	10			Light brown, damp, loose, fine to coarse SAND, some gravel, some silt. (Fill).	G#237518 A-1-b, SM WC=9.5%
								201.50	8.00	
10	3D	24/15	10.00 - 12.00	5/7/6/12	13	18	50		Grey, saturated, medium dense, gravelly, fine to coarse SAND, trace silt, trace wood fragments; gravel is fine to coarse rounded to coarse angular granite rock fragments. (Fill)	
							53			
							68			
	R1	60/60	13.10 - 18.10	RQD = 17%			a30 NQ-2	196.40	a30 blows for 0.1 ft. 13.10 Top of Bedrock at Elev. 196.4 ft. R1:Bedrock: Grey, fine grained, interbedded SILTSTONE and SLATE, hard, fresh, thinly laminated in slate to massive siltstone beds, two joint sets, one at 45 degrees along bedding, 2nd vertical. Flume Ridge Formation. Rock Mass Quality: Very Poor. R1:Core Times (min:sec) 13.1-14.1 ft (7:00) 14.1-15.1 ft (4:00) 15.1-16.1 ft (3:45) 16.1-17.1 ft (4:10) No Water Return 17.1-18.1 ft (4:45) " " " 100% Recovery	
15										
								191.40	18.10 Bottom of Exploration at 18.10 feet below ground surface.	
20										
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-PIT-201

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-203
PIN: 12662.00

Driller:	MaineDOT	Elevation (ft.)	197.9	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/28/10; 13:00-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	18+63, 7.5 ft Lt.	Casing ID/OD:	NW	Water Level*:	Water Boring
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	4/1/1/7	2	3	SPUN CASE		Grey, saturated, very loose, gravelly, fine to coarse SAND, trace silt. (Reworked Alluvium).	
	2D	24/12	2.00 - 4.00	5/10/13/7	23	32			Grey, saturated, medium dense, gravelly coarse Sand, little pockets of dark brown/grey silty fine sand, odor of "cut back asphalt"; gravel is slate fragments.	
5	R1	32.4/ 32.4	5.30 - 8.00	RQD = 0%			NQ-2	192.60	5.30 R1: Weathered bedrock fragments, trace sand and silt. Cobbles or Fractured Bedrock, 5.3-7.3 ft bgs. R1:Core Times (min:sec) 5.3-6.3 ft (2:20) 6.3-7.3 ft (6:45)	
								190.60	7.30 R1:Weathered BEDROCK. 7.3-8.0 ft (9:75) 100% Recovery Core Blocked	
	R2	48/48	8.00 - 12.00	RQD = 17%				189.90	8.00 Top of Intack Bedrock at Elev. 189.9 ft. R2:Bedrock: Grey, fine grained, moderately hard, SLATE, fractured along bedding at 1/4 to 1/2 inch spacing, silt covered surfaces, changing to grey, fine grained, hard, metamorphosed Siltstone, occassional calcite veins, slighty weathered, massive. Flume Ridge Formation. Rock Mass Quality: Very Poor. R2:Core Times (min:sec) 8.0-9.0 ft (3:45) No Water Return 9.0-10.0 ft (6:25) " " " 10.0-11.0 ft (3:25) " " " 11.0-12.0 ft (4:25) " " " 100% Recovery Core Blocked	
10										
								185.90	12.00 Bottom of Exploration at 12.00 feet below ground surface.	
15										
20										
25										

Remarks:

12.2 ft from Bridge Deck to Ground.
1.0 ft Concrete Bridge Deck.

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

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

Page 1 of 1

Boring No.: BB-PIT-203

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

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-202
PIN: 12662.00

Driller:	MaineDOT	Elevation (ft.)	210.0	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/28/10; 10:30-13:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	19+40, 8.0 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	9.5 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	209.30	PAVEMENT. 0.70	
	1D	8.4/8	1.50 - 2.20	14/50(2.4")	---				Brown, dry, very dense, gravelly, fine to coarse SAND, little silt, occasional cobbles. (Fill). Cobble from 3.4-3.9 ft bgs. Cobble from 4.5-4.8 ft bgs.	
5	2D	24/13	5.00 - 7.00	8/5/6/24	11	15		205.00	5.00 Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt. (Fill). Boulder from 7.2-8.6 ft bgs.	G#237519 A-1-b, SM WC=8.0%
							75			
							37	200.50	9.50	
10	3D	24/5	10.00 - 12.00	6/2/8/9	10	14	18		Telescoped with NW Casing through Boulder at 10.0 ft bgs. Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt; gravel is coarse, broken rock fragments.	
							32			
							34		Wood fibers and pieces in wash water from 12.5-13.5 ft bgs.	
							81			
							a178	195.20	a178 blows for 0.8 ft. 14.80	
15	R1	60/54	15.00 - 20.00	RQD = 27%			NQ-2	195.00	Top of Bedrock at Elev. 195.2 ft. Roller Coned ahead from 14.8-15.0 ft bgs. 15.00 R1:Bedrock: Grey, fine grained, hard, fresh, metamorphic SILTSTONE and SLATE, finely laminated slate to massive siltstone beds, jointing along bedding at moderately dipping angles, second set verical. Flume Ridge Formation. Rock Mass Quality: Poor. R1:Core Times (min:sec) 15.0-16.0 ft (3:15) 16.0-17.0 ft (3:50) No Water Return 17.0-18.0 ft (3:30) " " " 18.0-19.0 ft (3:25) " " " 19.0-20.0 ft (3:15) " " " 90% Recovery	
20								190.00	20.00 Bottom of Exploration at 20.00 feet below ground surface.	
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-PIT-202

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BH-1266(200)X

PIN 12662.00

BRIDGE PLANS

BRIDGE NO. 2688

	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
PROJ. MANAGER	D. ANDERSON				
DESIGN-DETAILED	L. KRUSINSKI				
CHECKED-REVIEWED	T. WHITE				
DESIGN2-DETAILED2					
DESIGN3-DETAILED3					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

PRINCETON BRIDGE
GRAND FALLS FLOWAGE
PRINCETON WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

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: Princeton Bridge #2688 over Grand Falls Flowage Location: Princeton, Maine	Boring No.: BB-PIT-101 PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	209.5	**Auger ID/OD:**	5" SSA
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/13/04-10/13/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+80.3, 14.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0 ft bgs

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	209.35		PAVEMENT. 0.15	
5	1D	24/7	5.00 - 7.00	25/24/24/21	48				Brown, damp, dense, SAND, some gravel, little silt, (Fill).	G#182506 A-1-b, SM WC=5.1%
							200.50		9.00 Dark brown, wet, medium to coarse SAND, little gravel, trace silt, old pavement, (Fill). [Strong Petroleum Odor].	
10	2D	16/12	10.00 - 11.33	54/87/75(4")	---	143				
						114				
						26			Brown, wet, silty fine to coarse SAND.	
						21				
						47				
15	3D	24/13	15.00 - 17.00	32/44/43/33	87	71	194.50		15.00 Grey, damp, very dense, SAND, some silt , little clay, little gravel, (Reworked Till).	G#182505 A-4, CL-ML WC=19.3%
						77				
						a100	192.10		a100 blows for 0.4'. 17.40	
	R1	54/52	18.00 - 22.50	RQD = 70%		NQ			Top of Bedrock at Elev. 192.1'. Washed ahead to 18.0' bgs. R1:Bedrock: Grey, fine grained, SLATE and SILTSTONE, hard, slightly weathered to fresh, joint set along beddirng at steep to vertical angles, unweathered surfaces. Flume Ridge Formation. Rock Mass Quality; Fair. R1:Core Times (min:sec) 18.0-19.0' (2:30) 19.0-20.0' (2:40) 20.0-21.0' (2:45) 21.0-22.0' (2:38) 22.0-22.5' (1:50) 96% Recovery	
20										
	R2	60/60	22.50 - 27.50	RQD = 83%					R2: Bedrock: Grey, fine-grained, SLATE, fresh, intensely foliated. Joint sets along foliation/bedding and horizontal. Flume Ridge Formation. Rock Mass Quality: Good. R2:Core Times (min:sec)	
25										

Remarks:

Existing bridge deck elevation at Abutment 1.
Boring locations painnted on pavement for possible future survey.

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-PIT-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Princeton Bridge #2688 over Grand Falls Flowage Location: Princeton, Maine	Boring No.: BB-PIT-101 PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	209.5	**Auger ID/OD:**	5" SSA
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/13/04-10/13/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+80.3, 14.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0 ft bgs

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25									22.5-23.5' (2:52) 23.5-24.5' (3:05) 24.5-25.5' (3:11) 25.5-26.5' (3:08) 26.5-27.5' (3:15) 100% Recovery	
							182.00		27.50	
									Bottom of Exploration at 27.50 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:

Existing bridge deck elevation at Abutment 1.
Boring locations painnted on pavement for possible future survey.

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:	Princeton Bridge #2688 over Grand Falls Flowage	**Location:**	Princeton, Maine	**Boring No.:**	BB-PIT-102
				PIN:	12662.00
Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	195.0	**Auger ID/OD:**	N/A
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/14/04-10/14/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	18+53.3, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	At ground surface

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)
0	1D/AB	24/4	0.00 - 2.00	24/41/32/13	73	22	
						33	193.50
	R1	25.2/21.8	2.80 - 4.90	RQD = 0%		a35 NQ	192.40
5	R2	21.6/21.6	4.90 - 6.70	RQD = 0%			
	R3	34.8/31	6.70 - 9.60	RQD = 62%			
10	R4	43.2/42	9.60 - 13.20	RQD = 9%			
							181.80
15							
20							
25							

Visual Description and Remarks

(1D/A) 0.0-1.5' bgs.
Brown, wet, fine to coarse SAND, wood, rock fragments, trace gravel. Density probably loose due to higher blow counts caused by wood.
1.50

(1D/B) 1.5-2.0' bgs.
Brown, wet, silty fine to coarse SAND, trace gravel.
a35 blows for 0.6'.
2.60

Top of Bedrock at Elev. 192.4'.
Roller coned ahead from 2.6-2.8' bgs.
R1:Bedrock: Grey, fine grained, SLATE, hard, slightly weathered, very fractured along steeply dipping bedding, surfaces fresh with no residue. Flume Ridge Formation. Rock Mass Quality: Very poor.
R1:Core Times (min:sec)
2.8-3.8' (3:17)
3.8-4.8' (2:55)
4.8-4.9' (0:45) 87% Recovery
R2:Bedrock: Same as R1, except highly fractured. Rock Mass Quality: Very Poor.
R2:Core Times (min:sec)
4.9-5.9' (2:50)
5.9-6.7' (2:38) 100% Recovery
R3:Bedrock: Same as R1, except moderately fractured. Rock Mass Quality: Fair.
R3:Core Times (min:sec)
6.7-7.7' (2:03)
7.7-8.7' (3:11)
8.7-9.6' (3:05) 91% Recovery
R4:Bedrock: Same as R1, with frequent quartz veins. Rock Mass Quality: Very Poor.
R4:Core Times (min:sec)
9.6-10.6' (2:35)
10.6-11.6' (2:50)
11.6-12.6' (3:20)
12.6-13.2' (2:10) 98% Recovery
13.20

Bottom of Exploration at 13.20 feet below ground surface.

Laboratory Testing Results/ AASHTO and Unified Class.

Remarks:

Concrete Deck 0.85' thick.
Top of Deck to water 8.7'.
Top of Deck to ground surface 16.4'.

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: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-103
PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	210.1	**Auger ID/OD:**	5"
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/13/04-10/13/04	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	19+35.2, 12.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	209.75		PAVEMENT. 0.35	
									Brown, damp, sandy GRAVEL, trace silt, (Fill).	
							207.60		2.50	
									Similar to above, but with cobbles, (Fill).	
5	1D	14.4/4	5.00 - 6.20	11/19/50(2.4")	---					
							203.90		6.20	
									Bottom of Exploration at 6.20 feet below ground surface. AUGER REFUSAL	
10										
15										
20										
25										

Remarks:

1. Existing Bridge Deck elevation at Abutment 2.

2. Soil Descriptions based on visual observations and drill attitude.

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-PIT-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-103A
PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	210.2	**Auger ID/OD:**	5"
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	N/A
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/13/04-10/13/04	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	19+38.2, 12.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	209.85		PAVEMENT. 0.35	
									Brown, damp, sandy GRAVEL, trace silt, (Fill).	
							207.70		2.50	
									Similar to above, but with cobbles, (Fill).	
5										
							202.10		8.10	
									Bottom of Exploration at 8.10 feet below ground surface. REFUSAL on Boulder	
10										
15										
20										
25										

Remarks:

1. xisting Bridge Deck elevation at Abutment 2.
2. Soil descriptions based on visual observations and drill attitude.

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-PIT-103A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-103B
PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	210.4	**Auger ID/OD:**	5"
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	N/A
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/13/04-10/13/04	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	19+43.2, 12.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	210.05		PAVEMENT. 0.35	
									Brown, damp, sandy GRAVEL, trace silt, (Fill).	
							208.90		1.50	
									Similar to above, but with cobbles, (Fill).	
							206.70		3.70	
									Bottom of Exploration at 3.70 feet below ground surface. REFUSAL on Boulder	
5										
10										
15										
20										
25										

Remarks:

1. Existing Bridge Deck elevation at Abutment 2.

2. Soil descriptions based on visual observations and drill attitude.

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-PIT-103B

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-103C
PIN: 12662.00

Driller:	Maine Testboring, Inc.	**Elevation (ft.)**	210.3	**Auger ID/OD:**	5"
Operator:	B. Enos/C. Wormley	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	Mobile B-47 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/13/04-10/14/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	19+43.2, 10.0 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	209.95		PAVEMENT. — 0.35	
									Brown, damp, sandy GRAVEL, trace silt, (Fill).	
							207.90		— 2.40	
									Similar to above, but with cobbles, (Fill).	
5										
							202.30		— 8.00	
									Brown, wet, sandy GRAVEL, trace silt, (Fill).	
10	1D/AB	24/13	10.00 - 12.00	28/67/26/43	93	68			(1D/A) 10.0-11.0' bgs.	
						90	199.30		— 11.00	G#182507 A-2-4, SC-SM WC=31.3%
									(1D/B) 11.0-12.0' bgs. Brown and grey, moist, SAND, some gravel, some silt, trace clay. (Reworked Till).	
						130				
						a200			a200 blows for 0.9'.	
	R1	44.4/43	14.00 - 17.70	RQD = 62%		NQ	196.40		— 13.90	
							196.30		Top of Bedrock at Elev. 196.4'. Roller coned ahead fron 13.9-14.0' bgs. — 14.00	
15									R1:Bedrock: Grey, fine grained, interbedded metamorphic SILTSTONE and SLATE, moderately hard to hard, slightly weathered in the upper 16 inches to fresh, highly foliated. Flume Ridge Formation. Rock Mass Quality: Fair. R1: Core Times (min:sec) 14.0-15.0' (3:05) 15.0-16.0' (2:38) 16.0-17.0' (2:30) 17.0-17.7' (2:18) 96% Recovery	
	R2	60/56	17.70 - 22.70	RQD = 92%					R2:Bedrock: Grey, fine-grained, Metamorphic SILTSTONE, fresh, some quartz veins. Flume Ridge Formation. Rock Mass Quality: Excellent R2:Core Times (min:sec) 17.7-18.7' (2:15) 18.7-19.7' (2:32) 19.7-20.7' (2:40) 20.7-21.7' (2:55) 21.7-22.7' (3:07) 93% Recovery	
20										
							187.60		— 22.70	
									Bottom of Exploration at 22.70 feet below ground surface.	
25										

Remarks:

Existing Bridge Deck elevation at Abutment 2.

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: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-201
PIN: 12662.00

Driller:	MaineDOT	**Elevation (ft.)**	209.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/28/10; 07:30-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+85, 8.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.0 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	208.80		PAVEMENT. 0.70	
	1D	24/12	1.00 - 3.00	10/7/6/5	13	18		206.50		Brown, damp, medium dense, gravelly, fine to coarse SAND, some silt. (Fill) 3.00	
5	2D	24/17	5.00 - 7.00	4/3/4/6	7	10		201.50		Light brown, damp, loose, fine to coarse SAND, some gravel, some silt. (Fill). 8.00	G#237518 A-1-b, SM WC=9.5%
10	3D	24/15	10.00 - 12.00	5/7/6/12	13	18	50			Grey, saturated, medium dense, gravelly, fine to coarse SAND, trace silt, trace wood fragments; gravel is fine to coarse rounded to coarse angular granite rock fragments. (Fill)	
							53				
							68	196.40		a30 blows for 0.1 ft. 13.10	
	R1	60/60	13.10 - 18.10	RQD = 17%			a30 NQ-2	191.40		Top of Bedrock at Elev. 196.4 ft. R1:Bedrock: Grey, fine grained, interbedded SILTSTONE and SLATE, hard, fresh, thinly laminated in slate to massive siltstone beds, two joint sets, one at 45 degrees along bedding, 2nd vertical. Flume Ridge Formation. Rock Mass Quality: Very Poor. R1:Core Times (min:sec) 13.1-14.1 ft (7:00) 14.1-15.1 ft (4:00) 15.1-16.1 ft (3:45) 16.1-17.1 ft (4:10) No Water Return 17.1-18.1 ft (4:45) " " 100% Recovery 18.10	
15											
20										**Bottom of Exploration at 18.10 feet below ground surface.**	
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Princeton Bridge #2688 over Grand Falls Flowage **Location:** Princeton, Maine	**Boring No.:** BB-PIT-202 **PIN:** 12662.00

Driller:	MaineDOT	**Elevation (ft.)**	210.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/28/10; 10:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	19+40, 8.0 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	9.5 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	209.30		PAVEMENT. 0.70	
	1D	8.4/8	1.50 - 2.20	14/50(2.4")	---					Brown, dry, very dense, gravelly, fine to coarse SAND, little silt, occasional cobbles. (Fill).	
										Cobble from 3.4-3.9 ft bgs.	
										Cobble from 4.5-4.8 ft bgs.	
5								205.00		5.00	
	2D	24/13	5.00 - 7.00	8/5/6/24	11	15				Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt. (Fill).	G#237519 A-1-b, SM WC=8.0%
										Boulder from 7.2-8.6 ft bgs.	
							75				
							37	200.50		9.50	
10	3D	24/5	10.00 - 12.00	6/2/8/9	10	14	18			Telescoped with NW Casing through Boulder at 10.0 ft bgs. Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt; gravel is coarse, broken rock fragments.	
							32				
							34			Wood fibers and pieces in wash water from 12.5-13.5 ft bgs.	
							81				
							a178	195.20		a178 blows for 0.8 ft. 14.80	
15	R1	60/54	15.00 - 20.00	RQD = 27%			NQ-2	195.00		Top of Bedrock at Elev. 195.2 ft. Roller Coned ahead from 14.8-15.0 ft bgs. 15.00 R1:Bedrock: Grey, fine grained, hard, fresh, metamorphic SILTSTONE and SLATE, finely laminated slate to massive siltstone beds, jointing along bedding at moderately dipping angles, second set verical. Flume Ridge Formation. Rock Mass Quality: Poor. R1:Core Times (min:sec) 15.0-16.0 ft (3:15) 16.0-17.0 ft (3:50) No Water Return 17.0-18.0 ft (3:30) " " " 18.0-19.0 ft (3:25) " " " 19.0-20.0 ft (3:15) " " " 90% Recovery	
20								190.00		20.00 **Bottom of Exploration at 20.00 feet below ground surface.**	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

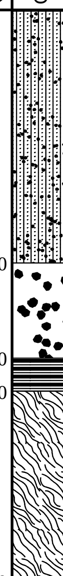
Project: Princeton Bridge #2688 over Grand Falls Flowage
Location: Princeton, Maine

Boring No.: BB-PIT-203
PIN: 12662.00

Driller:	MaineDOT	**Elevation (ft.)**	197.9	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/28/10; 13:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	18+63, 7.5 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Water Boring
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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	4/1/1/7	2	3	SPUN CASE			Grey, saturated, very loose, gravelly, fine to coarse SAND, trace silt. (Reworked Alluvium).	
	2D	24/12	2.00 - 4.00	5/10/13/7	23	32				Grey, saturated, medium dense, gravelly coarse Sand, little pockets of dark brown/grey silty fine sand, odor of "cut back asphalt"; gravel is slate fragments.	
5	R1	32.4/32.4	5.30 - 8.00	RQD = 0%			NQ-2	192.60		5.30 R1: Weathered bedrock fragments, trace sand and silt. Cobbles or Fractured Bedrock, 5.3-7.3 ft bgs. R1:Core Times (min:sec) 5.3-6.3 ft (2:20) 6.3-7.3 ft (6:45)	
								190.60			
	R2	48/48	8.00 - 12.00	RQD = 17%				189.90		7.30 R1:Weathered BEDROCK. 7.3-8.0 ft (9:75) 100% Recovery Core Blocked	
10										8.00 Top of Intack Bedrock at Elev. 189.9 ft. R2:Bedrock: Grey, fine grained, moderately hard, SLATE, fractured along bedding at 1/4 to 1/2 inch spacing, silt covered surfaces, changing to grey, fine grained, hard, metamorphosed Siltstone, occassional calcite veins, slighty weathered, massive. Flume Ridge Formation. Rock Mass Quality: Very Poor. R2:Core Times (min:sec) 8.0-9.0 ft (3:45) No Water Return 9.0-10.0 ft (6:25) " " 10.0-11.0 ft (3:25) " " 11.0-12.0 ft (4:25) " " 100% Recovery Core Blocked	
								185.90		12.00	
15										**Bottom of Exploration at 12.00 feet below ground surface.**	
20											
25											



Remarks:

12.2 ft from Bridge Deck to Ground.
1.0 ft Concrete Bridge Deck.

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-PIT-203

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Princeton Project Number: 12662.00

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-PIT-101, 1D	17+80.3	14.1 Rt.	5.0-7.0	182506	1	5.1			SM	A-1-b	II
BB-PIT-101, 3D	17+80.3	14.1 Rt.	15.0-17.0	182505	1	19.3			CL-ML	A-4	IV
BB-PIT-201, 2D	17+85	8.0 Lt.	5.0-7.0	237518	1	9.5			SM	A-1-b	II
BB-PIT-202, 2D	19+40	8.0 Lt.	5.0-7.0	237519	1	8.0			SM	A-1-b	II
BB-PIT-103C, 1D/B	19+43.2	10.0 Rt.	11.0-12.0	182507	1	31.3			SC-SM	A-2-4	III

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 MDOT and Corps of Engineers Classification Systems.

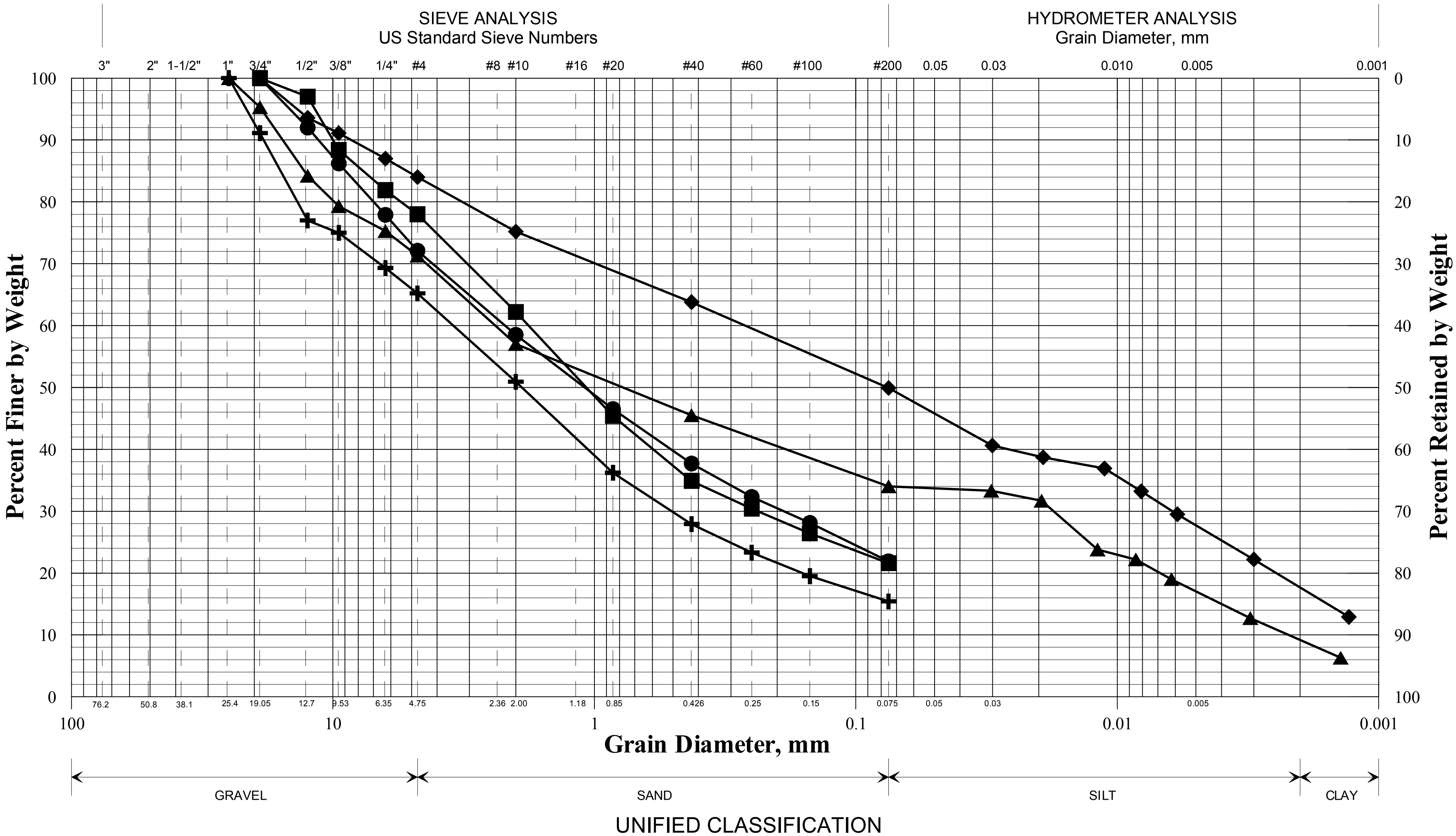
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-PIT-101/1D	17+80.3	14.1 RT	5.0-7.0	SAND, some gravel, little silt.	5.1			
◆	BB-PIT-101/3D	17+80.3	14.1 RT	15.0-17.0	SAND, some silt, little clay, little gravel.	19.3			
■	BB-PIT-201/2D	17+85	8.0 LT	5.0-7.0	SAND, some gravel, some silt.	9.5			
●	BB-PIT-202/2D	19+40	8.0 LT	5.0-7.0	SAND, some gravel, some silt.	8.0			
▲	BB-PIT-103C/1D(B)	19+43.2	10.0 RT	11.0-12.0	SAND, some gravel, some silt, trace clay.	31.3			
✖									

PIN
012662.00
Town
Indian Twp Res
Reported by/Date
WHITE, TERRY A 11/12/2010

SHEET 1

<u>Appendix C</u>

Calculations

Part I - Factored Bearing Resistance - Abutment Spread Footing Foundations 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".

Description of Bearing Material:

Abutment 1: Boring BB-PIT-101, upper 5-ft core, metasiltstone, hard, slightly weathered to fresh, joints/bedding stee. **RQD=70%**. Boring BB-PIT-201, **RQD = 17%**

Abutment 2: Boring BB-PIT-103A, upper 5-ft core is meta-siltstone, mod. hard to hard, slightly weathered, highly foliated with **RQD = 62%.** Boring BB-PIT-202 **RQD=27%**

Abutment 1 and 2

Bearing Material:	Weathered or broken bedrock of any kind except argillite (shale).
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16 - 24 ksf
Recommended Value	16 ksf

$q_{nominal} := 16 \cdot ksf$

Resistance Factor for Service Limit State

$\phi_r := 1.0$

Factored Bearing Resistance for Service Limit State Analyses; settlement limited to 1.0 inch

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

$q_{factored} = 16 \cdot ksf$

Recommendation for **Abutments & Wingwalls:** Use **16 ks**f for service limit state analysis - and for preliminary sizing of the footing.

Recommended value for **Pier** for the factored bearing resistance. Use **16 ksf** for service limit state analysis - and for preliminary sizing of the footing.

Part II - Factored Bearing Resistance for Abutment Footngs - Strength Limit State Analyses

Method 1 - Nominal & Factored Bearing Resistance of bedrock, per Kulhawy & Goodman, 1980

Reference: International Conference on Structural Foundations on Rock, Sydney, May 1980, Pells, "Design of foundations on discontinuous rock" Kulhawy and Goodman.

Equation (5) - For open joints, failure is likely to occur by uniaxial compression of the rock columns. In this case the ulitmate bearing capacity is given by the Mohr Coulomb theory qult=qu=2ctan(45 + ϕ/2) in which qu, c and ϕ are rock mass properties.

$\phi_{rock} := 20 \cdot deg$ — *Tomlinson, Page 139, Wyllie, phi for low friction rock, schists 20-27*

$q_{uc} := 9200 \cdot psi$ — *AASHTO, 2002, Table 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength, "siltstone" 1,400 to 17,000 psi*

$c := 0.1 \cdot q_{uc}$ — *Tomlinson, page 139, referencing Kulhawy & Goodman correlation for c based on RQD and quc*

$c = 920 \cdot psi$ — *OK - correlates to Bowles, pg 278, giving range for rock cohesion of 500-2500 psi*

$c := .55 \cdot \frac{MN}{m \cdot m}$ $\quad c = 80 \cdot psi$ — *Cohesion selected from reference: Hoek, Marinos & Benissi, Bull (AEG, 1988); sandstone; Short Course Lecture Notes, 2005, Estimation of Soil and Rock Properties for Foundation Design, Dr. Fred Kulhawy*

$$q_{nominal} := 2 \cdot c \cdot \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)$$

$$q_{nominal} = 33 \cdot ksf$$

Factored Bearing Resistance

U*se a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1*

$\phi_{bc} := 0.45$ $\quad q_{factored} := q_{nominal} \cdot \phi_{bc}$ $\quad q_{factored} = 15 \cdot ksf$

Method 2 - Nominal & Factored Bearing Resistance of bedrock, per Bowles, 5th Edition, Section 4-16 page 277

Typical Unit Weight, reference Bowles 5th Edition, page 278, Table 4-11

$\gamma := 26 \cdot \frac{kN}{m^3}$ $\gamma = 166 \cdot pcf$ for schist; similar to phyllite

Cohesion, Reference: Hoek, Marinos & Benissi, Bull (AEG, 1988)

$c := 0.55 \cdot MPa$ $c = 80 \cdot psi$

Bearing Capacity Factors

$$N_q := \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^6 \qquad N_q = 8.485$$

$$N_c := 5 \cdot \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^4 \qquad N_c = 20.8$$

$$N_\gamma := N_q + 1 \qquad N_\gamma = 9.485$$

Terzaghi Shape Factors, Bowles, Table 4-1, page 220

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

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

Embedment factor - footing placed on top of bedrock

$q := \gamma \cdot 0 \cdot ft$ $q = 0$

Nominal Bearing Resistance

$$q_{ult} := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{ult} = \begin{pmatrix} 244 \\ 245 \\ 247 \\ 248 \end{pmatrix} \cdot ksf$$

Reduce the calculated bearing resistance by RQD^2, per Bowles. Use averaged RQD encountered at top of bedrock in 4 borings: 17, 70, 27, 62%

$$RQD := \frac{0.17 + .70 + .27 + .62}{4} \qquad RQD = 0.44$$

$$q_{nominal} := q_{ult} \cdot RQD^2$$

$$q_{nominal} = \begin{pmatrix} 47 \\ 47 \\ 48 \\ 48 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$$\phi_{bc} := 0.45 \qquad q_{factored} := q_{nominal} \cdot \phi_{bc} \qquad \boxed{q_{factored} = \begin{pmatrix} 21 \\ 21 \\ 21 \\ 22 \end{pmatrix} \cdot ksf}$$

Recommended Factored Bearing Resistance of 20 ksf for strength limit state analyses.

Part I - Factored Bearing Resistance - Pier Spread Footing Foundations 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".

Description of Bearing Material:

Pier: Boring BB-PIT-102, upper 5-ft core, metasiltstone, hard, slightly weathered, very fractured on steep joints/bedding. **RQD=0% for upper 4 feet, then 62% then 9%.** Boring BB-PIT-203, **RQD=17%** for one 5-ft rock core run.

Pier

Bearing Material:	Weathered or broken bedrock of any kind except argillite (shale).
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16 - 24 ksf
Recommended Value	16 ksf

$$q_{nominal} := 16 \cdot ksf$$

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

Factored Bearing Resistance for Service Limit State Analyses; settlement limited to 1.0 inch

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

$$q_{factored} = 16 \cdot ksf$$

Recommended value for **Pier** for the factored bearing resistance. Use **16 ksf** for service limit state analysis - and for preliminary sizing of the footing.

Part II - Factored Bearing Resistance for Pier Foundtion - Strength Limit State Analyses

Method 1 - Nominal & Factored Bearing Resistance of bedrock, per Kulhawy & Goodman, 1980

Reference: International Conference on Structural Foundations on Rock, Sydney, May 1980, Pells, "Design of foundations on discontinuous rock" Kulhawy and Goodman.

Equation (5) - For open joints, failure is likely to occur by uniaxial compression of the rock columns. In this case the ulitmate bearing capacity is given by the Mohr Coulomb theory qult=qu=2ctan(45 + ϕ/2) in which qu, c and ϕ are rock mass properties.

$\phi_{rock} := 20 \cdot deg$ *Tomlinson, Page 139, Wyllie, phi for low friction rock, schists 20-27*

$q_{uc} := 1400 \cdot psi$ *AASHTO, 2002, Table 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength, "siltstone" 1,400 to 17,000 psi*

$c := 0.1 \cdot q_{uc}$ *Tomlinson, page 139, referencing Kulhawy & Goodman correlation for c based on RQD and quc*

$c = 140 \cdot psi$ *OK - correlates to Bowles, pg 278, giving range for rock cohesion of 500-2500 psi*

$c := .55 \cdot \frac{MN}{m \cdot m}$ $c = 80 \cdot psi$ *Cohesion selected from reference: Hoek, Marinos & Benissi, Bull (AEG, 1988); sandstone; Short Course Lecture Notes, 2005, Estimation of Soil and Rock Properties for Foundation Design, Dr. Fred Kulhawy*

$$q_{nominal} := 2 \cdot c \cdot \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)$$

$q_{nominal} = 33 \cdot ksf$

Factored Bearing Resistance

U*se a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1*

$\phi_{bc} := 0.45$ $q_{factored} := q_{nominal} \cdot \phi_{bc}$ $q_{factored} = 15 \cdot ksf$

Method 2 - Nominal & Factored Bearing Resistance of bedrock, per Bowles, 5th Edition, Section 4-16 page 277

Typical Unit Weight, reference Bowles 5th Edition, page 278, Table 4-11

$\gamma := 26 \cdot \frac{kN}{m^3}$ $\gamma = 166 \cdot pcf$ for schist; similar to phyllite

Cohesion, Reference: Hoek, Marinos & Benissi, Bull (AEG, 1988)

$c := 0.55 \cdot MPa$ $c = 80 \cdot psi$

Bearing Capacity Factors

$N_q := \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^6$ $N_q = 8.485$

$N_c := 5 \cdot \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^4$ $N_c = 20.8$

$N_\gamma := N_q + 1$ $N_\gamma = 9.485$

Terzaghi Shape Factors, Bowles, Table 4-1, page 220

$s_c := 1.0$ $s_\gamma := 1.0$ $B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$

Embedment factor - footing placed on top of bedrock

$q := \gamma \cdot 0 \cdot ft$ $q = 0$

Nominal Bearing Resistance

$q_{ult} := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma$

$$q_{ult} = \begin{pmatrix} 244 \\ 245 \\ 247 \\ 248 \end{pmatrix} \cdot ksf$$

Reduce the calculated bearing resistance by RQD^2, per Bowles.
Lowest RQD's encountered at top of bedrock in pier borings: 0% and 17% - assume
CD notes required excavation of loose fractured bedrock to a minimum RQD of 30%

$$RQD := 0.30$$

$$q_{nominal} := q_{ult} \cdot RQD^2$$

$$q_{nominal} = \begin{pmatrix} 22 \\ 22 \\ 22 \\ 22 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$$\phi_{bc} := 0.45 \qquad q_{factored} := q_{nominal} \cdot \phi_{bc} \qquad \boxed{q_{factored} = \begin{pmatrix} 10 \\ 10 \\ 10 \\ 10 \end{pmatrix} \cdot ksf}$$

Recommended Factored Bearing Resistance of 10 ksf for strength limit state analyses.

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

From Design Freezing Index Map:
Princeton, Maine
DFI = 1600 degree-days

Case I - Medium to coarse grained fill soils -WC=10%.

Use DFI = 1600

Depth of Frost Penetration = 84.8 inch

$d := 84.8 \cdot in$ $\quad d = 84.8 \cdot in$ $\quad d = 7.067 \cdot ft$

Method 2 - ModBerg Software

Examine coarse grained soils without 4 inches of asphalt

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

Project Location: Orono, Maine

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

--------------------------------------------------------
Layer
#:Type          t    w%    d    Cf  Cu   Kf   Ku    L
--------------------------------------------------------
1-Coarse      77.3 10.0 125.0  28  34   2.0  1.6  1,800
--------------------------------------------------------

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

*********************************************************
  Total Depth of Frost Penetration = 6.45 ft = 77.3 in.
*********************************************************
```

Recommendation: 6.5 feet for design of spread footings not founded on bedrock

Calculation

Determination of site class for Princeton Bridge substructures

Method

Use Shear wave velocity to determine site class per LRFD Table 3.10.3.1-1

Reference: Das, Fundamentals of Soil Dynamics, (1983) page 286.

Shear modulus for sands, sands and gravels, based on Seed and Idriss (1970), provided in Das (1983), Equation 8.48:

$$\boxed{G := 1000 \cdot K_2 \cdot \sqrt{\sigma_o}}$$

G and effective overburden stress in lb/ft^2

Estimate K2 from Das (1983) Figure 8.16 and 8.15

Use Curve from Figure 8.16 for "sand, gravel, cobbles with little clay

$$K_2 := 90$$

Typical Values for density of Sands and Gravels, Holtz and Kovacs (1981), An Introduction to Geotechnical Engineering, Table 2-1

Report as unitless (lb/sf)

$$\rho_{sat} := 2.1 \cdot \frac{1000 \cdot kg}{m^3} \qquad \rho_{sat} = 131.099 \cdot \frac{lb}{ft^3} \qquad \rho_{sat} := 131$$

$$\rho_{dry} := 1.9 \cdot \frac{1000kg}{m^3} \qquad \rho_{dry} = 118.613 \cdot \frac{lb}{ft^3} \qquad \rho_{dry} := 118$$

$$\rho' := 1.15 \cdot \frac{1000 \cdot kg}{m^3} \qquad \rho' = 71.792 \cdot \frac{lb}{ft^3} \qquad \rho' := 72$$

Determination of G based on Bowles Eq. 20.15

$$\boxed{V_s := \sqrt{\frac{G}{\rho}}}$$

Groundwater conditons;

$\gamma_w := 62.4$

$D_w := 5 \cdot ft$

Soil Profile at BB-PIT-201

Layer 1 - 3 feet of fill
Layer 2 - 5 feet of fill
Layer 3 - 5.1 feet of fill

Layer 1

Thickess Layer $H_1 := 3$ Remove units - report in ft

Effective overburden stress at midpoint of layer

$$\sigma'_{v1} := \frac{H_1}{2} \cdot \rho_{dry} \qquad \sigma'_{v1} = 177$$

Spring constant $K := 90$ Unitless

Shear Modulus $\boxed{G_1 := 1000 \cdot K \cdot \sqrt{\sigma'_{v1}}}$ $G_1 = 1 \times 10^6$

Determination of Shear Velociy based on Bowles Eq. 20.15

$$\boxed{V_{s\_1} := \sqrt{\frac{G_1}{\rho_{dry}}}} \qquad V_{s\_1} = 100.733$$ in ft/sec

Ratio of di / Vsi

$$\frac{H_1}{V_{s\_1}} = 0.03$$

Layer 2

Thickess Layer $H_2 := 5$ groundwater 2 feet into the 5 feet

Effective overburden stress at midpoint of layer

$$\sigma'_{v2} := 5 \cdot \rho_{dry} + 0.5 \cdot (\rho_{sat} - 62.4)$$

$$\sigma'_{v2} = 624.3$$

Spring constant $K := 90$ Unitless

Shear Modulus $G_2 := 1000 \cdot K \cdot \sqrt{\sigma'_{v2}}$ $G_2 = 2248740$

Determination of Shear Velociy based on Bowles Eq. 20.14

$$V_{s\_2} := \sqrt{\frac{G_2}{\rho_{dry}}}$$

$V_{s\_2} = 138.048$ in ft/sec

Ratio of di / Vsi

$$\frac{H_2}{V_{s\_2}} = 0.036$$

Layer 3

Thickess Layer $H_3 := 5$

Effective overburden stress at midpoint of layer

$$\sigma'_{v3} := 5 \cdot \rho_{dry} + 5.55 \cdot (\rho_{sat} - 62.4)$$

$$\sigma'_{v3} = 970.73$$

Spring constant $K := 90$ Unitless

Shear Modulus $G_3 := 1000 \cdot K \cdot \sqrt{\sigma'_{v3}}$ $G_3 = 2804089$

Determination of Shear Velociy based on Bowles Eq. 20.14

$$V_{s\_3} := \sqrt{\frac{G_3}{\rho_{dry}}} \qquad V_{s\_3} = 154.154$$ in ft/sec

Ratio of di / Vsi

$$\frac{H_3}{V_{s\_3}} = 0.032$$

Layer 4 - Bedrock - Interbedded Slate and Siltstone

$$H_4 := 100 - \left(H_1 + H_2 + H_3\right) \qquad H_4 = 87$$

Shear wave velocity

$$V_{s\_4} := 5000$$ ft/sec

$$\frac{H_4}{V_{s\_4}} = 0.017$$

Average Vs for the top 100 ft is determined per LRFD Table C3.10.3.1-1, Method A

$$v_s := \frac{100}{\frac{H_1}{V_{s\_1}} + \frac{H_2}{V_{s\_2}} + \frac{H_3}{V_{s\_3}} + \frac{H_4}{V_{s\_4}}}$$

$$v_s = 863.289$$

Site Class D - 600 ft/s < vs < 1,200 ft/s

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04668
Zip Code Latitude = 45.201700
Zip Code Longitude = -067.592000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.081	PGA - Site Class B
0.2	0.162	Ss - Site Class B
1.0	0.043	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04668
Zip Code Latitude = 45.201700
Zip Code Longitude = -067.592000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.129	As - Site Class D
0.2	0.260	SDs - Site Class D
1.0	0.104	SD1 - Site Class D