MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

PERKINS BRIDGE
HERRICK ROAD OVER LITTLE RIVER
BELFAST, MAINE

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Soils Report No. 2010-02
Bridge No. 5143

Fed No. BH-1668(500)X
January 28, 2010

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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 Perkins Bridge which is located on Herrick Road and spans the Little River in Belfast, Maine. The existing bridge is a single span, concrete T-beam superstructure on a combination of mass concrete gravity abutments and concrete-jacketed stone abutments. The Maine Department of Transportation (MaineDOT) has selected the Perkins Bridge site as a location to install a 50-foot span composite tubular arch bridge. Currently, it is proposed that the 50-foot span replacement structure will be founded on reinforced concrete footings founded directly on bedrock or concrete seals on bedrock. The following design recommendations are discussed in detail in this report:

Arch and Wingwall Spread Footings – Arch footings shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches. Footings shall be designed for all relevant strength, service and extreme limit states.

The design of arch spread footings and wingwall footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), lateral sliding and reinforced-concrete structural failure. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of cast-in-place spread footings founded on bedrock or seal concrete. Sliding calculations shall assume maximum frictional coefficients of 0.70 at level bedrock-concrete interfaces and 0.60 at arch footings-seal concrete interfaces.

For arch 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.

Assuming the arch footings are to be prevented from movement, earth pressures on footings resisting lateral arch thrust loads shall be calculated using an at rest earth pressure coefficient, K_o, of 0.47. A resistance factor for at rest earth pressures mobilized to resist lateral sliding forces is not specified in LRFD, therefore use the resistance factor for passive pressure, φ_{ep}, of 0.50. For designing the arch footing reinforcing steel for at rest earth pressure, a maximum load factor, γ_{EH}, of 1.50 is recommended.

Independent wingwalls shall be designed as free to rotate at the top in a state of active earth pressure. An active earth pressure coefficient, K_a, of 0.31, is recommended.

Additional lateral earth pressure due to construction surcharge and/or live load surcharge is required for the arch foundations and wingwalls.

All arch foundation designs and wingwall designs shall include a drainage system behind the arch and walls to intercept any groundwater.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

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

In no instance shall the bearing stress exceed the nominal 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.

Precast Concrete Modular Gravity (PCMG) Retaining Walls - PCMG walls founded on bedrock or fill soils may be used to retain approach fills above the ordinary high water (Q1.1) elevation. Should PCMG walls be used below Q1.1, the design flow velocity should be low and the potential for severe ice or wave action should be minimal. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for PCMG walls founded directly on bedrock shall be investigated at the strength limit state using a factored bearing resistance of 16 ksf and at the service limit state using a factored bearing resistance of 20 ksf. The bearing resistance for PCMG walls founded on granular bedding material on bedrock, concrete fill or fill soils shall be investigated at the strength limit state using a factored bearing resistance of 5 ksf for wall system bases 8 feet wide or less and 6.5 ksf for wall bases from 10 to 14 feet wide. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state.

Scour and Riprap - The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. For scour protection, bridge approach slopes and slopes at arch footings should be armored with 3 feet of riprap. The riprap should be underlain by a 1 foot thick layer of bedding material and Class 1 nonwoven erosion control geotextile.

Settlement - The grades of existing bridge approaches will be raised approximately 2.5 feet. The bridge approach subgrades are predominantly bedrock, therefore we anticipate post-construction settlement of the approach fills will be negligible.

Any settlement of bridge arch foundations bearing on bedrock will be due to elastic compression of the bedrock and is anticipated 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 6.0 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.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

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

Construction Considerations – Construction of the foundation seals, arch footings and wingwalls will require soil and rock excavation and removal of the existing bridge substructures. Cofferdams and temporary lateral earth support systems will be required to permit arch footing and wingwall construction in the dry. Preparation of the bedrock subgrade for arch footings or wingwall footings 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 composite arch and wingwall foundations. This does not apply to concrete seals placed underwater.

Excavation of bedrock may be conducted using conventional equipment, but may require drilling and blasting methods. 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 the upstream dam, nearby residences and bridge structures in accordance with industry standards at the time of the blast.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Perkins Bridge which spans the Little River in Belfast, Maine, and carries Herrick Road. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for substructure design.

Perkins Bridge was built in the 1921 and is a 28-foot, single span, concrete T-beam bridge. The abutments are mass concrete gravity abutments in combination with concrete-jacketed stone masonry sections. The wingwalls consist of dry laid field stone and rough hewn stone. Portions of the wingwalls show bulging; several stones are dislodged or have shifted. The bridge is directly downstream of a dam owned by the Belfast Water District, which holds back Belfast Reservoir No. 2.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports from 2009 assign the substructures a condition rating of 4 – poor and indicate a Bridge Sufficiency Rating of 51.2.

The MaineDOT Bridge Program has selected the Perkins Bridge site as a location to install a rigidified, concrete-filled composite tubular arch bridge structure developed by the University of Maine's Advanced Engineering Wood Composites (AEWC) Center in Orono, Maine. The carbon fiber tubes are inflated and infused with resin. After hardening, the tubes are transported to the bridge site, and lowered into place and filled with concrete. The proposed arch structure will have a span length of approximately 50 feet and will be founded on spread footings founded directly on bedrock or on seal concrete founded on bedrock. The superstructure curb-to-curb width will be increased from 20 feet to 30 feet. The proposed bridge alignment will closely match into the existing and accommodate the wider roadway section. The vertical grade will be raised approximately 2.5 feet at the bridge.

2.0 GEOLOGIC SETTING

Perkins Bridge on Herrick Road in Belfast, Maine crosses the Little River as shown on Sheet 1 - Location Map, presented at the end of this report. The bridge is directly below the dam holding back Belfast Reservoir No. 2.

The Maine Geologic Survey (MGS) Surficial Geology of Belfast Quadrangle, Maine, Open-file No. 86-7 (1986) indicates the surficial soils in the vicinity of the site consist of glacial marine deposits.

Glacial marine deposits, also known as the Presumpscot Formation, are commonly a clayey silt, but sand is also abundant at the surface in some areas. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of the lowland areas in southern and coastal Maine.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the bridge site as the Penobscot Formation and consists of metamorphic, sulfidic/carbonaceous pelite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling six (6) test borings. Four (4) borings were terminated with bedrock cores. Test borings BB-BLR-101 and BB-BLR-102 were drilled at the proposed location of Arch Footing No. 1. Test borings BB-BLR-103, BB-BLR-103A, BB-BLR-104 and BB-BLR-104A were drilled at the location of proposed Arch Footing No. 2.

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 between December 14 and 21, 2009 by Maine Test Borings (MTB) of Brewer, Maine. 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.

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 with a rope and cathead hammer system. 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 bedrock was cored in four (4) 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. A GZA GeoEnvironmental, Inc. geotechnical engineer logged the subsurface conditions encountered. The borings were located in the field by taping to site features and also by a MaineDOT Survey Crew after completion of the drilling program.

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 four (4) standard grain size analyses, three (3) grain size analyses with hydrometer, seven (7) natural water content tests, and two (2) Atterberg Limits test. 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 and on Sheets 3 and 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 glacial till, all underlain by metasedimentary bedrock. Cobbles and boulders were frequently encountered in the fill and glacial till deposits. 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. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Fill

A layer of fill was encountered in all of the borings. The encountered fill layer is approximately 5.0 to 17.7 feet thick. The fill soils generally consisted of brown, fine to coarse sand, with little to some gravel, trace to some silt; sand and gravel, trace silt; and brown-grey, mottled silt, with some clay, trace fine sand, gravel, organic fibers and roots. Cobbles and boulders were frequently encountered throughout the fill layer.

At boring BB-BLR-104, a fill layer of boulders, metasandstone blocks and granite blocks was encountered and cored. The layer is directly underlain by bedrock. The encountered thickness of the boulders and block fill layer was approximately 6.7 feet thick.

SPT N-values in granular fill ranged from 10 to 33 blows per foot (bpf) indicating that the fill is loose to dense in consistency. SPT N-values in the silt fill unit ranged from 4 to 21 bpf indicating that the silt is soft to very stiff in consistency.

Two borings, BB-BLR-103 and BB-BLR-104A, were abandoned and relocated because of obstructions in the fill.

Four grain size analysis resulted in the soil being classified as A-1-b, A-2-4 and A-4 under the AASHTO Soil Classification System and SM and ML under the Unified Soil Classification System (USCS). The measured water contents of the samples tested ranged from approximately 5 percent to 29 percent. One (1) Atterberg Limits test on a sample from the silt fill subunit determined a moisture content of approximately 29 percent and a plastic limit of 23. The natural water content slightly exceeded the liquid limit and the calculated liquidity index for the soil tested was greater than 1.0. Therefore, this soil has the potential to be remolded and transformed into a viscous, flowable form if disturbed by construction activity.

5.2 Reworked Glacial Till and Glacial Till

A deposit of reworked glacial till and glacial till was encountered below the fill unit in three (3) borings. The encountered thickness is approximately 24.5 to 38.9 feet at proposed Arch Footing No. 1 and approximately 7.5 feet at proposed Arch Footing No. 2. The reworked till and glacial till deposits encountered have a high portion of rock fragments, weathered rock fragments, cobbles and boulders. The reworked till subunits consisted of brown/grey, sand and silt, some gravel; grey silt, some sand, little gravel, trace wood fragments; and brown/grey, moist, mottled silt, some clay, trace sand and gravel. The till deposit encountered generally consisted of brown, sand and silt, with rock fragments; grey-brown, mottled silt, little to trace gravel, trace weathered rock fragments; and brown sand, some gravel, some silt, trace of clay and little rock fragments.

SPT N-values in glacial till were 9 bpf in silt subunits and 21 to 28 bpf in granular subunits, indicating soils that are stiff and medium dense in consistency. Attempts to conduct SPT tests at twelve (12) other sampling depths were abandoned after 100 hammer blows with 2 inches of penetration, which infers a high portion of large aggregate, rock fragments and cobbles in the glacial till soil matrix.

Three (3) grain size analyses of the reworked till and till resulted in the soil being classified as A-4 and A-6 under the AASHTO Soil Classification System and SC-SM, CL and ML under the USCS. The measured water contents of the samples tested ranged from approximately 11 percent to 28 percent. One (1) Atterberg Limits test on a sample from the reworked glacial till determined a moisture content of approximately 19 percent and a plastic limit of 19. The natural water content was less than the liquid limit and the calculated liquidity index (LI) for the soil tested was less than 1.0. Therefore, the glacial silt deposit is heavily preconsolidated.

5.3 Bedrock

Bedrock at the site was encountered and cored at depths ranging from approximately 17.7 feet below ground surface (bgs) and approximate Elevation 39.9 feet in boring BB-BLR-104 to a depth of approximately 43.9 feet bgs and approximate Elevation 15.1 feet in boring BB-BLR-102.

The bedrock at the site is identified as grey, fine grained to aphanitic, phyllite, soft to hard, fresh to slightly weathered, and grey, fine grained, metasandstone, hard, fresh to slightly weathered. The RQD of the bedrock was determined to range from 7 to 88 percent, correlating to a rock mass quality of very poor to good.

Table 1 below summarizes depths and corresponding elevations of the top of bedrock at the four borings in which bedrock was encountered.

Proposed Substructure	Boring	Station	Offset	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
Arch Footing No. 1	BB-BLR-101	5+24.3	9.3 Lt	35.0	23.8
Arch Footing No. 1	BB-BLR-102	5+29.3	4.1 Rt.	43.9	15.1
Arch Footing No. 2	BB-BLR-103A	5+79.5	8.2 Lt.	22.5	34.7
Arch Footing No. 2	BB-BLR-104	5+87	4.0 Lt.	17.7	39.9

Table 1. Summary of Approximate Bedrock Elevations

5.4 Groundwater

No free groundwater was encountered in the borings at the time of drilling or when re-checked the following day. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The MaineDOT Bridge Program has selected the Perkins Bridge site as a location to install a rigidified, inflatable, composite tubular arch bridge structure developed by the University of Maine's AEWC Advanced Structures & Composites Center in Orono, Maine. AEWC's tubular arches are made of Fiber Reinforced Polymer (FRP) composite materials. The carbon fiber tubes are inflated off-site and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the tubular structure.

Due to the presence of bedrock close to the streambed elevation at proposed Arch Footing No. 2, the most effective foundations types are:

- spread footings founded directly on bedrock, or
- spread footings founded on concrete seals cast on bedrock.

At the proposed Arch Footing No. 1, bedrock was encountered at greater depths. The following foundation alternatives may be considered for proposed Arch Footing No. 1:

- spread footings founded on moderately deep seals cast on bedrock,
- driven H-piles or pipe piles, some with short rock-sockets to achieve fixity,
- drilled shafts.

For the purposes of this report it is assumed that spread footings cast directly on bedrock, or on seals cast on bedrock, will be used to support the proposed composite arch bridge. Design recommendations for this selected foundation alternative are discussed in detail in Section 7.0 - Geotechnical Design Recommendations. If during final design it is determined that a deep foundation (piles or shafts) is necessary at Arch Footing No. 1, geotechnical recommendations for piles or drilled shafts will be developed and provided to the designer.

Design recommendations are also provided for independent cantilever-type retaining walls and Prefabricated Concrete Modular Gravity (PCMG) walls, which will be required to support the bridge approaches.

The design of the FRP tubular arches and associated headwalls is the responsibility of the AEWC and will be supplied to the designer and Contractor prior to construction of the structure.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 General - Spread Footings on Bedrock

Bedrock was encountered at depths of approximately 18 to 44 feet below the roadway at proposed arch footing locations. It is therefore considered feasible that spread footings and spread footing on deep seals could be constructed to bear on bedrock. The excavations will require cofferdams and temporary soil support systems.

The borings indicate that suitable bedrock with an average RQD of approximately 50 percent will be encountered at the bedrock surface, however, the bedrock surface shall be cleared of all loose bedrock and loose, decomposed bedrock. Based on borings conducted at the site and top of bedrock elevation encountered in those borings, the bottom of footing or seal elevations are estimated to vary from approximately Elev. 15.1 to 23.8 feet at Arch Footing No. 1 and approximately Elev. 34.7 to 39.9 feet at Arch Footing No. 2.

7.2 Arch Footing and Wingwall Spread Footing Design

Arch spread footings and wingwall spread footings shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, with 2008 and 2009 interims (LRFD) Articles 3.4.1, 11.5.5., and 12.5. Arch spread footings shall be designed to resist all lateral earth loads, vehicular loads, arch dead and loads, and lateral thrust forces transferred through the bridge arches. The design of arch and wingwall spread footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), lateral sliding and reinforced-concrete structural design.

Spread footings shall be designed at the strength limit state considering the loss of lateral support due to scour from the design flood event. The design of spread footings at the service limit state shall consider tolerable horizontal movement of the footing, and overall stability of the footing considering changes in the foundation conditions due to the scour resulting from the design flood event. Extreme limit state design shall check that the nominal footing resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

Failure by sliding shall be investigated. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of cast-in-place arch spread footings constructed on bedrock or seal concrete. A sliding resistance factor, φ_τ, of 0.90 shall also be applied to the nominal sliding resistance of concrete seals bearing on bedrock. Sliding computations for resistance to lateral loads shall assume maximum frictional coefficients of 0.70 at level bedrock-to-concrete interfaces and 0.60 at cast-in-place arch footing-to-seal interfaces. Anchorage of the arch footings to seals, or of seals to bedrock, may be required to resist sliding forces and improve stability. Dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the designer. If bedrock is observed to slope steeper than 4H:1V at the arch subgrade elevation, the bedrock should be benched to create level steps.

For concrete seals or spread footings 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.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

Calculation of earth pressures on arch footings resisting lateral thrust forces from the arches should assume an at rest earth pressure coefficient, K_o, of 0.47, assuming the arch footings are to be prevented from movement. A resistance factor for at rest earth pressures mobilized to resist lateral sliding forces is not specified in LRFD, therefore use the resistance factor for passive pressure, φ_{ep}, of 0.50 per LRFD Table 10.5.5.2.2-1. For designing the arch footing reinforcing steel for at rest earth pressure, a maximum load factor, γ_{EH}, of 1.50 is recommended.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for arch footing and wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Independent wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever-type walls. The designer may assume BDG Soil Type 4 for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge on arch footings may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 2 below:

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 on Arch Footings

The live load surcharge on retaining walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 3 below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic: 0 feet	Distance from wall pressure surface to edge of traffic: >=1 foot
5	5.0	2.0
10	3.5	2.0
>=20	2.0	2.0

Table 3. Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

Arch foundations and wingwall designs shall include a drainage system behind the arch or wall 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 arches, arch footings, 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.

7.3 Arch Footing Bearing Resistance

Arch spread footings and seals 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 vertical stress may be calculated assuming a triangular or trapezoidal stress distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any footing founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 16 ksf. This assumes a bearing resistance factor, $\varphi_{b,}$ for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. See Appendix C – Calculations for supporting documentation.

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.

7.4 Precast Concrete Modular Gravity Retaining Walls

Precast Concrete Modular Gravity (PCMG) walls may be used to retain approach fills. In general, PCMG wingwalls should be used only at stream crossings where the flow velocities are low, and the potential for severe ice or wave action is low. PCMG walls should also only be used above the ordinary mean high water elevation (Q1.1). The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. PCMG walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to the equivalent height of soil (h_{eq}) taken from Table 3 of this report.

The bearing resistance for PCMG walls founded on a 6 by 12 inch (minimum) leveling slab and compacted granular bedding material shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases 8 feet wide or less and 6.5 ksf for bases 10 to 14 feet wide. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state. The vertical stress may be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1.

For the lowest PCMG unit on granular bedding material, the eccentricity of factored loads at the strength limit state shall not exceed one-fourth (1/4) of the footing dimensions, in either direction.

The bearing resistance for PCMG wall bases founded directly on bedrock or concrete fill may be investigated at the strength limit state using a factored bearing resistance of 16 ksf and at the service limit state using a factored bearing resistance of 20 ksf. The vertical stress may be calculated assuming a trapezoidal stress distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-2.

See Appendix C – Calculations, for supporting documentation.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, the PCMG wall units should be designed so that the factored bearing resistance, in conjunction with the depth of scour determined for the design flood for scour, provides adequate resistance to support the factored strength limit state loads with appropriate resistance factors. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated design scour depth. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Failure by sliding shall be investigated by the wall designer-supplier. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on bedrock, concrete fill or granular borrow. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 (0.80·tan 30°) at the bedding soil to concrete unit interfaces, a maximum frictional coefficient of 0.58 (tan 30°) at foundation soil to soil-infill interfaces, and a maximum friction coefficient of 0.56 (0.80·tan 35°) at interfaces between concrete units and concrete fill. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4 and Table 10.5.5.2.2-1.

7.5 Scour and Riprap

Grain size analyses were performed on soil samples taken from the glacial till deposit encountered in BB-BLR-101 and BB-BLR-103A, for the purpose of generating grain size curves for determining parameters to be used in scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50% passing, D_{50} = 0.30 mm
- Average diameter of particle at 95% passing, D_{95} = 30.0 mm
- Soil Classification: AASHTO Soil Type: A-4

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

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch footings and wingwalls. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Stone riprap shall conform to item number 703.26 of Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

7.6 Settlement

The grades of the bridge approaches will the raised approximately 2.5 feet. However, the approach subgrade is bedrock, and no post-construction settlement is anticipated. Any settlement of arch footings will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

7.7 Frost Protection

Project spread footings for arches and retaining walls will be constructed to bear directly on bedrock or seal concrete cast 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 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, Belfast has a design freezing index of approximately 1450 F-degree days. An assumed water content of 15% was used for granular soils above the water table. These components correlate to a frost depth of 6.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Belfast was assigned a design freezing index of approximately 1256 F-degree days. An assumed water content of 15% was used for granular soils above the water table. These components correlate to a frost depth of 5.8 feet. See Appendix C – Calculations for supporting documentation.

We recommend that foundations constructed within granular fill soils be founded a minimum of 6.0 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.

7.8 Seismic Design Considerations

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

7.9 Construction Considerations

Construction of the arch footings, seals and wingwalls will require soil and rock excavation and removal of the existing abutments and wingwalls. Cofferdams and temporary lateral earth support systems will be required to permit construction of underwater seals and arch footings and wingwalls in the dry.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is 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 subgrade for arch foundations shall no steeper than 4H:1V or it shall be benched in level steps or excavated to be completely level. This criterion also applies for the bedrock subgrade for wingwall footings. 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, at arch or wingwall footings.

The final bearing surface of bedrock or seal concrete shall then be washed with high pressure water and air prior to concrete being placed for the arch and wingwall footings. This does not apply to concrete seals placed underwater. Excavation of highly sloped and loose 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 the upstream dam, 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.

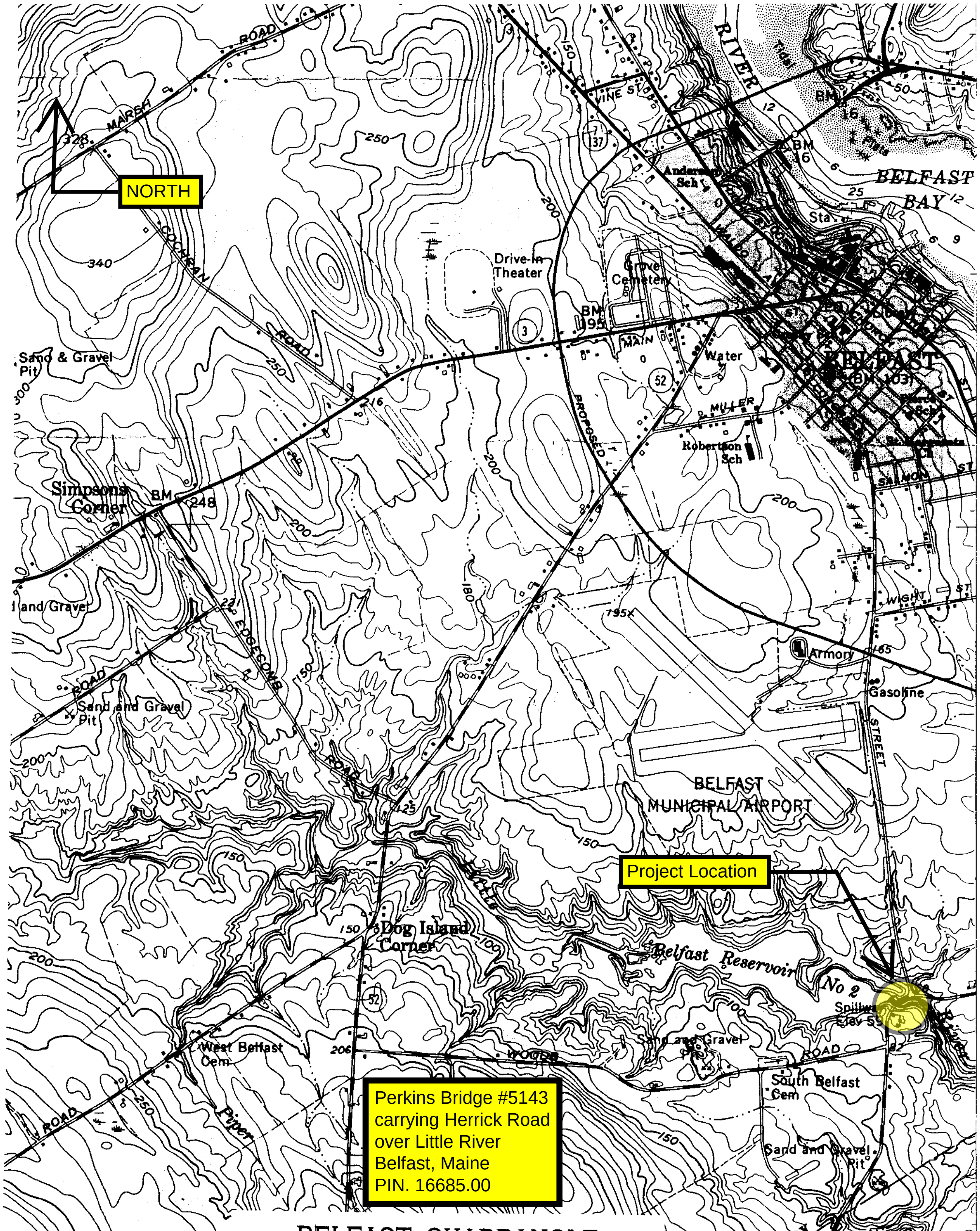
The fine grained glacial till deposits at the site will be susceptible to rutting as a result of exposure to water or construction traffic. The contractor shall protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance occurs, we recommend that the contractor remove and replace the disturbed materials with compacted MaineDOT Standard Specification 703.20, Gravel Borrow. Furthermore, the silt soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Perkins Bridge in Belfast, 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



BELFAST QUADRANGLE
MAINE–WALDO CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
NE/4 BELFAST 15' QUADRANGLE

SCALE 1:24 000

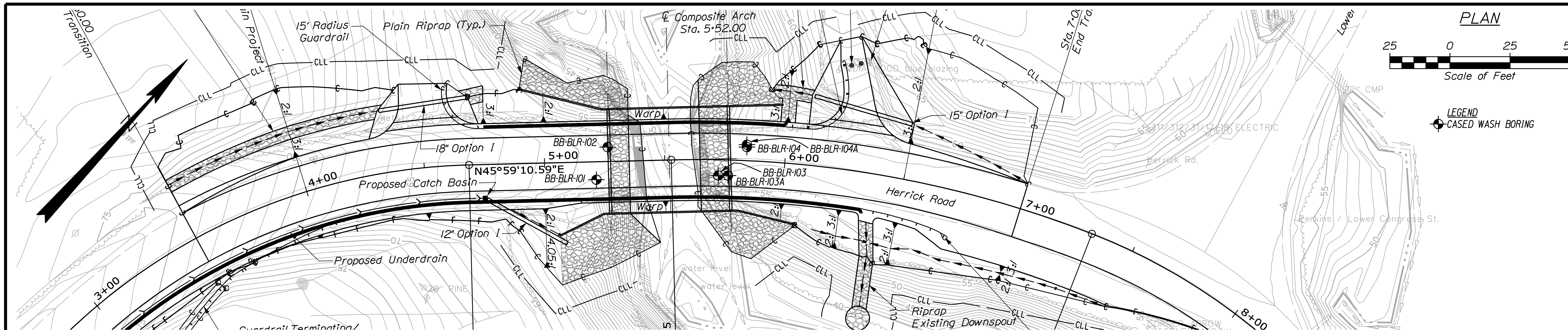
1 ½ 0 1 MILE

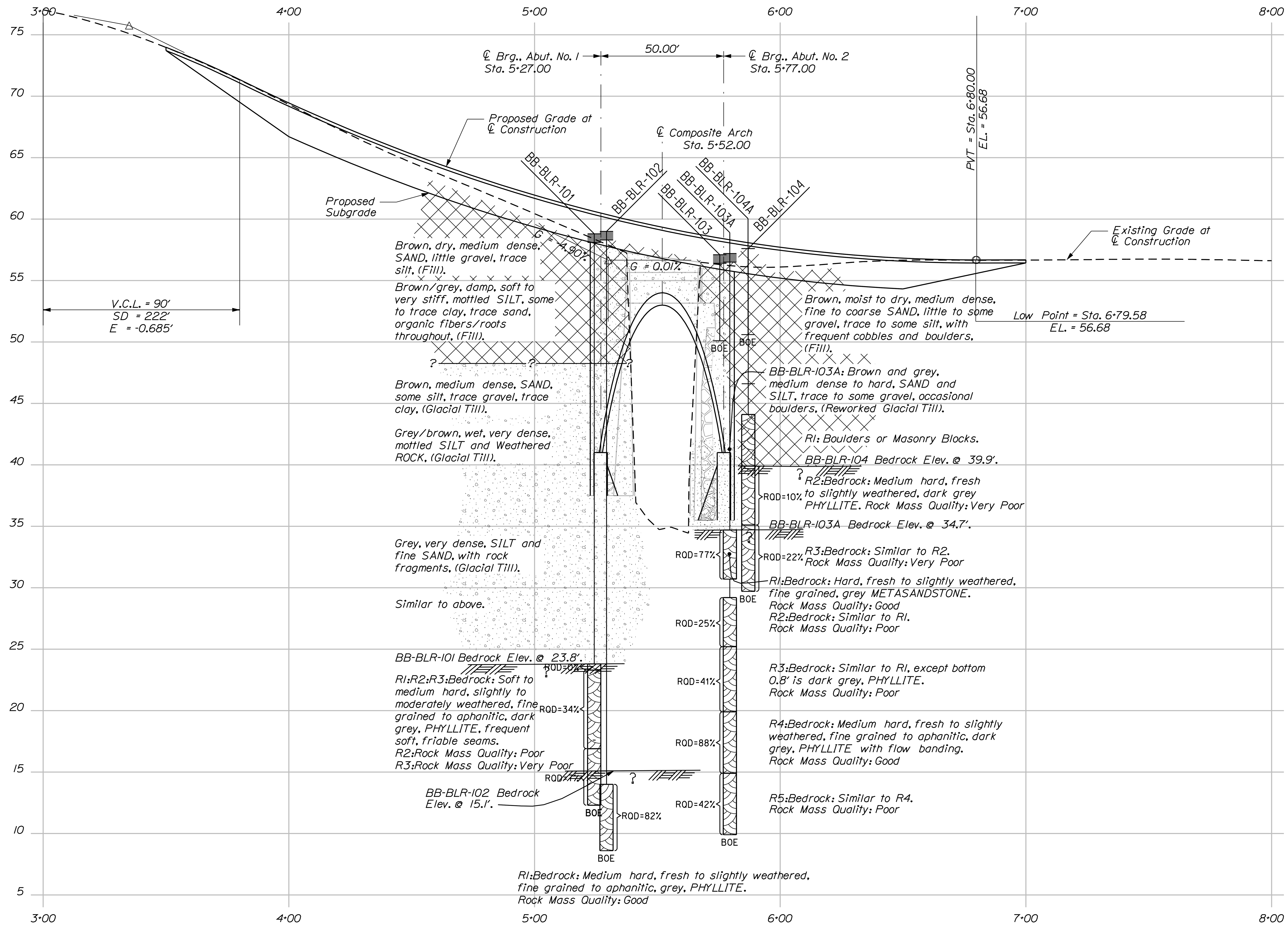
1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

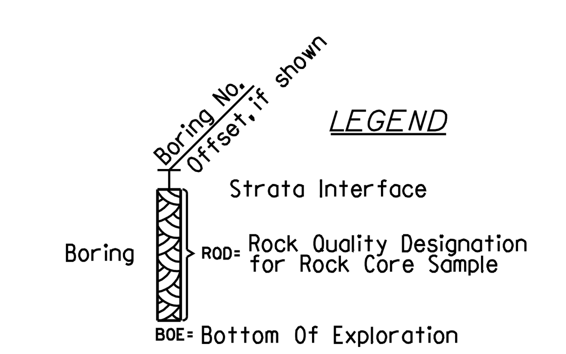
1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

PLAN







PROFILE

Notes: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

Soil statigraphy of BB-BLR-102 not shown for clarity. Refer to Boring Log Sheets.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(500)X

PIN 16685.00

BRIDGE PLANS

BRIDGE NO. 5143

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L.KRUSINSKI	JAN 2010
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

PERKINS BRIDGE
LITTLE RIVER
BELFAST WALDO COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 4

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:1/29/2010

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-101
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	58.8	Auger ID/OD:	5" Solid Stem
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/14/09-12/15/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+24.3, 9.3 Rt.	Casing ID/OD:	HW/NW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	58.10	Asphalt Pavement. —0.70	
	1D	24/10	1.00 - 3.00	26/16/11/7	27			Brown, dry, medium dense, fine to coarse SAND, some gravel, some silt. (Fill)	G#210036 A-1-b, SM WC=7.0%
5	2D	24/17	5.00 - 7.00	2/2/2/3	4			Brown/grey, damp, soft, mottled SILT, some clay, trace fine sand, trace gravel, blocky. Organic fibers/roots throughout sample. (Fill).	G#210037 A-4, ML WC=29.3% LL=27 PL=23 PI=4
10	3D	24/16	10.00 - 12.00	5/10/11/14	21	9	48.30	Top 6": Brown/grey, damp, very stiff, mottled SILT, trace fine sand. Organic fibers/roots throughout sample. (Fill). —10.50 Bottom 10": Brown, medium dense, fine to coarse SAND, some silt, some gravel, trace clay. Rock fragments in bottom 2" of sample. (Glacial Till)	G#210038 A-4, SC-SM WC=11.1%
						29			
						93			
						a50 bRC		a50 blows for 0.1'. bRC Roller Coned Ahead ro 15.0 feet.	
15	4D	2/2	15.00 - 15.17	100(2")	---			Grey/brown, wet, mottled SILT and weathered rock with iron oxide staining. Roller cone through weathered rock to 17.0 feet. At approximately 16.0 feet, wash water changed from rust colored to grey.	
20	5D	2/0	20.00 - 20.17	100(2")	---			Small rock fragments in tip of spoon. Roller cone through till or weathered rock to 25.0 feet.	
25	6D	2/0	25.00 - 25.17	130(2")	---			Small amount of grey SILT and fine SAND with rock fragments in tip of spoon. (Glacial Till)	
30	7D	2/0	30.00 - 30.17	100(2")	---			Small amount of grey SILT and fine SAND with rock fragments in tip of spoon.	
35	R1 R2	6/2 76.8/60	35.00 - 35.50 35.50 - 41.90	RQD = 0% RQD = 34%		NQ-2	23.80	—35.00 Top of Bedrock at Elev. 23.8'. R1: Recovered 2" of medium hard, slightly weathered, fine grained to aphanitic, grey PHYLLITE. Vertical bedding observed in recovered rock. R1: Core Times (min) 35.0-35.5 (1) Behavior of core barrel indicated numerous soft/friable seams from 35.0 to 37.5 in Run R1. R2: Soft to medium hard, slightly to moderately weathered, fine grained to aphanitic, dark grey PHYLLITE. Joints are very close to close, high angle, planar to undulating, rough, fresh to discolored, tight to moderately open. Several areas of core are composed of 2-3 inch or smaller friable fragments that can be easily broken along bedding (very soft to soft). Frequent pyrite on joints and foliation throughout rock core. Rock Mass Quality: Poor. R2: Core Times (min) 35.5-36.5 (1) 36.5-37.5 (1) 37.5-38.5 (2) 38.5-39.5 (2) 39.5-40.5 (1) 40.5-41.9 (2)	
40	R3	55.2/ 55.2	41.90 - 46.50	RQD = 7%				R3: Medium hard, slightly to moderately weathered, fine grained to aphanitic, dark grey PHYLLITE. Joints are very close to close, high angle, planar to undulating, rough, fresh to discolored, tight to moderately open. Frequent pyrite particles throughout rock core. Rock Mass Quality: Very Poor. R3: Core Times (min) 41.9-42.9 (1) 42.9-43.9 (1) 43.9-44.9 (1) 44.9-45.9 (1) 45.9-46.5 (1)	
45							12.30	—46.50 Bottom of Exploration at 46.50 feet below ground surface.	
50									

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-102
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	59.0	Auger ID/OD:	5" Solid Stem
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/17/09-12/18-09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+29.3, 4.1 Lt.	Casing ID/OD:	HW/NW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	58.30	Asphalt Pavement. —0.70	
	1D	24/14	1.00 - 3.00	24/20/13/13	33			Brown, dry, dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#210039 A-1-b, SM WC=4.8%
5	2D	24/14	5.00 - 7.00	3/4/5/6	9	27	54.00	—5.00 Brown/grey, moist, stiff, mottled SILT, some clay, trace sand, trace gravel, blocky. (Reworked Glacial Till).	G#210040 A-6, CL WC=19.1% LL=29 PL=19 PI=10
						41			
						RC		Encountered boulder at 8.0 feet; roller cone through to 10.0 feet; took sample 3D then advanced casing to 15.0 feet by spinning.	
10	3D	13/4	10.00 - 11.08	8/32/100(1")	---	SPUN		Brown/grey, moist to wet, hard, mottled, SILT, little gravel. (Glacial Till) Telescoped 3" casing at 10 feet; advanced 3" casing to 43.9 feet by spinning. Observed wash water below 11.1 feet as gray SILT, little to trace fine Sand with little Rock fragments. Consistent wash water observed to 43.9' bgs.	
15	4D	2/0	15.00 - 15.17	100(2")	---			No Recovery.	
20	5D	2/0	20.00 - 20.17	100(2")	---			No Recovery.	
25	6D	2/0	25.00 - 25.17	100(2")	---			No Recovery. (Glacial Till)	
30	7D	2/0	30.00 - 30.17	100(2")	---			No Recovery.	
35	8D	2/0	35.00 - 35.17	100(2")	---			No Recovery.	
40	9D	2/0	40.00 - 40.17	100(2")	---			No Recovery.	
						RC	15.10	—43.90 Top of Probable Bedrock at Elev. 15.1'. Probable bedrock encountered at 43.9 feet; advanced roller cone to 45.0 feet.	
45	R1	64.8/ 64.8	45.00 - 50.40	RQD = 82%		NQ-2	14.00	—45.00 R1: Bedrock: Medium hard, fresh to slightly weathered, fine grained to aphanitic, grey PHYLLITE. Joints are close to moderately spaced, low angle to moderately dipping, planar to undulating, rough to smooth, fresh and tight to partially open. Rock Mass Quality: Good. R1: Core Times (min) 45.0-46.0 (1) 46.0-47.0 (2) 47.0-48.0 (1) 48.0-49.0 (2) 49.0-50.4 (1)	
50							8.60	—50.40 Bottom of Exploration at 50.40 feet below ground surface.	
55									
60									
65									
70									
75									

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(500)X

BRIDGE NO. 5143
PIN 16685.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER ______		
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	JAN 2010
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

PERKINS BRIDGE
LITTLE RIVER
BELFAST WALDO COUNTY

BORING LOGS

SHEET NUMBER
3
OF 4

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

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-103
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	57.1	Auger ID/OD:	5" Solid Stem
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/15/09-12/15/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	5+75.4, 8.3 Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	56.40	Asphalt Pavement. (0.70)	
	1D	24/10	1.00 - 3.00	2/9/8/5	17			Brown, moist, medium dense, fine to coarse SAND, some gravel, some silt, (Fill).	G#210041 A-2-4, SM WC=8.7%
5	2D	24/2	5.00 - 7.00	3/6/4/4	10			Auger encountered boulder and advanced at angle at 4.5' bgs. Brown, dry, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
							50.10	(7.00) Bottom of Exploration at 7.00 feet below ground surface. Relocated and continued as BB-BLR-103A.	

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLR-103

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

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-103A
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	57.2	Auger ID/OD:	5" Solid Stem
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/16.21/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+79.5, 8.2 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN	56.50	Asphalt Pavement. (0.70)	
10	3D	24/2	10.50 - 12.50	5/8/9/10	17	RC		Brown, medium dense, fine to coarse SAND and GRAVEL, trace silt. Roller cone through numerous boulders/cobbles from 10.5 feet to 20.0 feet bgs, (Fill).	
15	4D	24/2	15.00 - 17.00	8/9/19/12	28		42.20	(15.00) Brown, medium dense, fine to coarse SAND and SILT, some gravel. (Reworked Glacial Till)	
20	5D	15/7	20.00 - 21.25	75/23/100(3")	---	RC		Grey, hard, SILT, some sand, little gravel. Wood fragments in sample near tip. Roller cone from 20.3 feet to 22.5 feet through boulder/ possible bedrock. Completed borehole at 26.5 feet on 12/16/ 2009, then on 12/21/09 roller coned ahead to 28.0 feet bgs to clean coring surface, stated R2.	G#210042 A-4, ML WC=28.1%
	R1	48/48	22.50 - 26.50	RQD = 77%		NQ-2	34.70	(22.50) Top of Bedrock at Elev. 34.7'. R1: Bedrock: Hard, fresh to slightly weathered, fine grained, grey METASANDSTONE. Joints are close, moderately dipping, planar, smooth, discolored (iron oxide staining), tight to open. Occasional quartz veins throughout core. Rock Mass Quality: Good. R1: Core Times (min) 22.5-23.5 (2) 23.5-24.5 (3) 24.5-25.5 (2) 25.5-26.5 (3)	
	R2	48/37	28.00 - 32.00	RQD = 25%		NQ-2		R2: Hard, fresh to slightly weathered, fine grained grey METASANDSTONE. Joints are very close to close, low angle to moderately dipping, planar, smooth, fresh to discolored, tight to open. Occasional quartz between joints (up to 3/4" thick). Rock Mass Quality: Poor. R2: Core Times (min) 28.0-29.0 (1) 29.0-30.0 (2) 30.0-31.0 (2) 31.0-32.0 (1)	
	R3	63.6/63.6	32.00 - 37.30	RQD = 41%			20.70	R3: Upper 4.5': Hard, fresh to slightly weathered, fine grained, grey METASANDSTONE. Joints are very close to close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored, tight to open. Occasional quartz between joints (up to 3/4" thick). Flow banding evident at 36.5 feet-contact between metasandstone and phyllite. Bottom 0.8': Medium hard, fresh to slightly weathered, fine grained to aphantic, dark grey, PHYLLITE with flow banding. Rock Mass Quality: Poor. R3: Core Times (min) 32.0-33.0 (1) 33.0-34.0 (1) 34.0-35.0 (2) 35.0-36.0 (2) 36.0-37.3 (2) (36.50)	
	R4	60/60	37.30 - 42.30	RQD = 88%				R4: Medium hard, fresh to slightly weathered, fine grained to aphanitic, dark grey, PHYLLITE with flow banding. Joints are close to moderately spaced, moderate to high angle, undulating, rough to smooth, fresh , tight to partially open. Occasional quartz veins. Rock Mass Quality: Good. R4: Core Times (min) 37.3-38.3 (1) 38.3-39.3 (2) 39.3-40.3 (2) 40.3-41.3 (2) 41.3-42.3 (1)	
	R5	60/60	42.30 - 47.30	RQD = 42%			9.90	R5: Medium hard, fresh to slightly weathered, fine grained to aphanitic, dark grey, PHYLLITE with flow banding. Joints are close, low angle to moderately dipping, planar to undulating, rough, fresh, tight to moderately wide. Quartz veins throughout core (up to 1/4" thick). Rock Mass Quality: Poor. R5: Core Times (min) 42.3-43.3 (1) 43.3-44.3 (1) 44.3-45.3 (2) 45.3-46.3 (2) 46.3-47.3 (2) (47.30) Bottom of Exploration at 47.30 feet below ground surface.	

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLR-103A

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

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-104A
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	57.6	Auger ID/OD:	5" Solid Stem
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	N/A
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/18/09-12/18/09	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	5+86.9, 5.0 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA		No sampling, auger not plumb. Attempted to advance borehole with HW Casing.	
							50.60	(7.00) Bottom of Exploration at 7.00 feet below ground surface. Abandoned hole, relocated to BB-BLR-104.	

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLR-104A

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

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-104
PIN: 16685.00

Driller:	Maine Test Borings	Elevation (ft.)	57.6	Auger ID/OD:	N/A
Operator:	Richard Leonard	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	Jennifer Tooley	Rig Type:	Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/18/09-12/18/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+87, 4.0 Lt.	Casing ID/OD:	4"/4.5"	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN		No sampling in overburden. Unable to observe wash water (no return water) likely due to presence of boulders and cobbles. (Fill).	
						RC	46.60	(11.00) Advanced roller cone into rock from 11.0 feet to 13.5 feet (boulder or block). (Fill).	
	R1	50.4/34	13.50 - 17.70			NQ-2	44.10	(13.50) R1: Upper 1.7': Hard, slightly weathered, fine grained, grey, METASANDSTONE (boulder or block). (Fill). R1: Core Times (min) 13.5-14.5 (1) 14.5-15.5 (2) 15.5-16.5 (2) 16.5-17.7 (2)	
15							42.40	(15.20) Lower 2.2': Hard, fresh to slightly weathered, medium grained, GRANITE (boulder or block). (Fill).	
	R2	57.6/24	17.70 - 22.50	RQD = 10%			39.90	(17.70) Top of Bedrock at Elev. 39.9'. R2: Bedrock: Medium hard, fresh to slightly weathered, dark grey, PHYLLITE. Joints are very close to close, low angle to moderately dipping, planar to undulating, smooth to rough, fresh, tight to partially open. Occasional quartz veins. Rock Mass Quality: Very poor. R2: Core Times (min) 17.7-18.7 (1) 18.7-19.7 (2) 19.7-20.7 (1) 20.7-21.7 (1) 21.7-22.5 (1)	
	R3	64.8/34	22.50 - 27.90	RQD = 22%			29.70	R3: Medium hard, fresh to slightly weathered, dark grey, PHYLLITE. Joints are very close to close, low angle to moderately dipping, planar to undulating, smooth to rough, fresh, tight to partially open. Occasional quartz veins. Rock Mass Quality: Very poor. R3: Core Times (min) 22.5-23.5 (1) 23.5-24.5 (1) 24.5-25.5 (2) 25.5-26.5 (1) 26.5-27.9 (2) (27.90) Bottom of Exploration at 27.90 feet below ground surface.	

Remarks:

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Page 1 of 1

Boring No.: BB-BLR-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1668(500)X

BRIDGE NO. 5143 PIN 16685.00 BRIDGE PLANS

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	JAN 2010
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

PERKINS BRIDGE
LITTLE RIVER
BELFAST WALDO COUNTY

BORING LOGS

SHEET NUMBER 4 OF 4

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-101
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	58.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14/09-12/15/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+24.3, 9.3 Rt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	58.10		Asphalt Pavement. 0.7	
	1D	24/10	1.0 - 3.0	26/16/11/7	27				Brown, dry, medium dense, fine to coarse SAND, some gravel, some silt. (Fill)	G#210036 A-1-b, SM WC=7.0%
5	2D	24/17	5.0 - 7.0	2/2/2/3	4				Brown/grey, damp, soft, mottled SILT, some clay, trace fine sand, trace gravel, blocky. Organic fibers/roots throughout sample, (Fill).	G#210037 A-4, ML WC=29.3% LL=27 PL=23 PI=4
10	3D	24/16	10.0 - 12.0	5/10/11/14	21	9	48.30		Top 6": Brown/grey, damp, very stiff, mottled SILT, trace fine sand. Organic fibers/roots throughout sample, (Fill). 10.5	G#210038 A-4, SC-SM WC=11.1%
						29			Bottom 10": Brown, medium dense, fine to coarse SAND, some silt, some gravel, trace clay. Rock fragments in bottom 2" of sample. (Glacial Till)	
						93				
						a50 bRC			a50 blows for 0.1'. bRC Roller Coned Ahead ro 15.0 feet.	
15	4D	2/2	15.0 - 15.2	100(2")	---				Grey/brown, wet, mottled SILT and weathered rock with iron oxide staining. Roller cone through weathered rock to 17.0 feet. At approximately 16.0 feet, wash water changed from rust colored to grey.	
20	5D	2/0	20.0 - 20.2	100(2")	---				Small rock fragments in tip of spoon. Roller cone through till or weathered rock to 25.0 feet.	
25										

Remarks:

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

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

Boring No.: BB-BLR-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Perkins Bridge #5143 carrying Herrick Road over Little River Location: Belfast, Maine	Boring No.: BB-BLR-101 PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	58.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14/09-12/15/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+24.3, 9.3 Rt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	2/0	25.0 - 25.2	130(2")	---				Small amount of grey SILT and fine SAND with rock fragments in tip of spoon. (Glacial Till)	
30	7D	2/0	30.0 - 30.2	100(2")	---				Small amount of grey SILT and fine SAND with rock fragments in tip of spoon.	
35	R1	6/2	35.0 - 35.5	RQD = 0%		NQ-2	23.80		35.0 Top of Bedrock at Elev. 23.8'. R1: Recovered 2" of medium hard, slightly weathered, fine grained to aphanitic, grey PHYLLITE. Vertical bedding observed in recovered rock. R1: Core Times (min) 35.0-35.5 (1) Behavior of core barrel indicated numerous soft/friable seams from 35.0 to 37.5 in Run R1. R2: Soft to medium hard, slightly to moderately weathered, fine grained to aphanitic, dark grey PHYLLITE. Joints are very close to close, high angle, planar to undulating, rough, fresh to discolored, tight to moderately open. Several areas of core are composed of 2-3 inch or smaller friable fragments that can be easily broken along bedding (very soft to soft). Frequent pyrite on joints and foliation throughout rock core. Rock Mass Quality: Poor. R2: Core Times (min) 35.5-36.5 (1) 36.5-37.5 (1) 37.5-38.5 (2) 38.5-39.5 (2) 39.5-40.5 (1) 40.5-41.9 (2)	
	R2	76.8/60	35.5 - 41.9	RQD = 34%						
40										
	R3	55.2/55.2	41.9 - 46.5	RQD = 7%					R3: Medium hard, slightly to moderately weathered, fine grained to aphanitic, dark grey PHYLLITE. Joints are very close to close, high angle, planar to undulating, rough, fresh to discolored, tight to moderately open. Frequent pyrite particles throughout rock core. Rock Mass Quality: Very Poor. R3: Core Times (min) 41.9-42.9 (1) 42.9-43.9 (1) 43.9-44.9 (1) 44.9-45.9 (1) 45.9-46.5 (1)	
45							12.30		46.5	
50										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-101
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	58.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/14/09-12/15/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+24.3, 9.3 Rt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50									**Bottom of Exploration at 46.50 feet below ground surface.**	
55										
60										
65										
70										
75										

Remarks:

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

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

Boring No.: BB-BLR-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Perkins Bridge #5143 carrying Herrick Road over Little River **Location:** Belfast, Maine	**Boring No.:** BB-BLR-102 **PIN:** 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	59.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/17/09-12/18-09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+29.3, 4.1 Lt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	58.30		Asphalt Pavement. 0.7	
	1D	24/14	1.0 - 3.0	24/20/13/13	33				Brown, dry, dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#210039 A-1-b, SM WC=4.8%
5	2D	24/14	5.0 - 7.0	3/4/5/6	9	27	54.00		5.0 Brown/grey, moist, stiff, mottled SILT, some clay, trace sand, trace gravel, blocky. (Reworked Glacial Till).	G#210040 A-6, CL WC=19.1% LL=29 PL=19 PI=10
						41				
						RC			Encountered boulder at 8.0 feet; roller cone through to 10.0 feet; took sample 3D then advanced casing to 15.0 feet by spinning.	
10	3D	13/4	10.0 - 11.1	8/32/100(1")	---	SPUN			Brown/grey, moist to wet, hard, mottled, SILT, little gravel. (Glacial Till) Telescoped 3" casing at 10 feet; advanced 3" casing to 43.9 feet by spinning. Observed wash water below 11.1 feet as gray SILT, little to trace fine Sand with little Rock fragments. Consistent wash water observed to 43.9' bgs.	
15	4D	2/0	15.0 - 15.2	100(2")	---				No Recovery.	
20	5D	2/0	20.0 - 20.2	100(2")	---				No Recovery.	
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Perkins Bridge #5143 carrying Herrick Road over Little River	**Location:**	Belfast, Maine	**Boring No.:**	BB-BLR-102
				PIN:	16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	59.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/17/09-12/18-09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+29.3, 4.1 Lt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	2/0	25.0 - 25.2	100(2")	---				No Recovery. (Glacial Till)	
30	7D	2/0	30.0 - 30.2	100(2")	---				No Recovery.	
35	8D	2/0	35.0 - 35.2	100(2")	---				No Recovery.	
40	9D	2/0	40.0 - 40.2	100(2")	---				No Recovery.	
						RC	15.10		43.9 Top of Probable Bedrock at Elev. 15.1'. Probable bedrock encountered at 43.9 feet; advanced roller cone to 45.0 feet.	
45	R1	64.8/64.8	45.0 - 50.4	RQD = 82%		NQ-2	14.00		45.0 R1: Bedrock: Medium hard, fresh to slightly weathered, fine grained to aphanitic, grey PHYLLITE. Joints are close to moderately spaced, low angle to moderately dipping, planar to undulating, rough to smooth, fresh and tight to partially open. Rock Mass Quality: Good. R1: Core Times (min) 45.0-46.0 (1) 46.0-47.0 (2) 47.0-48.0 (1) 48.0-49.0 (2) 49.0-50.4 (1)	
50										

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Perkins Bridge #5143 carrying Herrick Road over Little River Location: Belfast, Maine	Boring No.: BB-BLR-102 PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	59.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/17/09-12/18-09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+29.3, 4.1 Lt.	**Casing ID/OD:**	HW/NW	**Water Level*:**	None Observed

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							8.60		50.4 **Bottom of Exploration at 50.40 feet below ground surface.**	
55										
60										
65										
70										
75										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-103
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	57.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/15/09-12/15/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	5+75.4, 8.3 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	56.40		Asphalt Pavement. 0.7	
	1D	24/10	1.0 - 3.0	2/9/8/5	17				Brown, moist, medium dense, fine to coarse SAND, some gravel, some silt, (Fill).	G#210041 A-2-4, SM WC=8.7%
									Auger encountered boulder and advanced at angle at 4.5' bgs.	
5	2D	24/2	5.0 - 7.0	3/6/4/4	10				Brown, dry, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
							50.10		7.0	
									Bottom of Exploration at 7.00 feet below ground surface. Relocated and continued as BB-BLR-103A.	
10										
15										
20										
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-103A
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	57.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/16,21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+79.5, 8.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN	56.50		Asphalt Pavement. 0.7	
10	3D	24/2	10.5 - 12.5	5/8/9/10	17	RC			Brown, medium dense, fine to coarse SAND and GRAVEL, trace silt. Roller cone through numerous boulders/cobbles from 10.5 feet to 20.0 feet bgs, (Fill).	
15	4D	24/2	15.0 - 17.0	8/9/19/12	28		42.20		15.0 Brown, medium dense, fine to coarse SAND and SILT, some gravel. (Reworked Glacial Till)	
20	5D	15/7	20.0 - 21.3	75/23/100(3")	---	RC			Grey, hard, SILT, some sand, little gravel. Wood fragments in sample near tip. Roller cone from 20.3 feet to 22.5 feet through boulder/possible bedrock. Completed borehole at 26.5 feet on 12/16/2009, then on 12/21/09 roller coned ahead to 28.0 feet bgs to clean coring surface, stated R2.	G#210042 A-4, ML WC=28.1%
	R1	48/48	22.5 - 26.5	RQD = 77%		NQ-2	34.70		22.5 Top of Bedrock at Elev. 34.7'. R1: Bedrock: Hard, fresh to slightly weathered, fine grained, grey METASANDSTONE. Joints are close, moderately dipping, planar, smooth, discolored (iron oxide staining), tight to open. Occasional quartz veins throughout core. Rock Mass Quality: Good.	
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Perkins Bridge #5143 carrying Herrick Road over Little River	**Location:**	Belfast, Maine	**Boring No.:**	BB-BLR-103A
				PIN:	16685.00
Driller:	Maine Test Borings	**Elevation (ft.)**	57.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/16,21/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+79.5, 8.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25									R1: Core Times (min) 22.5-23.5 (2) 23.5-24.5 (3) 24.5-25.5 (2) 25.5-26.5 (3)	
	R2	48/37	28.0 - 32.0	RQD = 25%		NQ-2			R2: Hard, fresh to slightly weathered, fine grained grey METASANDSTONE. Joints are very close to close, low angle to moderately dipping, planar, smooth, fresh to discolored, tight to open. Occasional quartz between joints (up to 3/4" thick). Rock Mass Quality: Poor. R2: Core Times (min) 28.0-29.0 (1) 29.0-30.0 (2) 30.0-31.0 (2) 31.0-32.0 (1)	
30										
	R3	63.6/63.6	32.0 - 37.3	RQD = 41%					R3: Upper 4.5': Hard, fresh to slightly weathered, fine grained, grey METASANDSTONE. Joints are very close to close, low angle to moderately dipping, planar, smooth to rough, fresh to discolored, tight to open. Occasional quartz between joints (up to 3/4" thick). Flow banding evident at 36.5 feet-contact between metasandstone and phyllite. Bottom 0.8': Medium hard, fresh to slightly weathered, fine grained to aphantic, dark grey, PHYLLITE with flow banding. Rock Mass Quality: Poor. R3: Core Times (min) 32.0-33.0 (1) 33.0-34.0 (1) 34.0-35.0 (2) 35.0-36.0 (2) 36.0-37.3 (2)	
35										
							20.70		36.5	
	R4	60/60	37.3 - 42.3	RQD = 88%					R4: Medium hard, fresh to slightly weathered, fine grained to aphanitic, dark grey, PHYLLITE with flow banding. Joints are close to moderately spaced, moderate to high angle, undulating, rough to smooth, fresh , tight to partially open. Occasional quartz veins. Rock Mass Quality: Good. R4: Core Times (min) 37.3-38.3 (1) 38.3-39.3 (2) 39.3-40.3 (2) 40.3-41.3 (2) 41.3-42.3 (1)	
40										
	R5	60/60	42.3 - 47.3	RQD = 42%					R5: Medium hard, fresh to slightly weathered, fine grained to aphanitic, dark grey, PHYLLITE with flow banding. Joints are close, low angle to moderately dipping, planar to undulating, rough, fresh, tight to moderately wide. Quartz veins throughout core (up to 1/4" thick). Rock Mass Quality: Poor. R5: Core Times (min) 42.3-43.3 (1) 43.3-44.3 (1) 44.3-45.3 (2) 45.3-46.3 (2) 46.3-47.3 (2)	
45										
							9.90		47.3	
									Bottom of Exploration at 47.30 feet below ground surface.	
50										

Remarks:

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

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

Boring No.: BB-BLR-103A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-104
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	57.6	**Auger ID/OD:**	N/A
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/09-12/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+87, 4.0 Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN			No sampling in overburden. Unable to observe wash water (no return water) likely due to presence of boulders and cobbles. (Fill).	
5										
10										
11.0						RC	46.60		Advanced roller cone into rock from 11.0 feet to 13.5 feet (boulder or block). (Fill).	
13.5	R1	50.4/34	13.5 - 17.7			NQ-2	44.10		R1: Upper 1.7': Hard, slightly weathered, fine grained, grey, METASANDSTONE (boulder or block). (Fill). R1: Core Times (min) 13.5-14.5 (1) 14.5-15.5 (2) 15.5-16.5 (2) 16.5-17.7 (2)	
15.2							42.40		Lower 2.2': Hard, fresh to slightly weathered, medium grained, GRANITE (boulder or block). (Fill).	
17.7	R2	57.6/24	17.7 - 22.5	RQD = 10%			39.90		Top of Bedrock at Elev. 39.9'. R2: Bedrock: Medium hard, fresh to slightly weathered, dark grey, PHYLLITE. Joints are very close to close, low angle to moderately dipping, planar to undulating, smooth to rough, fresh, tight to partially open. Occasional quartz veins. Rock Mass Quality: Very poor. R2: Core Times (min) 17.7-18.7 (1) 18.7-19.7 (2) 19.7-20.7 (1) 20.7-21.7 (1) 21.7-22.5 (1)	
20										
22.5	R3	64.8/34	22.5 - 27.9	RQD = 22%					R3: Medium hard, fresh to slightly weathered, dark grey, PHYLLITE. Joints are very close to close, low angle to moderately dipping, planar to undulating,	
25										

Remarks:

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

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

Boring No.: BB-BLR-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-104
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	57.6	**Auger ID/OD:**	N/A
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/09-12/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+87, 4.0 Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							29.70		smooth to rough, fresh, tight to partially open. Occasional quartz veins. Rock Mass Quality: Very poor. R3: Core Times (min) 22.5-23.5 (1) 23.5-24.5 (1) 24.5-25.5 (2) 25.5-26.5 (1) 26.5-27.9 (2) —27.9	
									Bottom of Exploration at 27.90 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Perkins Bridge #5143 carrying Herrick Road over Little River
Location: Belfast, Maine

Boring No.: BB-BLR-104A
PIN: 16685.00

Driller:	Maine Test Borings	**Elevation (ft.)**	57.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Richard Leonard	**Datum:**	NAD 1983	**Sampler:**	N/A
Logged By:	Jennifer Tooley	**Rig Type:**	Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/18/09-12/18/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	5+86.9, 5.0 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			No sampling, auger not plumb. Attempted to advance borehole with HW Casing.	
							50.60		7.0	
									Bottom of Exploration at 7.00 feet below ground surface. Abandoned hole, relocated to BB-BLR-104.	
5										
10										
15										
20										
25										

Remarks:

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Boring No.: BB-BLR-104A

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Belfast Project Number: 16685.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-BLR-101, 1D	5+24.3	9.3 Rt.	1.0-3.0	210036	1	7.0			SM	A-1-b	II
BB-BLR-101, 2D	5+24.3	9.3 Rt.	5.0-7.0	210037	1	29.3	27	4	ML	A-4	IV
BB-BLR-101, 3D	5+24.3	9.3 Rt.	10.0-12.0	210038	1	11.1			SC-SM	A-4	III
BB-BLR-102, 1D	5+29.3	4.1 Lt.	1.0-3.0	210039	2	4.8			SM	A-1-b	II
BB-BLR-102, 2D	5+29.3	4.1 Lt.	5.0-7.0	210040	2	19.1	29	10	CL	A-6	IV
BB-BLR-103, 1D	5+75.4	8.3 Rt.	1.0-3.0	210041	2	8.7			SM	A-2-4	II
BB-BLR-103A, 5D	5+79.5	8.2 Rt.	20.0-21.25	210042	2	28.1			ML	A-4	IV

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

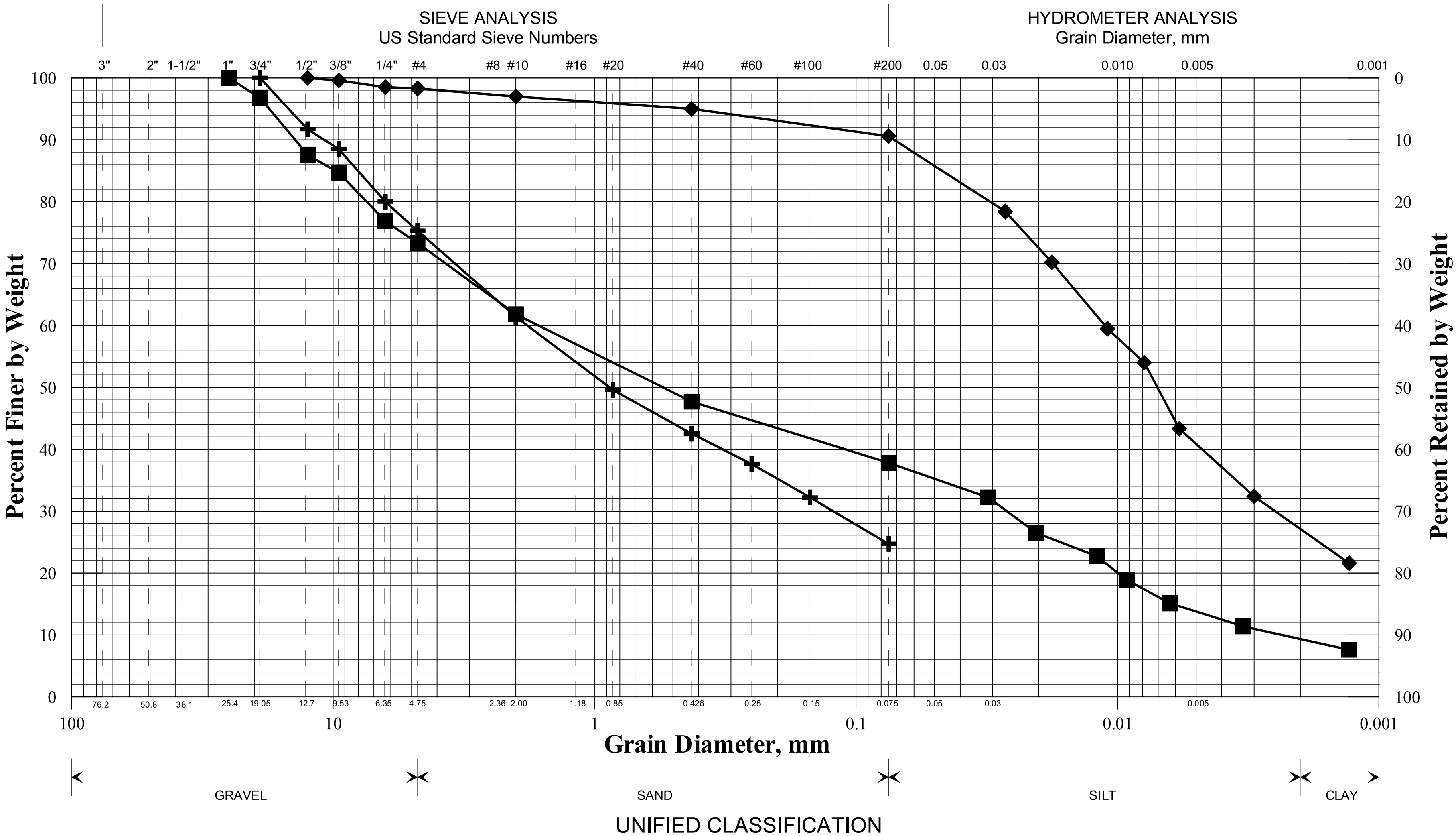
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

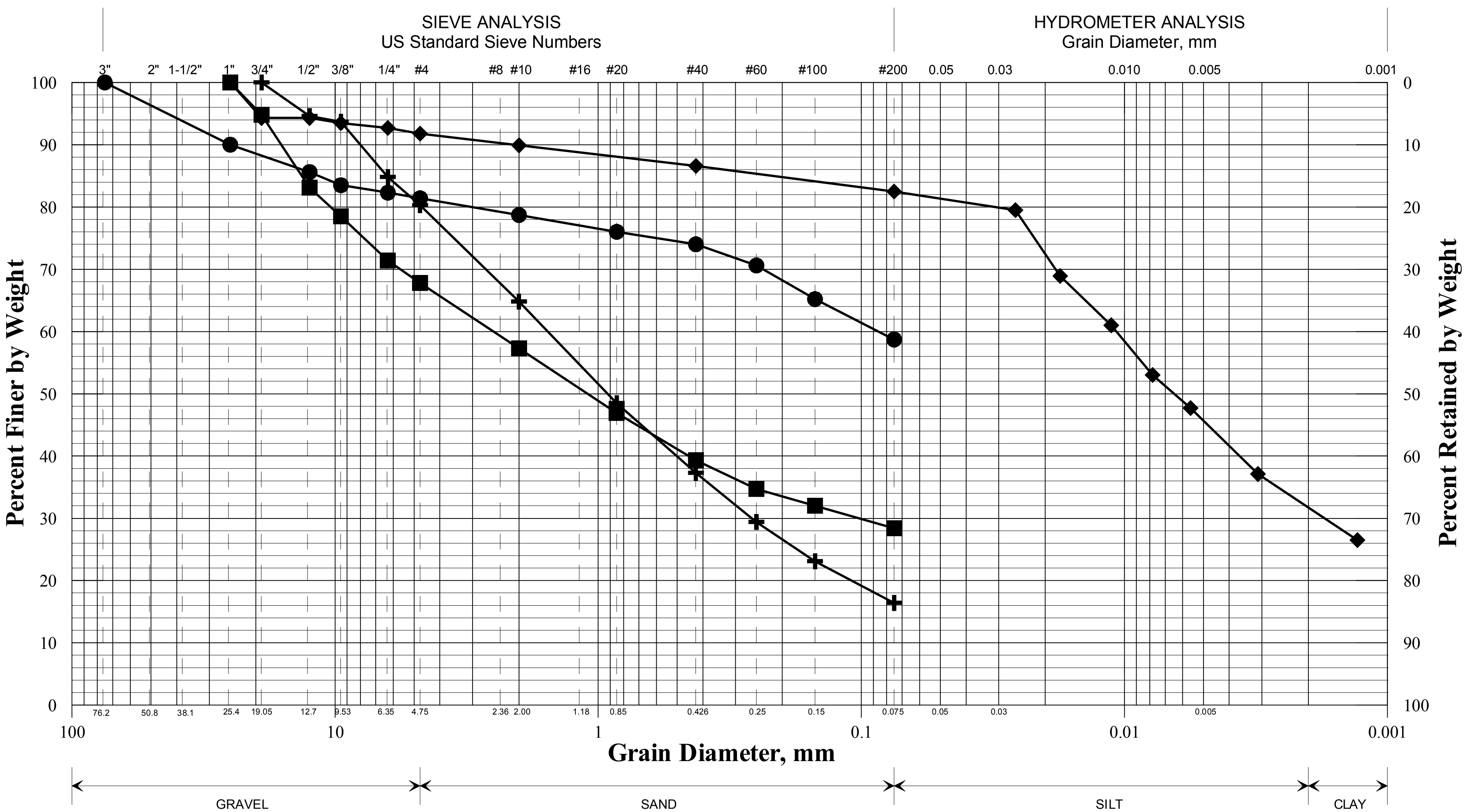


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BLR-101/1D	5+24.3	9.3 RT	1.0-3.0	SAND, some gravel, some silt.	7.0			
◆	BB-BLR-101/2D	5+24.3	9.3 RT	5.0-7.0	SILT, some clay, trace sand, trace gravel.	29.3	27	23	4
■	BB-BLR-101/3D	5+24.3	9.3 RT	10.0-12.0	SAND, some silt, some gravel, trace clay.	11.1			
●									
▲									
✖									

PIN
016685.00
Town
Belfast
Reported by/Date
WHITE, TERRY A 1/12/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BLR-102/1D	5+29.3	4.1 LT	1.0-3.0	SAND, little gravel, little silt.	4.8			
◆	BB-BLR-102/2D	5+29.3	4.1 LT	5.0-7.0	SILT, some clay, trace sand, trace gravel.	19.1	29	19	10
■	BB-BLR-103/1D	5+75.4	8.3 RT	1.0-3.0	SAND, some gravel, some silt.	8.7			
●	BB-BLR-103A/5D	5+79.5	8.2 RT	20.0-21.25	SILT, some sand, little gravel.	28.1			
▲									
✖									

PIN
016685.00
Town
Belfast
Reported by/Date
WHITE, TERRY A 1/12/2010

SHEET 2

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
210037	BB-BLR-101/2D	GEOTECHNICAL (DISTURBED)	12/14/2009	12/30/2009

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **5+24.3** Offset, ft: **10.7 RT** Dbfg, ft: **5.0-7.0**

PIN: **016685.00** Town: **Belfast** Sampler: **CONSULTANT PERSONNEL**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	100.0
⅜ in. [9.5 mm]	99.6
¼ in. [6.3 mm]	98.5
No. 4 [4.75 mm]	98.3
No. 10 [2.00 mm]	97.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	95.0
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	90.6
[0.0268 mm]	78.4
[0.0178 mm]	70.2
[0.0109 mm]	59.5
[0.0079 mm]	54.0
[0.0058 mm]	43.3
[0.0030 mm]	32.4
[0.0013 mm]	21.6

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	27
Plastic Limit (T 90), %	23
Plasticity Index (T 90), %	4
Specific Gravity, Corrected to 20°C (T 100)	2.66
Loss on Ignition (T 267) — Loss, %	
Loss on Ignition (T 267) — H2O, %	
Water Content (T 265), %	29.3

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

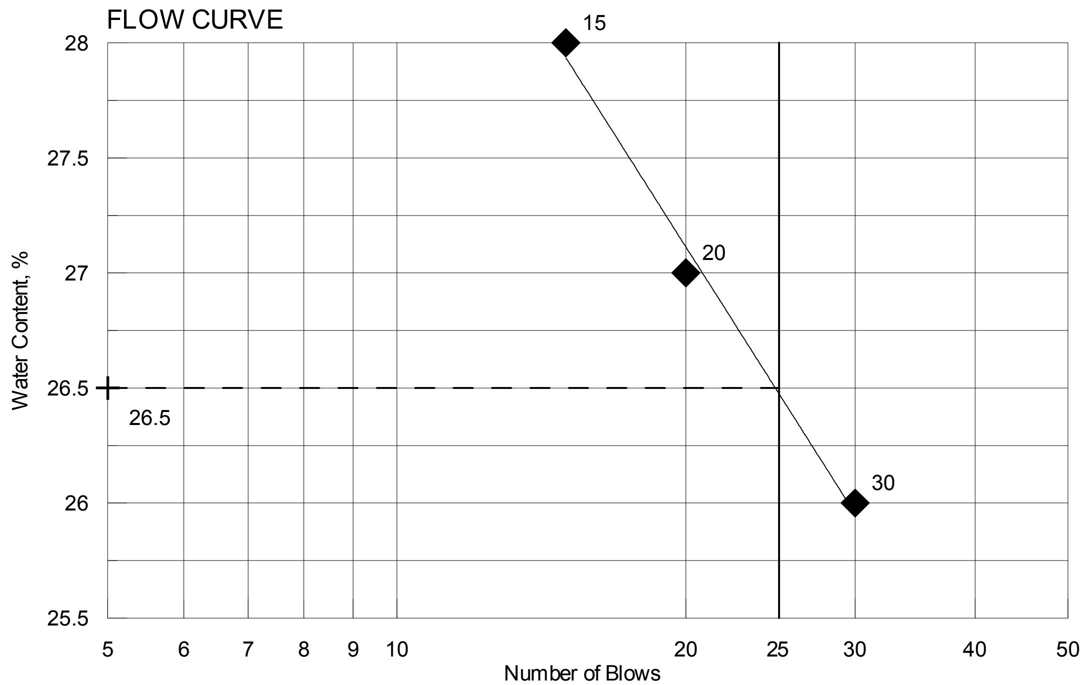
Comments:

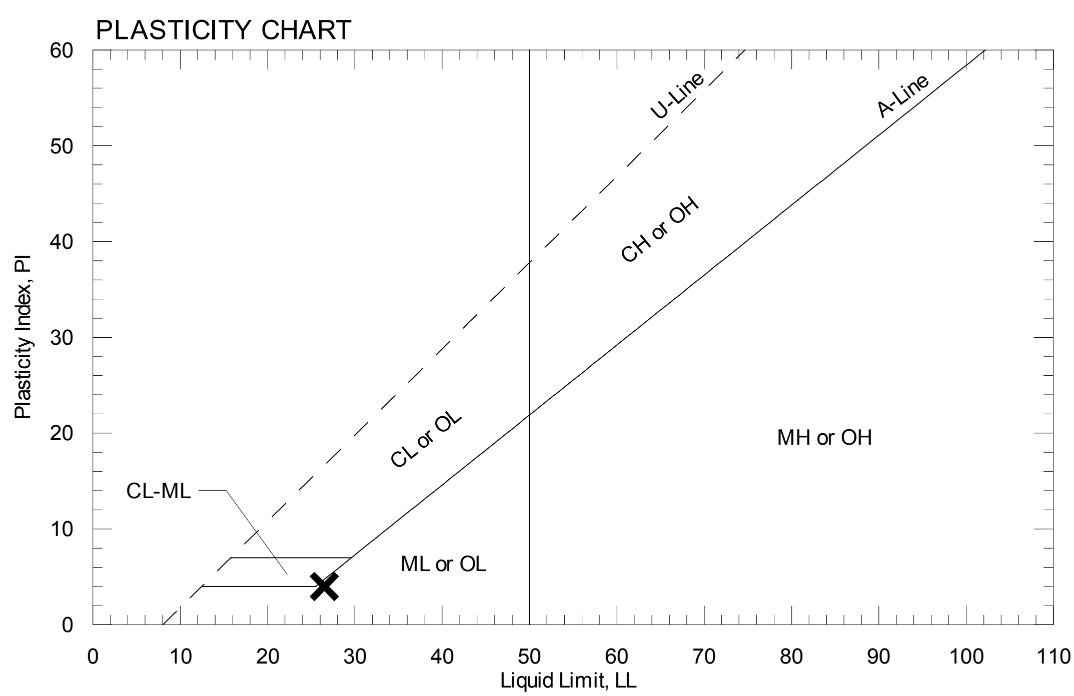
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **1/11/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Belfast	Reference No.	210037
PIN	016685.00	Water Content, %	29.3
Sampled	12/14/2009	Plastic Limit	23
Boring No./Sample No.	BB-BLR-101/2D	Liquid Limit	27
Station	5+24.3	Plasticity Index	4
Depth	5.0-7.0	Tested By	BBURR





GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
210040	BB-BLR-102/2D	GEOTECHNICAL (DISTURBED)	12/16/2009	12/30/2009

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **5+29.3** Offset, ft: **4.1 LT** Dbfg, ft: **5.0-7.0**

PIN: **016685.00** Town: **Belfast** Sampler: **CONSULTANT PERSONNEL**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	94.3
½ in. [12.5 mm]	94.3
⅜ in. [9.5 mm]	93.5
¼ in. [6.3 mm]	92.7
No. 4 [4.75 mm]	91.8
No. 10 [2.00 mm]	89.9
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	86.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	82.5
[0.0259 mm]	79.5
[0.0175 mm]	68.9
[0.0112 mm]	61.0
[0.0078 mm]	53.0
[0.0056 mm]	47.7
[0.0031 mm]	37.1
[0.0013 mm]	26.5

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	29
Plastic Limit (T 90), %	19
Plasticity Index (T 90), %	10
Specific Gravity, Corrected to 20°C (T 100)	2.68
Loss on Ignition (T 267) — Loss, % / H2O, %	
Water Content (T 265), %	19.1

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

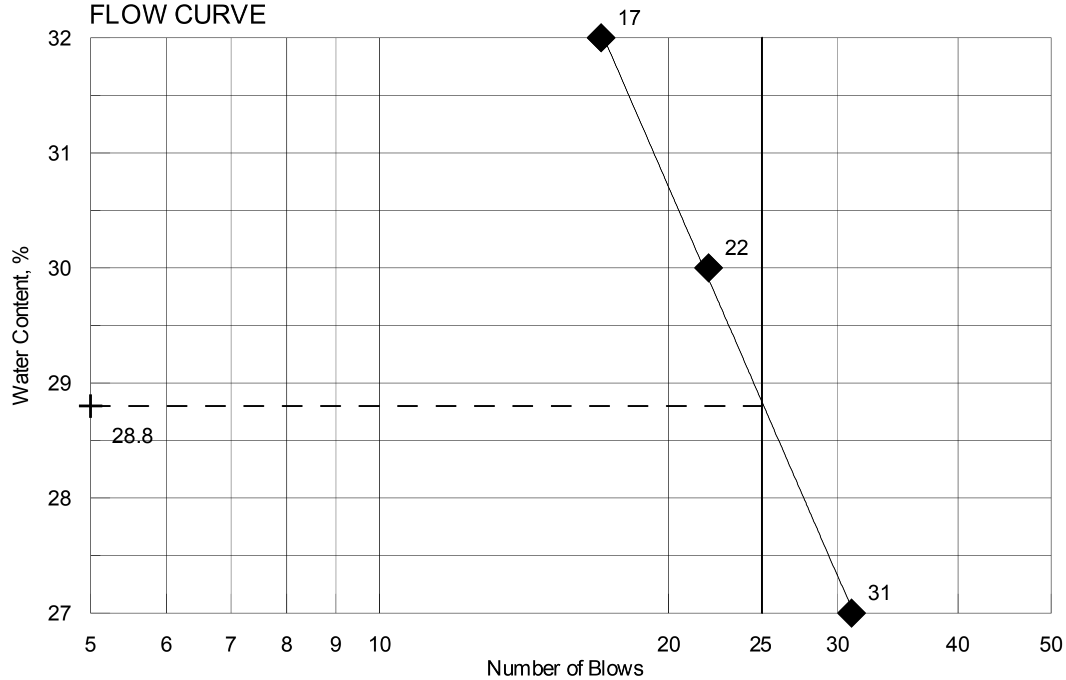
Comments:

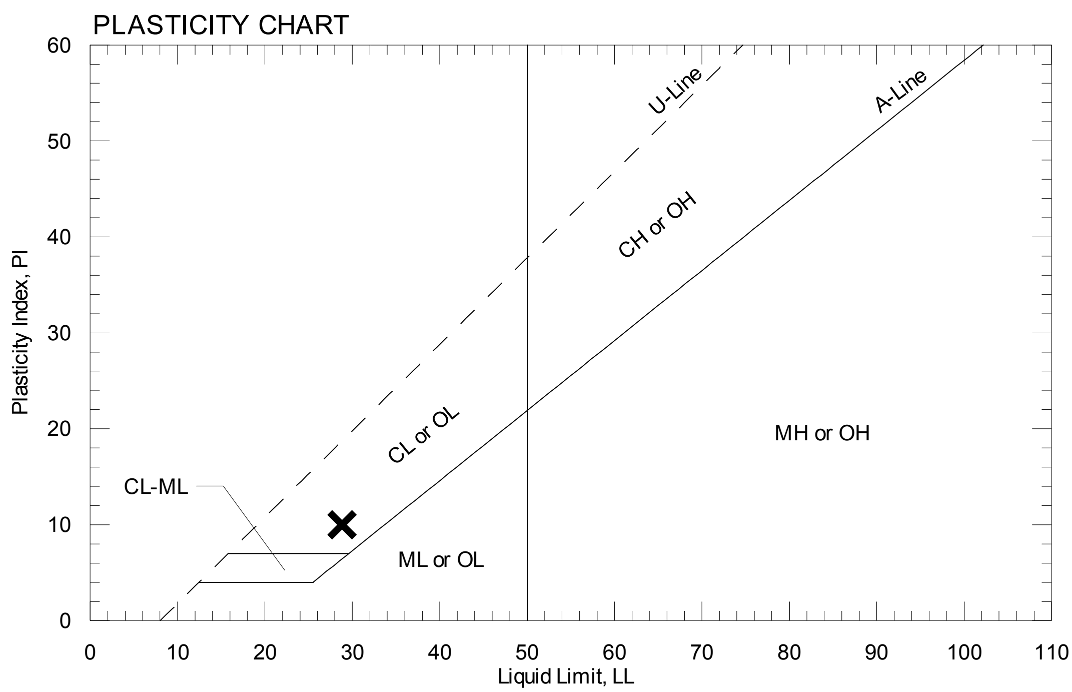
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **1/11/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Belfast	Reference No.	210040
PIN	016685.00	Water Content, %	19.1
Sampled	12/16/2009	Plastic Limit	19
Boring No./Sample No.	BB-BLR-102/2D	Liquid Limit	29
Station	5+29.3	Plasticity Index	10
Depth	5.0-7.0	Tested By	BBURR





<u>Appendix C</u>

Calculations

Part I - Factored Bearing Resistance - Arch 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:

Arch Footing 1: Boring BB-BLR-101, upper 5-ft core, PHYLLITE, soft to medium hard, slightly to moderately weathered, joints/bedding very close to close, high angle. **RQD=0-34%**. Lower 5-ft core similar but RQD=7%. BB-BLR-102, bedrock description similar, except medium hard, and **RQD = 82%.**

Arch Footing 2: Boring BB-BLR-103A, upper 5-ft core is metamorphic METASANDSTONE, hard, fresh to slightly weathered, moderately dipping with **RQD = 77%**. Lower rock core RQD's are 25, 41, 88, 42%. BB-BLR-104, upper 5-ft core is PHYLLITE, medium hard, fresh to slightly weathered, joints at low angles to mod. dipping, **RQD=10%**, Lower core run is 22%.

Arch Footings 1 and 2

Bearing Material:	Weathered or broken bedrock of any kind except argillite (shale).
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16 - 24 ksf
Recommended Value	20 ksf

$$q_{nominal} := 20 \cdot ksf$$

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

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

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

$$q_{factored} = 20 \cdot ksf$$

Use a recommended value for the factored bearing resistance. Use 20 ksf for service limit state analysis - and for preliminary sizing of the footing.

Part II - Factored Bearing Resistance for Strength Limit State Analyses

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

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

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

$\phi_{rock} := 24 \cdot deg$ — *Tomlinson, Page 139, Wyllie, phi for low friction rock, schists 20-27*

$q_{uc} := 10000 \cdot psi$ — *AASHTO, 2002, Table 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength, "phyllite" 3,500 to 35,000 psi*

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

$c = 1000 \cdot psi$ — *OK - correlates to Bowles, pg 278, giving range for rock cohesion of 500-2500 psi*

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

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

$$q_{nominal} = 35 \cdot ksf$$

Factored Bearing Resistance

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

$\phi_{bc} := 0.45$ $\quad q_{factored} := q_{nominal} \cdot \phi_{bc}$ $\quad q_{factored} = 16 \cdot ksf$

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

$N_q := \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^6$ $N_q = 13.332$

$N_c := 5 \cdot \tan\left(45 \cdot deg + \frac{\phi_{rock}}{2}\right)^4$ $N_c = 28.113$

$N_\gamma := N_q + 1$ $N_\gamma = 14.332$

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

$s_c := 1.0$ $s_\gamma := 1.0$ $B := \begin{pmatrix} 3 \\ 4 \\ 5 \\ 6 \end{pmatrix} \cdot ft$

Embedment factor - footing placed on top of bedrock

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

Nominal Bearing Resistance

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

$$q_{ult} = \begin{pmatrix} 326 \\ 328 \\ 329 \\ 330 \end{pmatrix} \cdot ksf$$

Reduce the calculated bearing resistance by RQD^2, per Bowles. Use lowest RQD encountered at top of bedrock in 4 borings: 34%

$$RQD := 0.34$$

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

$$q_{nominal} = \begin{pmatrix} 38 \\ 38 \\ 38 \\ 38 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance

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

$$\phi_{bc} := 0.45 \qquad q_{factored} := q_{nominal} \cdot \phi_{bc} \qquad \boxed{q_{factored} = \begin{pmatrix} 17 \\ 17 \\ 17 \\ 17 \end{pmatrix} \cdot ksf}$$

Recommended Factored Bearing Resistance of 16 ksf for service limit state analyses.

Analysis : Bearing Resistance of PCMG walls on granular bedding material

Assumptions

1. Base of footing founded with 4 feet embedment for frost (conservative, 6 feet is recommended.)

2. Assumed parameters for compacted granular backfill
   Saturated unit weight = 130 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 125 pcf
   ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
   ϕ and SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
   ϕ = 32 degrees (Bowles Tables 3-4 and 2-6).
   Su= undrained shear strength (c) 0 psf

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

PCMG Footing Width and Depth

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft \qquad D_f := 4.0 \cdot ft \qquad D_w := 0 \cdot ft \qquad \gamma_w := 62.4 \cdot pcf$$

Foundation Soil (Granular Fill)

$$\gamma 1_{sat} := 130 \cdot pcf$$

$$\gamma 1_d := 125 \cdot pcf$$

$$\phi := 32 \cdot deg$$

$$c_1 := 0 \cdot psf$$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

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".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Coarse to medium sand, little gravel	Very dense	8 to 12	8
	Medium dense to dense	4 to 8	6
	Loose	2 to 6	4

Recommend 6 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing.

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

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

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

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\qquad N_q := 23.2$ $\qquad N_\gamma := 22$

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

$$q := D_f \cdot (\gamma1_{sat} - \gamma_w) \qquad q = 0.27 \cdot ksf$$

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = \begin{pmatrix} 10.7 \\ 12.2 \\ 13.7 \\ 15.2 \\ 16.7 \\ 18.2 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\varphi_b := 0.45$$

$$q_r := q_n \cdot \varphi_b$$

$$q_r = \begin{pmatrix} 4.8 \\ 5.5 \\ 6.2 \\ 6.8 \\ 7.5 \\ 8.2 \end{pmatrix} \cdot ksf \qquad \textbf{for} \qquad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 5 ksf for footings 8 feet wide or smaller, on compacted granular fill.
Recommend a bearing resistance of 6.5 ksf for footings 10 to 14 feet wide or greater.

If on bedrock; recommend factored bearing resistance values calculated for footings bearing on bedrock: 16 ksf and 20 ksf for strength and service limit states, respectively.

L. Krusinski
December 2009
Checked by: KM 1-2010

Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$

Internal friction angle $\phi_1 := 32 \cdot \text{deg}$

Cohesion $c_1 := 0 \cdot \text{psf}$

Restrained Arch Footing

At Rest Earth Pressure - Rankine Theory

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

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

$$K_o = 0.47$$

Wingwalls - Active Earth Pressure

Active Earth Pressure - Rankine Theory

Either Rankine or Coulomb may be used for **long heeled** cantilever walls, where the failure surface is uninterupted by the top of the wall stem. In general, use Rankine though. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$$\beta := 0 \cdot \text{deg}$$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.307$$

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

Coulomb Theory

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.

Angle of back face of wall to the horizontal, θ :

$$\theta := 90 \cdot deg$$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" δ = 17 to 22 degrees; select 20 degrees.

$\delta := 20 \cdot deg$ for a gravity shaped wall where the interface friction is between soil and concrete

to $\delta := 24 \cdot deg$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, δ=1/3 to 2/3 Φ

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot deg$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{ac} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{ac} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface or 'pressure surface.'

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

From Design Freezing Index Map:
Belfast, Maine
DFI = 1450 degree-days

Case I - Assume granular fill soils at elevation of possible footings of WC=15%.

Interpolate between frost depth of 80.65" for WC=10% at 1450 DFI and 66.7" inches for WC=20%

Depth of Frost Penetration =

$$d := \frac{80.65 - 66.7}{2} \cdot in + 66.7 \cdot in \qquad d = 73.675 \cdot in \qquad d = 6.14 \cdot ft$$

Method 2 - ModBerg Software

Belfast lies on the same Design Freezing Index contour as Ellsworth, Maine, BDG Fig. 5-1

Case 1 - coarse grained soils with water content of 15%

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

Project Location: Ellsworth, Maine

Air Design Freezing Index       =  1256 F-days
N-Factor                        =  0.80
Surface Design Freezing Index   =  1005 F-days
Mean Annual Temperature         =  44.6 deg F
Design Length of Freezing Season =  126 days

---------------------------------------------------------
Layer
#:Type        t    w%     d    Cf  Cu   Kf   Ku    L
---------------------------------------------------------
1-Coarse     69.1 15.0 125.0  31  40   2.9  1.8  2,700
---------------------------------------------------------

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 = 5.76 ft = 69.1 in.
*********************************************************
```

Recommendation: 6.0 feet for design of spread footings not founded on bedrock

Appendix D

Special Provisions

s

SPECIAL PROVISION
<u>SECTION 635</u>
PREFABRICATED BIN TYPE RETAINING WALL
(Prefabricated Concrete Modular Gravity Wall)

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

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

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

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

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

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

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Precast Concrete Units	712.061
Drainage Geotextile	722.02

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

Concrete Units:

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

1. All dimensions shall be within (edge to edge of concrete) ±3/16 in.

2. Squareness. The length differences between the two diagonals shall not exceed 5/16 in.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 in. in 4 ft will be rejected. For textured surfaces, any surface defects in excess of 5/16 in. in 5 ft shall be rejected.

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

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

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

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

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

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

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

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a Professional Engineer registered in accordance with the laws of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength, Service and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:

1. Overturning: For foundations on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width.
2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$

   Where: R_R = Factored Sliding Resistance

   $\gamma_{p(max)}$ = Maximum Load Factor

   EH = Horizontal Earth Pressure

   ES = Earth Surcharge (as applicable)
3. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure

   Where: q_R = Factored Bearing Resistance, as shown on the plans

   Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
4. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

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

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

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

The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

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

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

E. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.

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

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

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

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

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

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with

Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

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

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

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

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

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

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

635.05 Construction Requirements

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

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

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

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

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

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

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

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

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

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

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

Payment will be made under:

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

SPECIAL PROVISION

SECTION 610

STONE FILL, RIPRAP, STONE BLANKET,

AND STONE DITCH PROTECTION

Add the following paragraph to Section 610.02:

Materials shall meet the requirements of the following Sections of Special Provision 703:

Stone Fill	703.25
Plain and Hand Laid Riprap	703.26
Stone Blanket	703.27
Heavy Riprap	703.28
Definitions	703.32

Add the following paragraph to Section 610.032.a.

Stone fill and stone blanket shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following paragraph to Section 610.032.b:

Riprap shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following to Section 610.032:

Section 610.032.d. The grading of riprap, stone fill, stone blanket and stone ditch protection shall be determined by the Resident by visual inspection of the load before it is dumped into place, or, if ordered by the Resident, by dumping individual loads on a flat surface and sorting and measuring the individual rocks contained in the load. A separate, reference pile of stone with the required gradation will be placed by the Contractor at a convenient location where the Resident can see and judge by eye the suitability of the rock being placed during the duration of the project. The Resident reserves the right to reject stone at the job site or stockpile, and in place. Stone rejected at the job site or in place shall be removed from the site at no additional cost to the Department.

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

703.25 Stone Fill   Stones for stone fill shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for stone fill shall be angular and rough. Rounded, subrounded, or long thin stones will not be allowed. Stone for stone fill may be obtained from quarries or by screening oversized rock from earth borrow pits. The maximum allowable length to thickness ratio will be 3:1. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (500 lbs) shall have a maximum dimension of approximately 36 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension of 12 inches (200 lbs).

703.26 Plain and Hand Laid Riprap   Stone for riprap shall consist of hard, sound durable rock that will not disintegrate by exposure to water or weather. Stone for riprap shall be angular and rough. Rounded, subrounded or long thin stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (200 lbs) shall have an average dimension of approximately 12 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension greater than 9 inches (50 lbs).

703.27 Stone Blanket   Stones for stone blanket shall consist of sound durable rock that will not disintegrate by exposure to water or weather. Stone for stone blanket shall be angular and rough. Rounded or subrounded stones will not be allowed. Stones may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (300 lbs) shall have minimum dimension of 14 inches, and the maximum stone size (3000 lbs) shall have a maximum dimension of approximately 66 inches. Fifty percent of the stones by volume shall have average dimension greater than 24 inches (1000 lbs).

703.28 Heavy Riprap   Stone for heavy riprap shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for heavy riprap shall be angular and rough. Rounded, subrounded, or thin, flat stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for heavy riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (500 lbs) shall have minimum dimension of 15 inches, and at least fifty percent of the stones by volume shall have an average dimension greater than 24 inches (1000 lbs).

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces
Subrounded: Particles have nearly plane sides but have well-rounded corners and edges
Rounded: Particles have smoothly curved sides and no edges