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

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of 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 $\phi = 32$ degrees, $\gamma = 125$ 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 <u>abutment</u> 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 <u>pier</u> 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 to 196 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: $\phi = 32$ degrees, $\gamma = 125$ 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 multiplespan 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



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



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Logged E Date Sto Boring L Definition D = Split MD = Unsuc U = Thin U R = Rock (V = Insitu SSA = Sol	By: art/F ocat ns: Spoon ccessfu	G inish: 1 ion: 1	. Lidstone 0/13/04-10/	′14/04	Rig	Туре:		Mobi	le B-47 Trailer
Date Sto Boring L Definition D = Split MD = Unsud U = Thin L R = Rock (V = Insite SSA = Sol	art/F _ocat ns: Spoon ccessfu	inish: 1 ion: 1	0/13/04-10/	14/04	0:				
Boring L Definition D = Split MD = Unsua U = Thin N R = Rock (V = Insite SSA = Sol	OCat ns: Spoon ccessfu	ion: 1	0+43 2 10		Uri	lling I	Method:	Case	ed Wash Boring
U = Thin I MD = Unsue U = Thin I R = Rock (V = Insitu SSA = Sol	ns: Spoon ccessfi Wall Ti		5+45.21 10.	0 ft Rt.	Cas	ing ID.	/OD:	HW	
	Core So u Vane id Ster	Sample ul Split Sp ube Sample ample Shear Test n Auger	oon Sample att	emp†	5 _u = T _v = q _D = S _{u(1} WOH WOR	= Insitu = Pocket = Unconfi ab) = La = weight = weight	Field Van Torvane S ned Compr b Vane Sh of 1401b of rods	e Shear hear Sti essive S ear Stri hammei WOC = 1	Strength (psf) rength (psf) Strength (ksf) ength (psf) r weight of casing
		2	S	ample Information		-	-		
Depth (ft.)	Sample No.	Pen./Rec. (ir	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or ROD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	
0						SSA	209.95	****	PAVEMENT.
5							207.90		Similar to ab
10 10)/AB	24/13	10.00 - 12.00	28/67/26/43	93	68	199.30		(1D/A) 10.0-1 (1D/B) 11.0-1
						130 9200	196.40		Brown and gre clay. (Rework 9200 blows fo
15	R1	44.4/43	14.00 - 17.70	ROD = 62%		QИ		Di AMONI	Roller coned Roller coned R1:Bedrock: C SILTSTONE and weathered in Flume Ridge F
	R2	60/56	17.70 - 22.70	ROD = 92%				ANOV AND ANOVANA	R1: Core Time 14.0-15.0' (3 15.0-16.0' (2 16.0-17.0' (2 17.0-17.7' (2 R2:Bedrock: C fresh, some c
20								A CONTRACTION OF CONT	Ouglity: Exce R2:Core Times 17.7-18.7' (2 18.7-19.7' (2 19.7-20.7' (2 20.7-21.7' (2 21.7-22.7' (3
						v	187.60	2119 212	Bottom of Ex

ar	tment c	of Transport	ati	on	roject:	Princ	idge #2688	Boring No.:	<u></u>	T-103A
<u>So i</u>	US CUSTOMA	loration Log RY UNITS		ι	.ocation	over n: Prin	Maine	PIN:	126	62.00
	Maine Testb	oring, Inc.	Ele	vation	(ft.)	210		Auger ID/OD:	5″	
	B. Enos/C.	Wormley	Dat	um:		NAV(Sampler:	N/A	
	G. Lidstone		Rig	Type:		Mob	'Trailer	Hammer Wt./Fall:	N/A	
sh:	10/13/04-10	/13/04	Dri	lling	Method:	Sol	Auger	Core Barrel:	N/A	
:	19+38.2. 12	.0 ft Rt.	Cas	ing ID	/OD:	N/A		Water Level*:	None Observ	ved
ole olitS Sample s ar Tes	poon Sample at	tempt	Defi S _U = T _V = Q _D = S _{U(1} WOH	nitions: Insitu Pocket Unconfi ab) = Lo = weight	Field Var Torvane S ned Compr bb Vane Sh of 1401b	hear St hear St essive hear Str hamme	(psf) sf) (ksf) f)	Definitions: WC = water content, perc LL = Liquid Limit PL = Plastic Limit Pl = Plasticity Index G = Grain Size Analysis C = Complete Year	ent	
jer O		Sample Information	WUR	= weight	ofrous	WUC =	casing	c = consolidation lest		
Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in Shear (/6 in Strength (psf) or ROD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Descr	ription and Remarks		Laboratory Testing Results/ AASHTO and Unified Clas
				SSA	209.85		ENT.		-0.35	
					207.70		ar to above, but with ar on of Exploration at	8.10 feet below groun	2.50 8.10 nd surfoce.)-

n lines represent approximate boundaries between soil typest transitions may be gradual. Page 1 of 1 readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other Boring No.: BB-PIT-103A

							T 4070
t at i	on _F	Project:	Princ over	eton Bridge #2688 Grand Falls Flowage	Boring No.:	<u></u>	<u>1–103C</u>
	L	ocation	: Prin	nceton, Maine	PIN:	1266	2.00
Ele	vation	(ft.)	210.	3	Auger 1D/OD:	5″	
Dat	um:		NAV) 88	Sampler:	Standard Spl	it Spoon
Rig	Туре:		Mob	ile B-47 Trailer	Hammer Wt./Fall:	140#/30"	
Dri	lling I	Method:	Case	ed Wash Boring	Core Barrel:	NQ-2"	
Cas	ing ID.	/00:	НW		Water Level*:	8.0 ft bgs	
S _u =	Insitu	Field Van	e Shear	Strength (psf)	WC = water content, perc	ent	
• v q _p =	= Pocket = Unconfi	ned Compr	essive	strength (ksf)	PL = Plastic Limit		
S _{u(1} WOH	ab) = La = weight	ab Vane Sh t of 1401b	ear Str • hamme	ength (psf) r	PI = Plasticity Index G = Grain Size Analysis		
WOR	= weight	t of rods	WOC =	weight of casing	C = Consolidation Test		
N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Descri	iption and Remarks	U	Laboratory Testing Results/ AASHTO and nified Class
_	554	209.95	~~~~	PAVEMENT.		0.35	
	334	_		Brown, damp, sandy GRAVEL,	trace silt, (Fill).		
		207.90		Similar to above but with		2.40-	
		1					
		-					
		-					
		-					
		202.30				8,00-	
		202.30		Brown, wet, sandy GRAVEL, t	race silt, (Fill).	0.00	
	$\mathbb{N}/$						
	V	_		(1D/A) 10.0-11.0' bgs.			
93	68	100 30				11_00-	
	90	133.30		(1D/B) 11.0-12.0' bgs. Brown and grey, moist, SAND	. some gravel, some	silt. trace	G#182507 4-2-4, SC-SM
	130	-		clay. (Reworked Till).			WC=31.3%
	150	-		@200 blows for 0.9'.			
	a200	196.40					
	NQ	196.30	90	Top of Bedrock at Elev. 196 Roller coned ahead fron 13.	.4'. 9-14.0' bgs.		
			H.H.	R1:Bedrock: Grey, fine grai	ned, interbedded meto	14.00 amorphic	
		_	ISTAL.	SILTSTONE and SLATE, modera weathered in the upper 16 i	tely hard to hard, s nches to fresh, high	lightly ly foliated.	
			9199	Flume Ridge Formation, Rock R1: Core Times (min:sec)	Mass Quality: Fair.		
			Roll	14.0-15.0' (3:05) 15.0-16.0' (2:38)			
		1	AU.	16.0-17.0' (2:30)	05V		
-		-	SC 11	R2:Bedrock: Grey, fine-grai	ned, Metamorphic SIL	TSTONE . Rock Moss	
			19 <u>1</u> 0	Quality: Excellent	rana ninge runnurfulli	. AGON MUSS	
			01%D	17.7-18.7' (2:15)			
		1	<u>USM</u>	18.7-19.7' (2:32) 19.7-20.7' (2:40)			
	/	-	66	20.7-21.7' (2:55) 21.7-22.7' (3:07) 93% Recov	ery		
<u> </u>	V	187.60	en inc	Bottom of Exploration at 2	2,70 feet below arou	22.70- nd surface.	
		1					
			au 1-	ratual			
soii typ	est tran	SITIONS M	uy be gr				
tions st	ated. G	roundwater	r fluctu	uations may occur due to conditions o	Boring No.	: BB-PIT-1	03C
					•		

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION					PIN	RRIDGE NO 2688 12662 00 RRIDGE DI ANS	
		SIGNATURE			- P.E. NUMBER				-
DATE									
PROJ. MANAGER D. ANDERSON BY	DESIGN-DETAILED L. KRUSINSKI T. WHITE	CHECKED-REVIEWED	DESIGN2-DETAILED2	DESIGN3-DETAILED3	REVISIONS 1	REVISIONS 2	REVISIONS 3	REVISIONS 4	FIELD CHANGES
PRINCETON BRIDGE		GRAND FALLS FLOWAGE		FRINCEION WADMINGION COUNTY					
s	HI	EE	T	N	U	ME	3E	R	
			6		3				
		(DF	-	4				

	ine l)epar <u>so</u>	tment c <u>il/Rock Exp</u> <u>us customa</u>	of Transport Ioration Log RY UNITS	-ati	on	Project Locatio	Princover	eton Br Grand F nceton,	ridge #2688 Falls Flowage Maine	Boring No.: PIN:	<u>BB-PI</u> 1266	<u>T-201</u> 2.00
D~: 1			Mai coDOT				. (200	E		Augor 10 (00.		
Opera				as (Dagaatt	Det			209				Stopdard Sol	
Upero				esibaggen				CME	450		Hammar Wt (Falls	140#/30"	11 30001
Logge	a By:		B. Wilder	7.70.40.70	RIÇ	j Type:		CME	450	0	Hummer WT./Fall:	140#730	
Date	Start/	rinish:	10/28/10: 0	07:30-10:30	Dri	lling	Method:	Cas	ed Wash	Boring	Core Barrel:	NQ-2"	
Borin	ng Loca	tion:	17+85, 8.0	ft Lt.	Cas	sing IC)/00:	NW			Water Level*:	5.0 ft bgs.	
Hamme	er Effi	ciency Fo	actor: 0.84		Han	nmer Ty	/pe:	Autom	ntic 🛛	Hydraulic 🗆	Rope & Cathead 🗌		
Defini D = Sp MD = U U = Th MU = U V = In MV = U	itions: Diit Spoo Insuccess Din Wall Insuccess Disitu Van Insuccess	n Sample ful Split S Tube Sample ful Thin Wo e Shear Tes ful Insitu	poon Sample at III Tube Sample It PP = Poo Vane Shear Tes	R = Rock SSA = So tempt HSA = Ho RC = Rol attempt WDH = we ket PenetrometerWDR/C = ket PenetrometerWDR/C = tigttempt WD1P = W Somo Le Information	Core So lid Ster llow Ster ler Con- ight of weight o <u>eight o</u>	ample m Auger e 1401b. of rods <u>f one pe</u>	or casing erson		$S_U = Ins$ $T_V = Poc$ $q_p = Unc$ N-uncorr Hammer E $N_{60} = SP$ $N_{60} = (H$	itu Field Vone Shear Strength (p onfined Compressive Strength ected = Raw field SPT N-value fficiency Factor = Annual Cal T N-uncorrected corrected for <u>ammer Efficiency Factor/60%</u>	h (pst) Su(1ab psf) WC = w (ksf) LL = L e PL = P libration Value P = P r hammer efficiency G = Gr NN-uncorrected C = Cc	<pre>> = Lab Vane Shear ater content, perc iquid Limit lastic Limit lasticity Index ain Size Analysis nsolidation Test</pre>	Strength (pst
(++.)	No.	ec. (in	Depth	(/e in.	rrected			ion	c Log	Visual Des	scription and Remarks	5	Laboratory Testing Results/ AASHTD
Depth	Samp I e	Pen./R	Sample (ft.)	Blows Shear Streng (psf) or ROD	N-Uncol	N60	Casing Blows	Elevat (ft.)	Graphi			U	and hified Class
0							SSA			PAVEMENT.			
	1D	24/12	1.00 - 3.00	10/7/6/5	13	18		208.80		Brown, damp, medium da SAND, some silt. (Fil	ense, gravelly, fine	0.70- to coarse 3.00-	
• 5 •	20	24/17	5.00 - 7.00	4/3/4/6	7	10				Light brown, damp, lo gravel, some silt, (F	ose, fine to coarse (ill).	SAND, some	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, media SAND, trace silt, trac to coarse rounded to a fragments. (Fill)	um dense, gravelly, ce wood fragments; g coarse angular grani	fine to coarse ravel is fine te rock	
			17 10				68	196.40		$\sqrt{30}$ blows for 0.1 ft.			
• 15 •	R1	60/60	18.10	ROD = 17%			030 N0−2			Top of Bedrock at Eler R1:Bedrock: Grey, find and SLATE, hard, fresh massive siltstone bed	v. 196.4 ft. e grained, interbedd h, thinly laminated s, two joint sets, o	-13.10- ed SILTSTONE in slate to ne at 45	
										Formation. Rock Mass R1:Core Times (min:sec 13.1-14.1 ft (7:00)	• 2nd vertical. Flum Quality: Very Poor. c)	e Ridge	
								191.40		14.1-15.1 ft (4:00) 15.1-16.1 ft (3:45) 16.1-17.1 ft (4:10) No 17.1-18.1 ft (4:45) " 100% Recovery	o Water Return "		
• 20 •										BOTTOM OT EXPLORATI	surface.	ow ground	
25													
<u>Remar</u> Strati	fication	lines repr	esent approxim	nate boundaries between s	soil typ	pes: tra	nsitions m	nay be g	radua I •		Page 1 of 1		

terry

User

Ма	ine [)epar	tment a	of Transpor	tati	on	Project	Princ	eton E	ridge #2688 Boring No.: <u>BB-PIT-2</u>	03
		<u>So</u>	I / ROCK EXP	loration Log			Location	over n: Pri	nceton	Maine PIN: 12662.0	0
Dr: 1	lort				E L	wation	(= +)	107	0		
Opero	ator:		Giquere/Gil	les/Daggett	Dat	tum:	(11.)	NAV			DOOD
Logge	ed By:		B. Wilder		Ric	j Type:		CME	45C	Hammer Wt./Fall: 140#/30"	p 0011
Date	Start/F	inish:	10/28/10; 1	13:00-15:30	Dri	illing	Method:	Cas	ed Was	Boring Core Barrel: NO-2"	
Borin	ng Loca	tion:	18+63, 7.5	ft Lt.	Cas	sing ID	/0D:	NW		Water Level*: Water Boring	
Hamme	er Effic	ciency F	actor: 0.84		Har	nmer Ty	pe:	Autom	otic ⊠	Hydraulic 🗌 Rope & Cathead 🗌	
Defini D = Sp MD = L U = Tr MU = L V = Ir MV = L	itions: blit Spoor Jnsuccessi nin Wall 1 Jnsuccessi nsitu Vane Jnsuccessi	n Sample Sul Split S Sube Sample Sul Thin We Shear Te Shear Te	Spoon Sample a e all Tube Sample st, PP = Poo Vane Shear Tes	R = Rock SSA = So ttempt HSA = Ho RC = Ro e attempt WDH = wo sket PenetrometerWOR/C = st attempt WD1P = 1	Core S olid Ste ollow St ler Con eight of weight <u>leight o</u>	ample em Auger em Auger 14016.1 of rods of rods	hammer or casing rson		$S_u = In$ $T_v = Po$ $a_p = Un$ N-uncorr Hammer I $N_{60} = Si$ $N_{60} = (1)$	situ Field Vane Shear Strength (psf) Su(lab) = Lab Vane Shear Strength (psf) sket Torvane Shear Strength (psf) WC = water content, percent sconfined Compressive Strength (ksf) LL = Liquid Limit scoted = Raw field SPT N-value PL = Plastic Limit Striciency Factor = Annual Calibration Value PI = Plasticity Index T N-uncorrected corrected for hammer efficiency G = Grain Size Analysis C = Consolidation Test	ngth (psf
		<u>.</u>	C	c	pe				1	Labo	oratory
Depth (ft.)	Sample No.	Pen./Rec. (Sample Depth (ft.)	Blows (/6 ir Shear Strength (psf) or ROD (%)	N-uncorrecte	N60	Casing Blows	Elevation (ft.)	Graphic Log	Te: Res AA Unifie	sting sults/ SHTO and ed Class
0	1 D	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 •		32.4/	5.30 -					192.60			
	KI	32.4	8.00	RUD = 0%			NU-2			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)	
	R2	48/48	8.00 -	ROD = 17%				190.60 189.90	SU150	-6.3-7.3 ft (6:45) R1:Weathered BEDROCK. 7.3-8.0 ft (9:75) 100% Recovery	
• 10 •			12:00							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,	
								185.90		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) " " " 11.0-11.0 ft (3:25) " " "	
·15 -										100% Recovery Core Blocked Bottom of Exploration at 12.00 feet below ground surface.	
20											
25											
12.3 1.0	<u>rks:</u> 2 ft fri ft Coni	om Bridg crete Br	e Deck to G idge Deck.	round.							
Strati	ification	lines rep	resent approxim	nate boundaries between	soil typ	bes: tran	nsitions m	nay be g	radual.	Page 1 of 1	
* Wate than	er level r those pr	eadings h esent at t	ave been made d he time measur	at times and under condi ements were made.	tions st	tated, G	Groundwate	r fluct	uations	may occur due to conditions other Boring No.: BB-PIT-203	

Mai	ine (Depar	tment c	of Transport	tati	on	Project	Princ	eton (Grand	ridge #2688 Boring No.: Falls Flowage	BB-PI	T-202
		20	US CUSTOMA	<u>RY UNITS</u>		ľ	Locatio	on: Pri	nceton	Maine PIN:	1266	2.00
Drill	er:		MaineDOT		Ele	evation	(ft.)	210	•0	Auger ID/0D: 5	" Solid Ste	em
Opero	itor:		Giguere/Gil	es/Daggett	Dat	'UM :		NAV	D 88	Sampler: S	tandard Sp	it Spoon
Logge	ed By:	Finicht	B. Wilder	0.30-13.00	Riç) Type:	Nothoda	CME	45C	Hammer Wt./Fall: 1	40#/30"	
Borin		tion:	19+40, 8.0	ft Lt.	Cas	sing ID	/0D:	. соз нw	8 NW	Water Level*: 9	.5 ft bgs.	
Hamme	er Effi	ciency Fo	actor: 0.84		Harr	nmer Ty	pe:	Autom	otic ⊠	Hydraulic 🗌 Rope & Cathead 🗌	-	
Defini D = Sp MD = U U = Th MU = U V = In MV = U	tions: Dit Spoor Insuccess Din Wall Insuccess Disitu Van Insuccess	n Sample ful Split S Tube Sample ful Thin Wo e Shear Tes ful Insitu	poon Sample at ili Tube Sample it, PP = Poo Vane Shear Tes	R = Rock SSA = So RC = Rol actempt WDH = we sket PenetrometerWDR/C = tottempt WD1P = W	Core So lid Stea llow Stea ler Cona ight of weight o eight o	ample m Auger em Auger i 14015.1 of rods f one pe	hammer or casing rson	g	$S_{U} = In$ $T_{V} = Pa$ $a_{p} = Un$ N-uncor Hammer $N_{60} = S$ $N_{60} = ($	itu Field Vane Shear Strength (psf) Su(lab) = ket Torvane Shear Strength (psf) WC = water onfined Compressive Strength (ksf) LL = Liqui ected = Raw field SPT N-value PL = Plast fficiency Factor = Annual Calibration Value PI = Plast T N-uncorrected corrected for hammer efficiency G = Grain ammer Efficiency Factor/60%)=N-uncorrected	Lab Vane Shear content, pero d Limit ic Limit icity Index Size Analysis idation Test	- Strength (psf cent
oDepth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	B lows (/6 in. Shear Shear Strength (pst) or ROD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	U	Laboratory Testing Results/ AASHTO and nified Class
-							SSA	209.30		I AVLMLNI.	0.70-	
	1D	8.4/8	1.50 - 2.20	14/50(2.4")						Brown, dry, very dense, gravelly, fine to cool little silt, occasional cobbles. (Fill). Cobble from 3.4-3.9 ft bgs.	arse SAND.	
- 5 -	2D	24/13	5.00 - 7.00	8/5/6/24	11	15		205.00		Cobble from 4.5-4.8 ft bgs. Brown, damp, medium dense, fine to coarse SAN gravel, some silt. (Fill).	5.00- ND, some	G#237519 A-1-b, SM WC=8.0%
							75			Boulder from 7.2-8.6 ft bgs.		нс <i>-</i> 010 <i>и</i>
• 10 •	3D	24/5	10.00 -	6/2/8/9	10	14	37 18	200.50		Telescoped with NW Casing through Boulder at bas.	9.50- 10.0 ft	
			12:00				32	1		Grey, wet, medium dense, gravelly, fine to ca trace silt: gravel is coarse, broken rock fra	oarse SAND, agments,	
							34 81			Wood fibers and pieces in wash water from 12. bgs.	.5-13.5 ft	
							Q178	195.20		0178 blows for 0.8 ft.	14.80-	
• 15 -	R1	60/54	15.00 - 20.00	ROD = 27%			N0-2	195.0(- -		Top of Bedrock at Elev. 195.2 ft. Roller Coned ahead from 14.8-15.0 ft bgs. R1:Bedrock: Grey, fine grained, hard, fresh, metamorphic SILTSTONE and SLATE, finely lamin to massive siltstone beds, jointing along ber moderately dipping angles, second set verica 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	15.00 hated slate dding at I. Flume	
• 20 •								- 190.0(- - -		18.0-19.0 ft (3:25) " " " 19.0-20.0 ft (3:15) " " " 90% Recovery Bottom of Exploration at 20.00 feet below surface.	20.00- ground	
25												
<u>Remar</u>	<u>'ks:</u>											
Strati * Wate	fication	lines repr	esent approxim	nate boundaries between s	soil typ	es; tran	nsitions	may be g	radual.	Page 1 of 1		

	S	PRINCETON BRIDGE	PROJ. MANAGER	D. ANDERSON BY	DATE		STATE OF MAINE	
	H		DESIGN-DETAILED	L. KRUSINSKI T. WHITE				T T
(EE	GRAND FALLS FLOWAGE	CHECKED-REVIEWED	(SIGNATURE	DEPARIMENT OF IKANSPOKIATION	Z
∠ ⊃F	T		DESIGN2-DETAILED2					
-		FRINCEIUN WADMINGIUN CUUNII	DESIGN3-DETAILED3				V/000/3261 DU 70	
F	IU I		REVISIONS 1			P.E. NUMBER		
	MI		REVISIONS 2					
	3E	RORING I DGS	REVISIONS 3				PIN	
	R		REVISIONS 4				RRINGE NO 2688 12662 00 RRINGE PI AN	ANS
			FIELD CHANGES					

Appendix A

Boring Logs

	UNIFIE		ASSIFICA	TION SYSTEM		TERMS I DENSITY/	DESCRIBING	CY .
MA			GROUP					
COARSE- GRAINED	GRAVELS	CLEAN GRAVELS	GW	Well-graded gravels, gravel- sand mixtures, little or no fines	Coarse-grained s sieve): Includes (1 clayey or gravelly penetration resist:	soils (more than half of) l) clean gravels; (2) si sands. Consistency	of material is larger ilty or clayey gravel is rated according t	than No. 200 s; and (3) silty, o standard
GOILO	[:] of coarse : than No. 4 ze)	(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines	Descrip	Modified B <u>ative Term</u> race	urmister System <u>Port</u>	<u>ion of Total</u>)% - 10%
is Ze)	ire than half ion is larger sieve si	GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-sill mixtures.	s adjective (e.g	ittle ome . sandy, clayey)	1 2 3	1% - 20% 1% - 35% 6% - 50%
of material 00 sieve si	(mc fracti	(Appreciable amount of fines)	GC	Clayey gravels, gravel-sand-clay mixtures.	Der Cohesio Very	<u>nsity of</u> nless Soils / loose	Standard Per <u>N-Value</u>	netration Resistance (blows per foot) 0 - 4 5 - 10
e than half than No. 2	SANDS	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	Mediu De Very	m Dense ense Dense		11 - 30 31 - 50 > 50
(mor larger	of coarse than No. [∠]	(little or no fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.	Fine-grained soil	ls (more than half of n	naterial is smaller th	nan No. 20(
	than half o is smaller sieve size	SANDS WITH FINES	SM	Silty sands, sand-silt mixtures	sieve): Includes (² or silty clays; and strength as indica	 inorganic and orgar clayey silts. Constead. 	hic silts and clays; (sistency is rated acc	2) gravelly, sandy cording to sheai
	(more fraction	(Appreciable amount of fines)	SC	Clayey sands, sand-clay mixtures.	Consistency of Cohesive soils	SPT N-Value blows per foot	Undrained Shear Strength (psf)	<u>Field</u> Guidelines
	SILTS AN	ID CLAYS	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	Very Soft Soft Medium Stiff	WOH, WOR, WOP, <2 2 - 4 5 - 8	0 - 250 250 - 500 500 - 1000	Fist easily Penetrates Thumb easily penetrates Thumb penetrates with moderate effort
FINE- GRAINED SOILS	(liquid limit l	(liquid limit less than 50)		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Stiff Very Stiff Hard	9 - 15 16 - 30 >30	1000 - 2000 2000 - 4000 over 4000	Indented by thumb with great effort Indented by thumbnai Indented by thumbnail with difficulty
() ()	(14010 11111 1		OL	Organic silts and organic silty clays of low plasticity.	Rock Quality Des RQD =	signation (RQD): sum of the lengths	of intact pieces of	of core* > 100 mm
f of material is 200 sieve siz	SILTS AND CLAYS		МН	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	<u>Rock M</u>	*Minimum Correlation of RQI ass Quality	NŎ rock core (1. D to Rock Mass (88 in. OD of core) Quality <u>RQD</u>
re than half er than No.			СН	Inorganic clays of high plasticity, fat clays.	Ver F	y Poor Poor Fair Tood	2 5' 7	<25% 6% - 50% 1% - 75% 6% - 90%
(mo smallt	(liquid limit gr	eater than 50)	ОН	Organic clays of medium to high plasticity, organic silts	Exc Desired Rock C Color (Munsell	cellent D bservations: (in t color chart)	91 his order)	% - 100%
	HIGHLY (SO	ORGANIC ILS	Pt	Peat and other highly organic soils.	Texture (aphan Lithology (igned Hardness (very Weathering (fre	itic, fine-grained, et ous, sedimentary, m hard, hard, mod. h sh, very slight, sligl	c.) netamorphic, etc. ard, etc.) ht, moderate, mo) d. severe,
Desired So	il Observat	ions: (in th	is order)		1	severe, etc.)		
Color (Muns Moisture (d	sell color cha rv. damp. m	art) oist wet sa	turated)		Geologic discor	ntinuities/jointing: -dip (horiz - 0-5, lo	w angle - 5-35 m	nod dipping -
Density/Cor	nsistency (fr	om above ri	ght hand si	de)		35-55, steep	- 55-85, vertical	- 85-90)
Name (sand	d, silty sand	clay, etc., i	ncluding po	rtions - trace, little, etc.)		-spacing (very clos	se - <5 cm, close	- 5-30 cm, mod.
Plasticity (n	weii-graded, on-plastic. s	lightly plasti	iea, uniforn c, moderat	i, etc.) ely plastic, highly plastic)		ciose 30-100 cr -tightness (tight. or	n, wide - 1-3 m, v pen or healed)	/ery wide >3 m)
Structure (la	ayering, frac	tures, crack	s, etc.)			-infilling (grain size	e, color, etc.)	
Bonding (we	eil, moderat	ely, loosely, iderate or s	etc., it appl trong if app	ncable)	Formation (Wat	erville, Ellsworth, C	ape Elizabeth, e	tc.) r poor etc.)
Geologic Or Unified Soil Groundwate	rigin (till, ma Classification er level	rine clay, all on Designat	luvium, etc. ion)	ref: AASHTO 17th Ed. Tabl Recovery	Standard Specifica e 4.4.8.1.2A	ation for Highway	Bridges
	Maina	Jonartma	nt of Tra	nsportation	Sample Cont	ainer Labeling I	Requirements	<u>.</u>
	wanie I	Geotoch	nical Soc	nsportation tion	PIN Bridge Name	/ Town	Blow Counts Sample Reco	overv
Ka	v to Soil	and Rock	Deecrin	tions and Terme	Boring Numbe	er	Date	
	Fie	ld Identific	cation Info	prmation	Sample Numl Sample Dept	ber n	Personnel Ini	tials

I	Maine Department of Transport Soil/Rock Exploration Log US CUSTOMARY UNITS			of Transportat	tion	P	roject:	Prince	ton Bridge #2688	Boring No.:	BB-P	IT-101
		- - -	Soil/Rock Explo US CUSTOMA	oration Log RY UNITS		L	ocatio	over G n: Princ	rand Falls Flowage ceton, Maine	PIN:	126	52.00
Drille	r:		Maine Testbori	ng, Inc.	Elevat	ion (ft.)	209.	5	Auger ID/OD:	5" SSA	
Opera	ator:		B. Enos/C. Wo	rmley	Datum	n:		NAV	/D 88	Sampler:	Standard Split	Spoon
Logg	ed By:		G. Lidstone		Rig Ty	/pe:		Mob	ile B-47 Trailer	Hammer Wt./Fall:	140#/30"	
Date	Start/Fir	nish:	10/13/04-10/13	/04	Drillin	g Me	thod:	Case	d Wash Boring	Core Barrel:	NQ-2"	
Borin	g Locat	ion:	17+80.3, 14.1 f	it Rt.	Casin	g ID/0	DD:	HW		Water Level*:	9.0 ft bgs	
Definition D = Spl MD = U U = Thi R = Roo V = Insi SSA = 2	ons: lit Spoon Si Insuccessfi n Wall Tub ck Core Sa itu Vane Sh Solid Stem	ample ul Split Spor e Sample mple near Test Auger	on Sample attempt		$\begin{array}{c} \text{Definition}\\ S_{\text{U}} = \text{Ins}\\ T_{\text{V}} = \text{Pool}\\ q_{\text{p}} = \text{Unol}\\ S_{\text{U}}(\text{lab}) = \text{WOH} = \text{WOR} = $	ns: itu Fiel cket To confine = Lab \ weight weight	d Vane S prvane Sh ed Compre /ane She of 140lb. of rods V	Shear Stre near Stren ressive Str ear Strengt . hammer WOC = we	ngth (psf) gth (psf) ength (ksf) h (psf) sight of casing	Definitions: WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
			<u> </u>	Sample Information								Laboratory
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 Descri	ption and Remarks		Testing Results/ AASHTO and Unified Class.
0					2	SSA	209.35	5	PAVEMENT.		0 15	
- 5 -	1D 2D	24/7	5.00 - 7.00	25/24/24/21 54/87/75(4")	48	143 114 26 21 47	200.50		Brown, damp, dense, SAND, some g Dark brown, wet, medium to coarse (Fill). [Strong Petroleum Odor]. Brown, wet, silty fine to coarse SAN	gravel, little silt, (Fill). 	- — — — — 9.00 ilt, old pavement	G#182506 A-1-b, SM WC=5.1%
- 15 -	3D	24/13	15.00 - 17.00	32/44/43/33	87	71 77	194.50		Grey, damp, very dense, SAND, son Till). a100 blows for 0.4'.	ne silt , little clay, little grav	15.00- vel, (Reworked	G#182505 A-4, CL-ML WC=19.3%
					8	100	192.10	0112	Top of Bedrock at Flev 1921'		17.40	
- 20 -	R1 R2	54/52 60/60	18.00 - 22.50 22.50 - 27.50	RQD = 70%		NQ	-		Washed ahead to 18.0' bgs. R1:Bedrock: Grey, fine grained, SL. weathered to fresh, joint set along be unweathered surfaces. Flume Ridge 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, SL	ATE and SILTSTONE, hard eddirng at steep to vertical a Formation. Rock Mass Qua	d, slightly Ingles, Ility; Fair. ed. Joint sets	
25	ırks:								along foliation/bedding and horizont Quality: Good. R2:Core Times (min:sec)	al. Flume Ridge Formation	. Rock Mass	

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.	Page 1 of 2
* 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

Ι	Soil/Rock Exploration Log US CUSTOMARY UNITS `` Maine Testboring, Inc.			of Transporta	tion	L	Project:	Prince	ton Bridge #2688	Boring No.:	BB-P	IT-101
		-	Soil/Rock Exp	loration Log ARY UNITS			Location	over C 1: Prin	rand Falls Flowage ceton, Maine	PIN:	126	52.00
Drille	r:		Maine Testbor	ring, Inc.	Ele	vation	(ft.)	209.	5	Auger ID/OD:	5" SSA	
Oper	ator:		B. Enos/C. We	ormley	Dat	um:		NA	/D 88	Sampler:	Standard Split	Spoon
Logg	ed By:		G. Lidstone	-	Rig	Type:		Mot	ile B-47 Trailer	Hammer Wt./Fall:	140#/30"	
Date	Start/Fir	nish:	10/13/04-10/1	3/04	Dril	ling M	ethod:	Case	d Wash Boring	Core Barrel:	NQ-2"	
Borin	g Locati	ion:	17+80.3, 14.1	ft Rt.	Cas	sing ID	/OD:	HW		Water Level*:	9.0 ft bgs	
Definiti D = Sp MD = U U = Th R = Ro V = Ins SSA =	ons: lit Spoon Si Jnsuccessfi in Wall Tub ck Core Sa itu Vane Sh Solid Stem	ample ul Split Sp e Sample mple near Test Auger	con Sample attemp	ot	Defir S _u = T _V = q _p = S _{u(la} WOF	nitions: Insitu Fie Pocket 1 Unconfir ab) = Lab H = weigh R = weigh	eld Vane Sl orvane Sh ned Compre Vane Shea nt of 140lb. nt of rods V	hear Stre ear Stren essive Str ar Streng hammer VOC = w	ngth (psf) gth (psf) ength (ksf) h (psf) <u>sight of casing</u>	Definitions: WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
				Sample Information		<u> </u>	1					Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (pst) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Descr	iption and Remarks		Testing Results/ AASHTO and Unified Class.
25								919	22.5-23.5' (2:52) 23.5-24.5' (3:05)			
						$ \downarrow \rangle$	182.00		24.5-25.5' (3:11) 25.5-26.5' (3:08) 26.5-27.5' (3:15) 100% Recovery		27.50	
							182.00		Bottom of Exploration at	27.50 feet below ground s	urface.	
- 30 -							-					
							_					
							1					
							-					
							_					
- 35 -												
							-					
							1					
							-					
- 40 -												
40												
							-					
							1					
- 45 -							-					
							1					
							-					
							4					
							1					
50 Rema	arks:					L						

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.	Page 2 of 2
* 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

I	Main	Soil/Rock Exploration Log US CUSTOMARY UNITS				P	roject	Prince	ton Bridge #2688	Boring No.:	BB-P	IT-102
		<u>s</u>	oil/Rock Expl	oration Log			ocatio	over (rand Falls Flowage			-
		L	IS CUSTOMA	ARY UNITS		^L	.ocatio	n: rim	ceton, mame	PIN:	1266	52.00
Drille	er:]	Maine Testbor	ing, Inc.	Ele	vation (ft.)	195	0	Auger ID/OD:	N/A	
Oper	ator:]	B. Enos/C. Wo	ormley	Dat	um:	-	NA	/D 88	Sampler:	Standard Split	Spoon
Logg	ed By:		G. Lidstone		Rig	Type:		Mol	ile B-47 Trailer	Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	nish:	10/14/04-10/14	4/04	Dril	ling Me	thod:	Cas	d Wash Boring	Core Barrel:	NQ-2"	
Borir	ng Locat	ion:	18+53.3, 9.0 ft	Rt.	Cas	sing ID/0	DD:	NW		Water Level*:	At ground surfa	ace
Definiti	ons:	No. 10. 10.			Defir	nitions:				Definitions:	0	
D = Sp MD = U	Jint Spoon S Jnsuccessi	ample ful Split Spoo	n Sample attemp	t	S _u = T _v =	Pocket To	ld Vane S prvane St	hear Stre	ngth (psf) gth (psf)	WC = water content, percent LL = Liquid Limit		
U = Th R = Ro	in Wall Tub ock Core Sa	be Sample ample			q _p = Su(la	Unconfine	ed Compi √ane She	ressive St ear Strend	ength (ksf) h (psf)	PL = Plastic Limit PI = Plasticity Index		
V = Ins	situ Vane S Solid Sterr	hear Test			WOH	H = weight	of 140lb	. hammer WOC = w	and t of casing	G = Grain Size Analysis C = Consolidation Test		
		i ragoi		Sample Information		<u>t = troigitt</u>	011000		light of odding			
		n.)	oth									Laboratory Testing
Ē	No.	(<u>i</u>) 	Dep	6 in. (%)				Log	Visual Descri	ption and Remarks		Results/
th (f	ple	/Re	ple	As (/ Mgth QD	Ine	s,	atio	hic				AASHTO and
Dep	Sam	Gen	Sarr (ft.)	Shee Shee or R	P-V9	Casi		Grag				Unified Class.
0	1D/AB	24/4	0.00 - 2.00	24/41/32/13	73	22		6.0.0 0.004	(1D/A) 0.0-1.5' bgs.			
							102.5	0 0 00 0 0 000	Brown, wet, fine to coarse SAND, w probably loose due to higher blow co	ounts caused by wood.	gravel. Density	
						33	193.5		(1D/B) 1.5-2.0' bgs.		1.50-	
	R1	25.2/21.8	2.80 - 4.90	RQD = 0%		a35	192.4		Brown, wet, silty fine to coarse SAN	ID, trace gravel.		
						-NQ-		90	a35 blows for 0.6'.		2.60-	
							-		Top of Bedrock at Elev. 192.4'. Roller coned ahead from 2.6.2.8' bg	e.		
_	R2	21.6/21.6	4.90 - 6.70	RQD = 0%					R1:Bedrock: Grey, fine grained, SL	ATE, hard, slightly weather	red, very	
							1	912	fractured along steeply dipping bedo Ridge Formation Rock Mass Qualit	ling, surfaces fresh with no v: Very poor	residue. Flume	
							-	<i>Mall</i>	R1:Core Times (min:sec)	j. telj pooli		
	R3	34.8/31	6.70 - 9.60	RQD = 62%				al	2.8-3.8' (3:17) 3 8-4 8' (2:55)			
									4.8-4.9' (0:45) 87% Recovery			
							1	<u>N</u> M	R2:Bedrock: Same as R1, except hig	ghly fractured. Rock Mass (Quality: Very	
							4	- MA	R2:Core Times (min:sec)			
	R4	43.2/42	9.60 - 13.20	RQD = 9%				<u>a</u>	4.9-5.9' (2:50) 5.9-6.7' (2:38) 100% Recovery			
- 10 -							1		R3:Bedrock: Same as R1, except mo	oderately fractured. Rock M	lass Quality:	
							-		R3:Core Times (min:sec)			
								<u>N</u>	6.7-7.7' (2:03)			
								- MA	8.7-9.6' (3:05) 91% Recovery			
						$\vdash \mathbb{V}$	181.8	0 EUM	R4:Bedrock: Same as R1, with freque	ent quartz veins. Rock Ma	ss Quality: Very	
							4		R4:Core Times (min:sec)			
									9.6-10.6' (2:35)			
- 15 -							1		11.6-12.6' (3:20)			
							-		12.6-13.2' (2:10) 98% Recovery		13.20-	
									Bottom of Exploration at 1	13.20 feet below ground su	ırface.	
							1					
	<u> </u>						1					
							4					
							1					
- 20 -							1					
							-					
							1					
							-					
							1					
25												
Rema	arks:	1					1	1				
Con	crete Dec	k 0.85' thic	k.									
Тор	of Deck t	to water 8.7	'. Inface 16 4'									
rop	UI DECK I	to ground si	ace 10.4.									
L_												
Stratifi	cation lines	represent ap	proximate bound	laries between soil types; tran	sitions r	nay be gra	adual.			Page 1 of 1		

* 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.	Soring No.:	BB-PIT-102
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Ι	Soil/Rock Exploration Log US CUSTOMARY UNITS Maine Testboring, Inc.			ation	F	roject:	Prince	on Bridge #2688	Boring No.:	BB-P	IT-103	
		<u>S</u>	Soil/Rock Expl JS CUSTOMA	oration Log ARY UNITS		L	ocation	over G 1: Princ	rand Falls Flowage ceton, Maine	PIN:	1266	52.00
Drille	r:		Maine Testbori	ing. Inc.	Eleva	ation (ft.)	210.	1	Auger ID/OD:	5"	
Opera	ator:		B. Enos/C. Wo	ormley	Datu	m:		NAV	′D 88	Sampler:	Standard Split S	Spoon
Logg	ed By:		G. Lidstone		Rig T	Гуре:		Mob	ile B-47 Trailer	Hammer Wt./Fall:	140#/30"	•
Date	Start/Fin	nish:	10/13/04-10/13	3/04	Drilli	ng Me	thod:	Solic	l Stem Auger	Core Barrel:	N/A	
Borin	g Locati	ion:	19+35.2, 12.0 1	ft Rt.	Casi	ng ID/	DD:	N/A		Water Level*:	None Observed	
Definition D = Spl MD = U U = Thi R = Roo V = Insi SSA = 3	ons: lit Spoon Sa Jnsuccessfu in Wall Tub ck Core Sa itu Vane Sh <u>Solid Stem</u>	ample ul Split Spoo e Sample mple near Test Auger	n Sample attemp	t	Definiti S _u = Ir T _V = P q _p = U S _{u(lab} WOH = WOR =	ions: hsitu Fiel locket To hconfine) = Lab) = Weight = weight	d Vane Sl prvane She d Compre /ane Shea of 140lb. of rods W	hear Stren ear Streng essive Str ar Strengt hammer VOC = we	ngth (psf) yth (psf) ength (ksf) h (psf) ight of casing	Definitions: WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
		_		Sample Information			<u> </u>					Laboratory
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 Descri	ption and Remarks		Testing Results/ AASHTO and Unified Class.
0						SSA	209.75	****	PAVEMENT.			
									Brown, damp, sandy GRAVEL, trac	ce silt, (Fill).	0.55	
							207.60		Similar to above, but with cobbles,		2.50-	
						_	-					
- 5 -	1D	14.4/4	5.00 - 6.20	11/19/50(2.4")		\checkmark					c 20	
						-	203.90		Bottom of Exploration at	6.20 feet below ground su	o.20	
]		AUGER REFUSAL			
							-					
- 10 -							4					
							1					
							1					
							-					
- 15 -							4					
]					
							1					
							-					
- 20 -							-					
							1					
							1					
					+		-					
25												
Rema	arks:											
1. Ex	kisting Bri	idge Deck	elevation at Ab	outment 2.								
2. Sc	oil Descrip	ptions base	d on visual obs	servations and drill attit	ude.							

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 1 of 1
* 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						P	Project:	Prince	ton Bridge #2688	Boring No.:	BB-PI	Г-103А
Soil/Rock Exploration Log						L	ocation	over G 1: Prin	rand Falls Flowage ceton, Maine	PIN:	1266	52.00
Drille			Maine Teether	ing Inc	Fla	ation ((4)	210	2		5"	
Oper	ator:		B Enos/C Wo	ormley	Dati	im.		210. NAV	2 /D 88	Sampler:	5 Ν/Δ	
	ed By:		G Lidstone	onniey	Rig	Type:		Moh	ile B-47 Trailer	Hammer Wt /Fall	N/A	
Date	Start/Fi	nish:	10/13/04-10/12	3/04	Drill	ina Me	thod:	Solie	l Stem Auger	Core Barrel:	N/A	
Borir	Boring Location: 19+38.2, 12.0 ft Rt. Casing I					ing ID/0	DD:	N/A	6	Water Level*:	None Observed	
D = Spilt Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample R = Rock Core Sample V = Insitu Vane Shear Test <u>SSA = Solid Stem Auger</u>					Defin S _U = T _V = Q _p = S _U (la WOH WOR	itions: Insitu Fiel Pocket Tc Unconfine b) = Lab \ = weight = weight	d Vane Si prvane She ed Compre Vane Shea of 140lb. I of rods W	hear Stre ear Stren essive Str ar Streng hammer VOC = we	ngth (psf) gth (psf) ength (ksf) ih (psf) eight of casing	Definitions: WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
				Sample Information			1					Laboratory
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 Descri	ption and Remarks		Testing Results/ AASHTO and Unified Class.
0						SSA	209.85	****	PAVEMENT.			
							1		Brown, damp, sandy GRAVEL, trac	e silt, (Fill).		
							207.70		Similar to above, but with cobbles	— — — — — — — –	2.50-	
							-).		
- 5 -												
						<u> </u>						
						\mathbb{V}	202.10		Bottom of Exploration at	8.10 feet below ground su		
- 10 -									REFUSAL on Boulder			
10							-					
							-					
- 15 -							-					
							-					
20							-					
- 20 -												
							-					
25												
Rema	arks:		-									
1. xi 2. So	sting Brid bil descrij	lge Deck ptions bas	elevation at Abu ed on visual obs	utment 2. servations and drill attitude	2.							

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 1 of 1
* 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 Transportati					tion		Project	Prince	ton Bridge #2688	Boring No.:	BB-PI	T-103B
		•	Soil/Rock Expl US CUSTOM/	loration Log ARY UNITS			Locatio	over (n: Prin	rand Falls Flowage ceton, Maine	PIN:	1260	52.00
Drille	r-		Maine Testhor	ing Inc	FIO	vation	(ft)	210	4		5"	
Opera	ator:		B Enos/C Wo	ormlev	Dat	um:	(11.)	NA	+ /D 88	Sampler:	N/A	
Loga	ed Bv:		G. Lidstone	Jiniey	Ria	Type:		Mol	ile B-47 Trailer	Hammer Wt./Fall:	N/A	
Date	Start/Fi	nish:	10/13/04-10/13	3/04	Dril	lina M	ethod:	Soli	l Stem Auger	Core Barrel:	N/A	
Boring Location: 19+43.2, 12.0 ft Rt.				Cas	ina ID	/OD:	N/A		Water Level*:	None Observed	1	
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				Defin S _u = T _V = q _p = S _{u(la} WOF	itions: Insitu Fi Pocket ⁻ Unconfin b) = Lat I = weigt C = weigt	eld Vane S Forvane Sh ned Compro Vane She nt of 140lb nt of rods	Shear Stren near Strer ressive St ear Streng . hammer WOC = w	ngth (psf) gth (psf) ength (ksf) th (psf) eight of casing	Definitions: WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
				Sample Information			-					Laboratory
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 Descr	ption and Remarks		Testing Results/ AASHTO and Unified Class.
0						SSA	210.0	5	PAVEMENT.		0.25	
							_		Brown, damp, sandy GRAVEL, tra	ce silt, (Fill).		
							208.9		Similar to above, but with cobbles,	(Fill).	— — — —1.50	
							,					
- 5 - - 10 - - 15 -									Bottom of Exploration at REFUSAL on Boulder	3.70 feet below ground st	urface.	
25												
25 Rema	arks:	1	1	1		L	_					I
1. Ex	kisting Br	idge Decl	c elevation at At	butment 2.								

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

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 1 of 1
* 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

Ι	Maine Department of Transporta				tion	I	Project	Princet	on Bridge #2688	Boring No.:	BB-PI	T-103C
		<u></u>	Soil/Rock Expl	oration Log			_ocatio	over G n: Princ	rand Falls Flowage eton, Maine	PIN·	126	52.00
		<u>[</u>	JS CUSTOMA	RYUNIIS	1						1200	52.00
Drille	r:		Maine Testbori	ng, Inc.	Elev	ation	(ft.)	210.3	3	Auger ID/OD:	5"	
Opera	ator:		B. Enos/C. Wo	rmley	Datu	im: Turnou		NAV	D 88	Sampler:	Standard Split	Spoon
Logg	ea By:	ichi	G. Lidstone	/04	Rig	iype:	thod	Case	d Weeh Dering	Coro Porrol:	140#/30"	
Borin	Start/Fil	on:	10/15/04-10/14	/04 7 Dt	Casi				u wash Boring	Water Level*:	NQ-2	
Definiti	ons:	011.	19+43.2, 10.01	t Kt.	Definit	tions:	00.	11 **		Definitions:	8.0 ft bgs	
D = Sp MD = U U = Thi R = Ro V = Ins SSA =	lit Spoon Sa Insuccessfu in Wall Tub ck Core Sa itu Vane Sh Solid Stem	ample ul Split Spoo e Sample mple lear Test Auger	on Sample attemp		S _u = I T _v = F q _p = U S _u (lab WOH WOR	nsitu Fie Pocket T Jnconfin) = Lab = weigh = weigh						
				Sample Information			-					Laboratory
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 Descri	otion and Remarks		Testing Results/ AASHTO and Unified Class.
0						SSA	209.9	, ****	_ PAVEMENT.		0.35	
					-+	+	1		Brown, damp, sandy GRAVEL, trac	e silt, (Fill).		
							4					
							207.9		Similar to above, but with cobbles.	— — — — — — — – Fill).		
							1					
							-					
- 5 -							_					
							1					
							-					
							202.3				8 00-	
							202.5	' 🗱	Brown, wet, sandy GRAVEL, trace	silt, (Fill).	8.00	
						+ /	-					
- 10 -						V	-		(1D/A) 10 0-11 0' bas			
	1D/AB	24/13	10.00 - 12.00	28/67/26/43	93	68	100.2		(1D/11) 10.0-11.0 0gs.		11.00	
						90	199.3		(1D/B) 11.0-12.0' bgs.			G#182507
							-		Brown and grey, moist, SAND, som	e gravel, some silt, trace cl	ay. (Reworked	WC=31.3%
						130	_	100 BO	3200 blows for 0.0'			
						a200	106.4		"200 blows for 0.9.		12.00	
	R1	44.4/43	14.00 - 17.70	ROD = 62%		NO	196.4		Top of Bedrock at Elev. 196.4'.		13.90	
- 15 -							-		Roller coned ahead fron 13.9-14.0' b	gs.	14.00	
								860	R1:Bedrock: Grey, fine grained, inte	rbedded metamorphic SIL'	ISTONE and	
									fresh, highly foliated. Flume Ridge I	Formation. Rock Mass Qua	lity: Fair.	
	R2	60/56	17 70 - 22 70	ROD - 92%			1		R1: Core Times (min:sec)			
	112	00/00	17.70 22.70	RQD = 7270			-		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, Me	tamorphic SILTSTONE, fr	esh, some quartz	
- 20 -							1		R2:Core Times (min:sec)	Mass Quanty: Excellent		
						_	-	<i>6616</i>	17.7-18.7' (2:15)			
						λ			19.7-20.7' (2:40)			
						\mathbb{V}	187.6		20.7-21.7' (2:55) 21 7-22 7' (3:07) 93% Recovery			
							+	1	Bottom of Evaluation	22 70 foot holess means -	22.70-	
							4		DOUGH OF EXPLORATION AT	22.70 reet below ground st	n lace.	
25												
Rema	arks:			I								
Exis	ting Bridg	e Deck ele	evation at Abut	ment 2.								

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 1 of 1
* 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-103C

	Maine Department of Transporta						Project	Princ	ton Brid	lge #2688	Boring No.:	BB-P	IT-201
		· · · · ·	Soil/Rock Exp	loration Log			Locatio	over (Grand Fa	Ills Flowage		106	(2 00
			US CUSTOM	ARY UNITS			Localio	11. 1111	ceton, iv	lanc	PIN:	126	62.00
Drill	er:		MaineDOT		Eleva	ation	(ft.)	209	.5		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Giguere/Giles/	Daggett	Datu	m:		NA	VD 88	Sampler:	Standard Split	Spoon	
Log	ged By:		B. Wilder		Rig T	ype		CM	E 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	nish:	10/28/10; 07:3	30-10:30	Drilli	ng M	lethod:	Cas	ed Wash	Boring	Core Barrel:	NQ-2"	
Bori	ng Loca	tion:	17+85, 8.0 ft I	Lt.	Casi	ng ID	0/OD:	NW			Water Level [*] :	5.0 ft bgs.	
Ham Defini	tions:	ciency Fa	actor: 0.84	R = Rock C	Core Same	mer	Type:	Autom	atic⊠ S = Insi	Hydraulic tu Field Vane Shear Strength (psf)	Rope & Cathead Su(lab)	= Lab Vane Shear S	strength (psf)
D = S MD = U = TI MU = V = In MV =	plit Spoon S Unsuccess hin Wall Tu Unsuccess situ Vane S <u>Unsuccess</u>	Sample sful Split Spo be Sample sful Thin Wal Shear Test, <u>sful Insitu Va</u>	oon Sample attemp I Tube Sample att PP = Pocket Per ne Shear Test atte	SSA = Soli bt HSA = Holl RC = Rolle empt WOR = we betrometer WOR/C = \lambda mpt WO1P = W Sample Information	d Stem Au low Stem / r Cone light of 140 weight of re /eight of o	Jger Auger Olb. ha ods or <u>ne per</u>	mmer casing son		$T_V = Poc$ $q_p = Unc$ N-uncorr Hammer $N_{60} = SF$ $N_{60} = (H$	ket Torvane Shear Strength (psf) confined Compressive Strength (psf) ected = Raw field SPT N-value Efficiency Factor = Annual Calibrati PT N-uncorrected corrected for ham lammer Efficiency Factor/60%)*N-un	on Value PI = Pia mer efficiency G = Gra toorrected C = Con	rater content, percen juid Limit astic Limit isticity Index nin Size Analysis asolidation Test	t
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 De	scription and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	200 0		PAVEMENT.		0.70	
	1D	24/12	1.00 - 3.00	10/7/6/5	13	18		- 206.50		Brown, damp, medium dens (Fill)	e, gravelly, fine to coarse S	AND, some silt.	
- 5 -	2D	24/17	5.00 - 7.00	4/3/4/6	7	10		-		Light brown, damp, loose, fi (Fill).	ne to coarse SAND, some	gravel, some silt.	G#237518 A-1-b, SM
								- 201.50				— — — —8.00	wC=9.3%
- 10 -	3D	24/15	10.00 - 12.00	5/7/6/12	13	18	50 53	-		Grey, saturated, medium der trace wood fragments; grave granite rock fragments. (Fill)	use, gravelly, fine to coarse l is fine to coarse rounded	SAND, trace silt, to coarse angular	
							08	196.40		a_{30} blows for 0.1 ft			
	R1	60/60	13.10 - 18.10	RQD = 17%			a30		<u>M</u>	Top of Podrock at Elay 106	4 ft	13.10	
- 15 -								- 191.40	A CONTRACTOR OF A CONTRACT OF A CONTRACTACT OF A CONTRACTACT OF A CONTRACT OF A CONTRACTACT OF A CONTRACTACTACT OF A CONTRACTACT OF A CONTRACTACT OF A CONTRACTACT OF A CONTRACTACTACTACTACTACTACTACTACTACTACTACT	R1:Bedrock: Grey, fine grain hard, fresh, thinly laminated sets, one at 45 degrees along Formation. Rock Mass Qua 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	ed, interbedded SILTSTO in slate to massive siltston bedding, 2nd vertical. Flu lity: Very Poor.	NE and SLATE, e beds, two joint me Ridge	
								-		17.1-18.1 ft (4:45) " " 100% Recovery	n		
- 20 -								-		Bottom of Exploration	at 18.10 feet below groun	18.10- nd surface.	
								1					
								-					
25	arke												
Stratif	ication lines	s represent a	approximate boun	daries between soil types; tran	sitions ma	ay be g	gradual.	may occ	Ir due to c	conditions other	Page 1 of 1		

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

Maine Department of Transportation					ation		Project	Prince	eton Brid	lge #2688	Boring No.:	BB-P	IT-202
			Soil/Rock Expl US CUSTOM/	loration Log ARY UNITS			Locatio	over (n: Prir	Grand Fa iceton, N	lls Flowage Iaine	PIN: 12662.0		
Drill	er:		MaineDOT		Eleva	ation	(ft.)	210	.0		Auger ID/OD:	5" Solid Stem	
Ope	rator:		Giguere/Giles/	Daggett	Datu	m:		NA	VD 88		Sampler:	Standard Split	Spoon
Log	ged By:		B. Wilder		Rig	Гуре:	:	СМ	E 45C		Hammer Wt./Fall:	140#/30"	
Date	Start/Fi	nish:	10/28/10; 10:3	0-13:00	Drilli	ng M	lethod:	od: Cased Wash Boring Core Barrel: NQ-2"					
Bori	ng Loca	tion:	19+40, 8.0 ft L	.t.	Casi	ng ID)/OD:	HW	' & NW		Water Level*:	9.5 ft bgs.	
Ham	mer Effi	ciency Fa	actor: 0.84		Ham	mer [·]	Туре:	Autom	atic 🖂	Hydraulic 🗆	Rope & Cathead □		
Definitions: R = Rock Core S: D = Split Spoon Sample SSA = Solid Stem MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Ste U = Thin Wall Tube Sample RC = Roller Cone MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of							mmer casing son		$\begin{array}{l} S_u = Ins\\ T_v = Poo\\ q_p = Uno\\ N-uncorri\\ Hammer\\ N_{60} = Si\\ N_{60} = (H) \end{array}$	tu Field Vane Shear Strength (psf) ket Torvane Shear Strength (psf) confined Compressive Strength (ksf) ected = Raw field SPT N-value Efficiency Factor = Annual Calibrat PT N-uncorrected corrected for ham lammer Efficiency Factor/60%)*N-ur	Su(lab) WC = w UL = Lic PL = Pic on Value PI = Pic mer efficiency G = Gra ncorrected C = Corr	= Lab Vane Shear S ater content, percen juid Limit astic Limit sticity Index in Size Analysis ssolidation Test	trength (psf) t
				Sample Information	_				-				Laboratory
Depth (ft.)	Pepth (ft.) Sample No. Pen./Rec. (in.) Sample Depth (ft.) Strength (psf) or RQD (%) N-uncorrected					N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual De	scription and Remarks		Testing Results/ AASHTO and Unified Class.
0							SSA	209 30)	PAVEMENT.			
	1D	8.4/8	1.50 - 2.20	14/50(2.4")						Brown, dry, very dense, grav occasional cobbles. (Fill).	velly, fine to coarse SAND,	little silt,	
										Cobble from 4.5-4.8 ft bgs.			
- 5 -	2D	24/13	5.00 - 7.00	8/5/6/24	11	15		- 205.00		Brown, damp, medium dense silt. (Fill).	e, fine to coarse SAND, so	5.00 me gravel, some	G#237519 A-1-b, SM WC=8.0%
										Boulder from 7.2-8.6 ft bgs.			
							75						
- 10 -							37	200.50			there h Devider at 10.0 ft	9.50	
	3D	24/5	10.00 - 12.00	6/2/8/9	10	14	18			Grey, wet, medium dense, gr gravel is coarse, broken rock	ravelly, fine to coarse SAN fragments.	D, trace silt;	
							32	-		Wood fibers and pieces in w	ash watar from 12.5.12.5 f	thas	
							81			wood fibers and pieces in w	asii watei 11011 12.5-13.5 1	l bgs.	
- 15 -							a178	195.20		a178 blows for 0.8 ft.	2.5	14.80	-
	R1	60/54	15.00 - 20.00	RQD = 27%			NQ-2	195.00		Roller Coned ahead from 14	.2 ft. .8-15.0 ft bgs.	15.00	
										R1:Bedrock: Grey, fine grain and SLATE, finely laminated along bedding at moderately	ned, hard, fresh, metamorpl d slate to massive siltstone dipping angles, second set	beds, jointing verical. Flume	
1								1		Ridge Formation. Rock Mas	s Quality: Poor.		
I I	L							4		15.0-16.0 ft (3:15)			
									9120	16.0-17.0 ft (3:50) No Water	Return		
- 20 -								- 190.00) 28.912	17.0-18.0 ft (3:50) 18.0-19.0 ft (3:25) " " 19.0-20.0 ft (3:15) " "			
										90% Recovery		20.00	
										Bottom of Exploration	at 20.00 feet below groun	id surface.	
1							+	-					
25													
<u>Rem</u> Stratif	ication lines	s represent a	approximate bound	daries between soil types; tra	insitions ma	ay be ç	gradual.				Page 1 of 1		
* Wate	er level read	dings have t	een made at time	s and under conditions state	d. Ground	water f	luctuations	may occi	ur due to o	conditions other	Deviner N.	מידים ממ	02
than	those pres	ent at the ti	me measurements	were made.				,		- -	Boring No.:	вв-ргг-2	02

Maine Department of Transporta				ation	tion Project: Princeton Bridge #2688					lge #2688	Boring No.:	BB-P	IT-203	
		<u>-</u>	Soil/Rock Exp US CUSTOM	loration Log ARY UNITS		over Grand Falls Flowage Location: Princeton, Maine					lls Flowage Iaine	PIN:	126	52.00
Drill	er:		MaineDOT		Eleva	tion	(ft.)		197.9)		Auger ID/OD:	N/A	
Ope	rator:		Giguere/Giles	/Daggett	Datur	n:	()		NAV	D 88		Sampler:	Standard Split	Spoon
Log	ged By:		B. Wilder		Rig T	vpe:			CME	45C		Hammer Wt./Fall:	140#/30"	.1
Date	Start/Fi	inish:	10/28/10; 13:0	00-15:30	Drillir	ng Me	ethod:		Case	d Wash	Boring	Core Barrel:	NQ-2"	
Boring Location: 18+63, 7.5 ft Lt. Cas				Casin	g ID/	/OD:		NW		-	Water Level*:	Water Boring		
Ham	mer Effi	iciency Fa	octor: 0.84		Hamn	ner T	ype:	Au	itoma	tic⊠	Hydraulic 🗆	Rope & Cathead 🗆		
Defin D = S MD = U = T MU = V = Ir MV =	itions: plit Spoon Unsuccess hin Wall Tu Unsuccess situ Vane <u>Unsuccess</u>	Sample sful Split Spoo be Sample sful Thin Wall Shear Test, sful Insitu Var	on Sample attem Tube Sample att PP = Pocket Per he Shear Test atte	R = Rock SSA = Sc pt HSA = HC RC = Rol kempt WOH = w ventor WOR/C = ampt WO1P = 1 Scample Information Scample Information	Core Sampl blid Stem Au blow Stem A ler Cone reight of 140 weight of ro Weight of on	le ger luger lb. har ids or ie pers	nmer casing son		2 7 1 1 1 1 1	$S_u = Ins$ $\Gamma_v = Poole q_p = Unole N-uncorrest Hammer N60 = S N60 = (H$	tu Field Vane Shear Strength (psf) ket Torvane Shear Strength (psf) confined Compressive Strength (ksf) ected = Raw field SPT N-value Efficiency Factor = Annual Calibrati PT N-uncorrected corrected for ham lammer Efficiency Factor/60%)*N-ur	Sul WC UL PL ion Value PI mer efficiency G = ncorrected C =	(tab) = Lab Vane Shear S = water content, percen = Liquid Limit = Plastic Limit = Plasticity Index = Grain Size Analysis = Consolidation Test	trength (psf) t
		<u>.</u>	£		g									Laboratory
Depth (ft.)	Sample No.	Pen./Rec. (in	Sample Dept (ft.)	Blows (/6 in.) Shear Strength (psť) or RQD (%)	N-uncorrecte	N60	Casing Blows	Elevation	(ft.)	Graphic Log	Visual De	scription and Remark	٢S	AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	4/1/1/7	2	3	SPUN CASE	1 2			Grey, saturated, very loose, s (Reworked Alluvium).	gravelly, fine to coarse	SAND, trace silt.	
	2D	24/12	2.00 - 4.00	5/10/13/7	23	32		_			Grey, saturated, medium der dark brown/grey silty fine sa fragments.	nse, gravelly coarse Sar and, odor of "cut back a	nd, little pockets of asphalt"; gravel is slate	
- 5														
	R1	32.4/32.4	5.30 - 8.00	RQD = 0%			NQ-2	19	92.60		R1: Weathered bedrock frag Fractured Bedrock, 5.3-7.3 f	ments, trace sand and a	5.30 silt. Cobbles or	
									90.60 39.90		5.3-6.3 ft (2:20) 6.3-7.3 ft (6:45)			
	R2	48/48	8.00 - 12.00	RQD = 17%				-			R1:Weathered BEDROCK. 7.3-8.0 ft (9:75) 100% Record Core Blocked	very	8.00	
- 10								_			Top of Intack Bedrock at Ele R2:Bedrock: Grey, fine grain along bedding at 1/4 to 1/2 in	ev. 189.9 ft. ned, moderately hard, S nch spacing, silt covere	SLATE, fractured	
								- 18 	35.90		to grey, fine grained, hard, n veins, slighty weathered, ma Quality: Very Poor. R2:Core Times (min:sec) 8.0-9.0 ft (3:45) No Water R 9.0-10.0 ft (6:25) " " "	etamorphosed Siltston ssive. Flume Ridge Fo eturn	e, occassional calcite rmation. Rock Mass	
- 15											10.0-11.0 ft (3.25) 11.0-12.0 ft (4:25) " " 100% Recovery Core Blocked			
											Bottom of Exploration	at 12.00 feet below g	round surface.	
20								_						
- 20														
								1						
25 Ren	l narks:	1					l							l
12. 1.0	2 ft from ft Concre	Bridge Dec te Bridge D	k to Ground. Deck.											

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.	Page 1 of 1
* 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

<u>Appendix B</u>

Laboratory Test Results

State of Maine - Department of Transportation Laboratory Testing Summary Sheet

Town(s):	Prince	eton			Proje	ect	Nur	nbo	er: 12	662.	00
Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.	W.C.	L.L.	P.I.	Cla	ssificatio	n
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet				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	
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	
BB-PIT-103C, 1D/B	19+43.2	10.0 Rt.	11.0-12.0	182507	1	31.3			SC-SM	A-2-4	
Classification of th	nese soil sam	ples is in a	ccordance wit	h AASHTO C	lassificati	on Sys	tem M·	-145-4	0. This cla	ssificatio	n
is followed by the	"Frost Susce	ptibility Ra	ting" from zer	o (non-frost s	susceptibl	e) to Cl	ass IV	(high	ly frost su	sceptible).
The "Frost S	usceptibility I	Rating" is h	based upon th	e MDOT and	Corps of E	ngine	ers Cla	ssific	ation Syst	ems.	
GSDC = Grain Size Distrib	oution Curve as	s determine	d by AASHTO	т 88-93 (1996	and/or A	STM D	422-63	(Rea	pproved 19	998)	

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

SHEET 1

PIN 12662.00	Town	idian Twp Res	Reported by/Date	/HITE, TERRY A 11/12/2010	
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↓ BB-PIT-101/1D 17+80.3 14.1 RT 5.0-7.0 SAND, some gravel, little sitt. 5.1 ◆ BB-PIT-101/3D 17+80.3 14.1 RT 15.0-17.0 SAND, some gravel, little clay, little gravel. 19.3 ● BB-PIT-201/2D 17+85 8.0 LT 5.0-7.0 SAND, some gravel, some sitt, little clay, little gravel. 19.3 ● BB-PIT-201/2D 17+85 8.0 LT 5.0-7.0 SAND, some gravel, some sitt. 9.5 ● BB-PIT-202/2D 19+40 8.0 LT 5.0-7.0 SAND, some gravel, some sitt. 8.0 ▲ BB-PIT-103C/1D(B) 19+43.2 10.0 RT 11.0-12.0 SAND, some gravel, some sitt. 8.0		Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, % LL	٦d	Ы
◆ 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-201/2D 17+86 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. 8.0	÷	BB-PIT-101/1D	17+80.3	14.1 RT	5.0-7.0	SAND, some gravel, little silt.	5.1		
■ BB-PIT-201/2D 17+85 8.0 LT 5.0-7.0 SAND, some gravel, some sitt. 9.5 ● BB-PIT-202/2D 19+40 8.0 LT 5.0-7.0 SAND, some gravel, some sitt. 8.0 ▲ BB-PIT-103C/1D(B) 19+43.2 10.0 RT 11.0-12.0 SAND, some gravel, some sitt, trace clay. 31.3	•	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-202/2D 19+40 8.0 LT 5.0-7.0 SAND, some gravel, some sitt. 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 ★ 10.0 RT 11.0-12.0 SAND, some gravel, some silt, trace clay. 31.3		BB-PIT-201/2D	17+85	8.0 LT	5.0-7.0	SAND, some gravel, some silt.	9.5		
▲ BB-PIT-103C/1D(B) 19+43.2 10.0 RT 11.0-12.0 SAND, some gravel, some silt, trace clay. 31.3 ★ 31.3	•	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		
	×								



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

 $q_{\text{factored}} \coloneqq \phi_r \cdot q_{\text{nominal}}$

 $q_{factored} = 16 \cdot ksf$

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 + $\frac{1}{2}$) in which qu, c and $\frac{1}{2}$ are rock mass properties.

$\phi_{\text{rock}} \coloneqq 20 \cdot \text{deg}$	Tomlinson, Page 139, Wyllie, phi for low friction rock, schists 20-27
q _{uc} := 9200∙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·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·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

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

 $q_{nominal} = 33 \cdot ksf$

Factored Bearing Resistance

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

 $\phi_{bc} \coloneqq 0.45$

 $q_{factored} \coloneqq q_{nominal} \cdot \phi_{bc}$

 $q_{factored} = 15 \cdot ksf$

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

$$N_{q} := \tan\left(45 \cdot \deg + \frac{\Phi_{rock}}{2}\right)^{6}$$

$$N_{q} = 8.485$$

$$N_{c} := 5 \cdot \tan\left(45 \cdot \deg + \frac{\Phi_{rock}}{2}\right)^{4}$$

$$N_{c} = 20.8$$

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

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

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

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

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%

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

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

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

Factored Bearing Resistance

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

			(21)	١	
h · 0.45			21	Irof	
$\phi_{bc} := 0.43$	$q_{\text{factored}} := q_{\text{nominal}} \cdot \phi_{\text{bc}}$	$q_{factored} =$	21	· KSI	
			22)	

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

 $q_{nominal} \coloneqq 16 \cdot ksf$

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 + $\frac{1}{2}$) in which qu, c and $\frac{1}{2}$ are rock mass properties.

$\phi_{\text{rock}} \coloneqq 20 \cdot \text{deg}$	Tomlinson, Page 139, Wyllie, phi for low friction rock, schists 20-27
q _{uc} := 1400∙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·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·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

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

 $\phi_{bc} \coloneqq 0.45$

 $q_{factored} \coloneqq q_{nominal} \cdot \phi_{bc}$

 $q_{factored} = 15 \cdot ksf$

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

$$N_{q} := \tan\left(45 \cdot \deg + \frac{\Phi_{rock}}{2}\right)^{6}$$

$$N_{q} = 8.485$$

$$N_{c} := 5 \cdot \tan\left(45 \cdot \deg + \frac{\Phi_{rock}}{2}\right)^{4}$$

$$N_{c} = 20.8$$

$$N_{\gamma} \coloneqq N_q + 1 \qquad \qquad N_{\gamma} = 9.485$$

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

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

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

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%

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

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

Factored Bearing Resistance

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

			(10)	
0.45	a i- a b		10	Irot
$\phi_{bc} = 0.43$	$q_{factored} = q_{nominal} q_{bc}$	$q_{factored} =$	10	·KSI
			10)

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



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** Remove units - report in ft $H_1 := 3$ Effective overburden stress at midpoint of layer $\sigma'_{v1} := \frac{H_1}{2} \cdot \rho_{dry} \qquad \qquad \sigma'_{v1} = 177$ Spring constant K := 90 Unitless $G_1 = 1 \times 10^6$ $G_1 := 1000 \cdot K \cdot \sqrt{\sigma'_{v1}}$ Shear Modulus Determination of Shear Velociy based on Bowles Eq. 20.15 $\frac{G_1}{\rho_{dry}}$ $V_{s_1} = 100.733$ in ft/sec Ratio of di / Vsi $\frac{\mathrm{H}_{1}}{\mathrm{V}_{\mathrm{s}_1}} = 0.03$ Layer 2 groundwater 2 feet into the 5 feet Thickess Layer $H_2 := 5$



Princeton Princeton Bridge Determination of Site Class Sheet 4 of 4 By: L. Krusinski Date: Jan 3 2011 Checked by: MJM 1/12/2011

Determination of Shear Velociy based on Bowles Eq. 20.14

$$V_{s_3} := \sqrt{\frac{G_3}{\rho_{dry}}}$$
 $V_{s_3} = 154.154$ in ft/sec

Ratio of di / Vsi

$$\frac{H_3}{V_{s_3}} = 0.032$$

Layer 4 - Bedrock - Interbedded Slate and Siltstone

$$H_4 := 100 - (H_1 + H_2 + H_3) \quad H_4 = 87$$

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

Sa (sec) (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