MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MILL BRIDGE
MILL CREEK ROAD OVER MILL CREEK
ISLESBORO, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Mill Bridge which carries Mill Creek Road over Mill Creek in Islesboro, Maine. At the time of this report, the proposed replacement bridge is a pile-supported integral abutment bridge (IAB) with a voided slab superstructure, on the same horizontal alignment as the existing bridge. The proposed span length of the voided slab superstructure is 56 feet. The following design recommendations are discussed in detail in this report:

Integral Abutment H-piles - H-piles for support of the integral abutments should be end bearing and driven to the required resistance on or within bedrock. Piles should be fitted with pile points to protect the tips and improve penetration into bedrock. The H-piles shall be designed for all relevant strength, service and extreme limit state load groups. Due to shallow bedrock at the south abutment (Abutment 2), it may not be possible to achieve adequate pile embedment. Based on the proposed superstructure depth and abutment height, we estimate free pile lengths of approximately 9 to 10 feet at Abutment 2 and approximately 19 to 23 feet at Abutment 1. During final design, L-Pile or FB-Pier analyses will be conducted to determine if the pile length provided is enough to prevent translation of the pile and to evaluate the soil-pile interaction for combined axial and flexure loads and thermal displacements. The resulting bending moment in the pile would be supplied to the structural engineer for evaluation. The structural resistance of the piles should be determined for compliance with the interaction equation by the structural engineer.

If L-Pile or FB-Pier analyses indicate that the short H-pile design does not achieve fixity and requires a pinned boundary condition at the pile tip, the piles should be fitted with Rock Injector HP-80500 Pile Points, manufactured by Associated Pile and Fitting (APF), LLC, to improve penetration and friction at the pile tips, or the piles installed in shallow rock sockets.

Lateral Pile Resistance - Lateral loads will be reacted by plumb piles. We recommend final design tasks include a series of lateral pile resistance analyses using L-Pile® software or FB-Pier software. Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Section 7.1.3 of this report.

Pile Quality Control - The contractor is required to perform a wave equation analysis. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. With this level of quality control, the pile should be driven to a nominal resistance equal to the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65.

Restrike tests may be required as part of the pile field quality control program if pile behavior indicates the pile is not seated firmly on bedrock or if piles "walk" out of position. Any piles installed in bedrock sockets at Abutment 2 should be seated at the driving resistance specified for driven piles at Abutment 1.

Integral Abutments - Integral abutments shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 (herein referred to as AASHTO LRFD). Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design.

Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating small abutment movements, or a Coulomb K_p of 6.73 should the ratio of lateral abutment movement to the abutment height (y/H) exceed 0.005. For designing the integral abutment reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach stab is specified, reduction, not elimination of the surcharge load is permitted.

Global Stability - Proposed cross-section changes at the bridge approaches include widened and over-steepened roadway embankments. The designer has proposed 1.5H:1V slopes at some stations to minimize wetland impacts. Stability analyses were conducted to determine factors of safety against global failure of the proposed 1.5H:1V and 1.75H:1V side slopes at Abutments 1 and 2. The evaluations indicate that the minimum required factor of safety of 1.3 for slope stability is achievable with the use of 1.5H:1V and 1.75H:1V slopes treated with 4 feet of riprap and with a riprap toe-in of 4 feet.

Settlement - The recent marine sediments and the underlying marine clays are loose and/or soft in nature, and are characteristically compressible. Therefore, the cross-section changes consisting of widened, over-steepened slopes behind Abutments 1 and 2 in combination with an approximate 2-foot grade raise of Mill Creek Road will result in long-term consolidation of the soft foundation materials. Long-term, consolidation settlement of widened slopes and raised approach fills is estimated to range from approximately 2 to 8 inches, with the maximum settlements occurring at the toes of the slopes. The foundation piles will be short, therefore downdrag loads due to settlement are anticipated to be negligible.

It is not recommended that any foundations be supported on shallow foundations bearing on the recent marine sediments or marine clay.

Frost Protection – Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Foundations placed on or in the fill soils should be founded a minimum of 5.5 feet below finished exterior grade for frost protection.

Seismic Design Considerations – In conformance with LRFD Article 3.10.1, seismic analysis is not required for single-span bridges regardless of seismic zone. However,

superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

Scour and Riprap - In accordance with Section 2.3.11.3 of the BDG, bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap and bridges located immediately on the ocean should use heavy riprap. The top of the riprap should be located at a minimum elevation of 2 feet above Mean Higher High Water (MHHW), estimated at Elevation 5.85 feet. Anecdotal information indicated a recent high tide event of Elev. 7.1 feet. In accordance with the MaineDOT Bridge Design Guide (BDG), consideration should be given to placing riprap higher than the MHHW due to waves and wave run-up.

Based on the results of global stability analyses, riprap slopes should be 4 feet thick and the toe should be constructed with an approximate 4-foot by 4-foot "key-in" or toe berm. The toe berm shall be overlain by 6 to 12 inches of streambed material to minimize permanent wetland impacts. The riprap slopes shall be underlain by a 1 foot thick layer of bedding material and Class 1 nonwoven erosion control geotextile.

Construction Considerations – Construction of the abutments will require soil excavation and pile driving. Cofferdams or temporary lateral earth support systems may be required to permit construction of the pile foundations and abutments.

The removal of fill soils will result in the exposure of naturally deposited pockets of potentially sensitive clays and silts. These soils will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance occurs, we recommend that the contractor remove and replace the disturbed materials with ¾-inch stone or compacted MaineDOT Standard Specification 703.20, Gravel Borrow.

The recent marine sediments and marine clays 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.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Mill Bridge which carries Mill Creek Road over Mill Creek in Islesboro, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for substructure design.

Mill Bridge was built in 1937 and is a single-span, concrete slab superstructure, supported on courses of dry-laid, granite block abutments. The first course of granite comprising the south abutment is constructed on a 3-foot thick concrete subfooting cast on bedrock. The granite blocks comprising the north abutment are half (eastern portion) supported on a concrete subfooting bearing on bedrock and half (western portion) supported on 14-ton timber piles with a concrete cap. The dry laid granite abutments are in "poor" condition according to a 2010 MaineDOT Bridge Inspection Report. The bridge has a Sufficiency Rating of 46.2. The clear span of the existing bridge is 22 ft. The existing bridge width is narrow at 22 feet.

At the time of this report, the proposed new bridge will be a 56-foot span, pile-supported integral abutment bridge (IAB) with a precast, voided slab superstructure. The new bridge will be located on the same horizontal alignment as the existing bridge. The finished grade on the bridge will be raised approximately 1.7 and 2 feet at the proposed north and south abutments, respectively. It is proposed that the new bridge accommodate two 10-foot lanes with 3-foot shoulders.

2.0 GEOLOGIC SETTING

Mill Bridge in Islesboro carries Mill Creek Road over Mill Creek as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of Maine (1985) indicates the surficial soils in the vicinity of the bridge project consist primarily of glaciomarine deposits with nearby glacial till soil unit contacts. The predominant native soil units at the site based on our subsurface explorations are recent marine sediments and marine clay.

Glaciomarine deposits generally consist of silt, clay, sand and minor amounts of gravel. These fine-grained deposits are composed of glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the proposed bridge site as Rocks of Islesboro consisting of metamorphic, interbedded pelite and sandstone. Bedrock cores obtained from test borings at the site are identified as predominantly phyllite with interbeds of metasandstone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling four test borings. The four borings were terminated with bedrock cores. Test Borings BB-IMC-101 and BB-IMC-102 where drilled behind the existing south abutment. Test borings BB-IMC-103 and BB-IMC-104 were drilled behind the existing north abutment.

Test borings BB-IMC-101 and BB-IMC-104 were drilled on opposite sides of Mill Creek at an approximate 62-foot spacing to determine subsurface conditions for a pile-supported integral abutment bridge option with an estimated 58-foot span. Test borings BB-IMC-102 and BB-IMC-103 were drilled to explore subsurface conditions for a possible 30-foot single-span Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) bridge or a precast concrete arch bridge option. Additionally, power auger probes were advanced behind the existing granite block abutments using solid-stem augers in an attempt to define the location of the back of the existing granite blocks.

The boring locations and power auger probe locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. The borings were drilled between November 8 and 10, 2011 by Maine Test Boring (MTB) of Hermon, 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 Power Auger Probes, and on Sheet 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. 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 MTB drill rig is equipped with a rope-and-cathead hammer therefore no correction for average hammer energy transfer is required. The raw N-values presented for samples are also the "corrected" values (N_{60}) as an average energy transfer of 60% is assumed for rope-and-cathead hammer systems.

The bedrock was cored in the four borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated. A consultant geotechnical engineer logged the subsurface conditions encountered. The MaineDOT geotechnical engineer and geotechnical consultant selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed drafted boring logs and identified field and laboratory testing requirements. The as-drilled boring locations and elevations were surveyed by the MaineDOT Survey Crew at the completion of the drilling program.

4.0 LABORATORY TESTING

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

Soil laboratory testing consisted of two standard grain size analyses, five grain size analyses with hydrometer, seven natural water content tests, and four Atterberg Limits test. Rock laboratory testing consisted of two unconfined compression tests. Soil tests were performed in the R. W. Gillespie & Associates Geotechnical Testing Laboratory in Saco, Maine. Rock testing was performed at the Thielsch Engineering Geotechnical Testing Laboratory in Hopkinton, Massachusetts. The results of soil and rock 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 Power Auger Probes and on Sheet 4 - Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of miscellaneous fill soils, recent marine sediments, marine clay, and glacial till, all underlain by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and Power Auger Probes and on Sheet 4 – Boring Logs. A generalized subsurface profile is shown on Sheet 3 – Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Miscellaneous Fill Soils

A layer of granular fill and till fill soils comprised of reworked till soils was encountered beneath the pavement in all the borings. The encountered thickness is approximately 6.2 to 8.3 feet thick. The fill subunits are comprised of predominately fine to medium sand, with lesser portions of gravel and silt. One reworked till fill subunit included fragments of wood.

SPT N-values in fill soils were 1 to 18 blows per foot (bpf), indicating the fill soils are very loose to medium dense in consistency.

Two grain size analyses of the fill unit classify the soils as A-4 under the AASHTO Soil Classification System and ML and SM under the Unified Soil Classification System (USCS). Water contents were approximately 17 and 18 percent.

5.2 Recent Marine Sediments

Recent marine sediments were encountered below the fill unit in all of the borings. The deposit is highly variable and consisted of:

- Grey-black, wet, silty, fine to medium sand, little to some gravel, trace to some silt, with wood, reeds and shell fragments
- Grey-black to grey-brown to dark grey, wet, fine to medium and fine to coarse sand, little to some silt, some gravel, with wood, reeds and shell fragments, weathered rock fragments
- Grey, silt and clay, with shell fragments

- Grey-black, sandy, organic silt, trace to little fine gravel, with wood and shell fragments
- Black, fine to coarse sand, some gravel, little silt
- Dark grey, fine to medium sand, little silt, trace coarse sand with fibrous peat pockets, shell fragments and wood.

The encountered thickness of the recent marine sediments is approximately 6.8 to 11.5 feet. SPT N-values in the variable marine sediments ranged from 3 to 19 bpf indicating the granular sediments are very loose to medium dense in consistency. One fine grained marine sediment sample had an N-value of 2 bpf indicating the fine grained sediments are very soft in consistency.

Three grain size analyses of the recent marine sediments resulted in the soil being classified as A-4 and A-2-4 under the AASHTO Soil Classification System and ML, GM and SC under the Unified Soil Classification System (USCS).

Atterberg Limits tests on two samples from the marine sediments determined moisture contents of approximately 21 and 34 percent and plastic limits of 20 and 25. For one of the two samples tested, the natural water content was greater than the liquid limit and greater than the plastic limit. The calculated liquidity indexes (LI) were less than 1.0 for one sample and greater than 1.0 for the second. Therefore, the relative consistency of the recent marine sediments is lightly preconsolidated to unconsolidated.

Table 5-1 summarizes the results of Atterberg Limits tests made from the recent marine sediment samples:

Sample No.	Visual Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-IMC-101, 3D	Organic Silt	34.3	33	25	8	1.18
BB-IMC-103, 3D	Silty Sand, little to some gravel, with wood, reeds & shell fragments	21.1	28	20	7	0.16

Table 5-1 Atterberg Limits Test Results – Recent Marine Sediments

5.3 Marine Clay

A marine clay deposit was encountered below the recent marine sediments in the two borings located behind the existing north abutment. The encountered thickness ranged from approximately 5.9 to 9.9 feet. The marine deposit encountered consisted of:

- Grey, silty clay, with black streaks, trace fine sand;
- Grey, silty clay, trace fine sand with sand and gravel seams;

One vane shear test conducted within the marine clay layer showed a measured undrained shear strength of the layer of 636 pounds per square foot (psf) indicating that the marine clay is medium stiff in consistency. Where an SPT test was conducted in the marine clay the N-value was 2 indicting that some marine clay subunits are very soft in consistency. The remolded vane shear strength at the one test interval was 111 psf. Based on the ratio of peak to remolded shear strength at the test interval, the marine clay is classified as medium sensitive.

Two grain size analyses of the marine clay resulted in the soil being classified as A-6 under the AASHTO Soil Classification System and CL under the Unified Soil Classification System (USCS).

Atterberg Limits tests on two samples from the marine clay determined moisture contents of approximately 30 and 31 percent and plastic limits of 18 and 19. For the two samples tested, the natural water contents were within 1% of the liquid limits and greater than the plastic limits; the calculated liquidity indexes (LI) were less than 1.0 and approximately 1.0. As the natural water contents are close to the liquid limits, the marine clay deposit is generally normally consolidated. Table 5-2 summarizes the results of Atterberg Limits test made from samples of the marine clay unit:

Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-IMC-103, 5D	Silty Clay	31.0	32	19	14	0.86
BB-IMC-104, 5D	Clayey Silt, trace sand	30.4	30	18	12	1.03

Table 5-2 Atterberg Limits Test Results – Marine Clay

5.4 Glacial Till

Based on drilling behavior we suspect a layer of gravelly, glacial till was encountered in boring BB-IMC-104 directly overlying bedrock. No samples were retrieved or SPT tests conducted. The encountered thickness was approximately 3.6 feet.

5.5 Bedrock

Bedrock at the south abutment was encountered and cored at depths of approximately 15.8 to 17.0 feet below ground surface (bgs) below the south approach. Bedrock below the existing north bridge approach was encountered and cored at depths of approximately 25.9 and 30.1 feet.

The bedrock at the site is identified as light greenish-grey, aphanitic to fine grained, predominately phyllite interbedded with metasandstone, hard, fresh to slightly weathered, joints closely spaced, typically moderated to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. The RQD of the bedrock was determined to range from 37 to 65 percent, correlating to a rock mass quality of poor to fair.

Two laboratory unconfined compressive strength tests conduced on bedrock samples yielded unconfined compressive strengths of 6.77 and 7.76 kips/square inch (ksi).

5.6 Groundwater

Groundwater was observed at depths ranging from approximately 7.0 feet to 8.0 feet bgs in three of the borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water levels indicated on the boring logs may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with tide cycles, seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the subsurface exploration program, the following bridge foundation alternatives were considered for the replacement bridge:

- Pile-supported integral abutments, with H-piles driven to bedrock at Abutment 1, and H-piles driven to bedrock with special pile tips or set in 1 to 3-foot bedrock sockets at Abutment 2.

- A Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) bridge with abutments consisting of geosynthetic reinforced fill supported on a Reinforced Soil Foundation (RSF) bearing on bedrock at Abutment 2 and a RSF bearing partially on the old pile foundation, marine sediments or marine clay at the Abutment 1.

- A pile-supported integral abutment at Abutment 1 in combination with a semi-integral, cantilever-type abutment founded on a spread footing on bedrock at Abutment 2.

- A precast concrete arch bridge supported on spread footings bearing on bedrock at Abutment 2 and a combination of piles to bedrock and a spread footing on bedrock at Abutment 1.

The Preliminary Design Report prepared by Becker Structural Engineers for MaineDOT Bridge Program and submitted January 2012 recommends a pile-supported integral abutment bridge with a precast concrete, voided slab superstructure. The proposed span length of the bridge is 56 feet.

For the purposes of this geotechnical report it is assumed that driven H-piles will be used to support abutments for an integral abutment bridge (IAB) with a span length of 56 feet. Design recommendations for this foundation alternative are discussed in detail in Section 7.0 - Geotechnical Design Recommendations. During final design, follow-on geotechnical design recommendations and parameters may need to be developed and provided to the designer based on the selected superstructure and span length.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

This section provides geotechnical design recommendations for H-pile supported integral bridge abutments.

7.1 Integral Abutment H-Piles

H-piles for support of the integral abutments should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Piles should be fitted with driving pile points to protect the tips and improve penetration into bedrock.

Subsurface conditions were explored for an estimated 58-foot span IAB bridge option. Bedrock was encountered at a depth of approximately 15.8 to 17.0 feet bgs at the proposed Abutment 2 (south abutment) location and approximately 25.9 to 30.1 feet bgs at proposed Abutment 1 (north abutment). Assuming a superstructure depth of 2.0 feet and an abutment height of 5 feet, we estimate free pile lengths of approximately 9 to 10 feet at Abutment 2 and approximately 19 to 23 feet at Abutment 1. This data is summarized Table 7-1 below.

Foundation & Relevant Borings	Approximate Depth to Bedrock From Ground Surface (feet)	Approximate Top of Bedrock Elevation (feet)	Range of Estimated Free Pile Lengths [1] (feet)	
Abutment 1 BB-IMC-103 BB-IMC-104	 25.9 30.1	 -15.4 -19.7	19.4	23.7
Abutment 2 BB-IMC-101 BB-IMC-102	 15.8 17.0	 -4.9 -6.0	8.9	10.0

[1] Free pile lengths assume a combined superstructure depth and abutment height of 7 feet (results in a bottom elevation of IAB abutment of +4.0 feet)

Table 7-1 Estimated Pile Lengths for Integral Abutments

Based on the data presented in Table 7-1, an integral abutment founded on H-piles driven behind a 1.75H:1V slope is ideal for the proposed Abutment 1, but considered marginally practicable at Abutment 2 where the estimated free pile length could potentially be only 9 feet if driven to bear on bedrock.

The MaineDOT and the University of Maine (UMaine) have investigated the performance of integral abutment bridges at sites with shallow bedrock and have monitored the instrumented the pile foundation at Nash Stream Bridge in Coplin Plantation, Maine. Evaluation of the field data from the research study indicates that integral abutment bridges with 'short' steel piles (defined as piles less than 13 feet) may not develop fixity but perform adequately and do not experience stresses larger than those seen by longer piles. The shortest pile instrumented by the researchers was a 14-foot long H-pile.

To accommodate integral abutment piles at the Mill Bridge site, the following design features are recommended:

- A minimum free pile length of 10 feet is recommended. Due to the shallow depth of bedrock and the need for adequate pile embedment to achieve the required pile equivalent length and fixity at the pile tip, piles at Abutment 2 may need to be socketed in bedrock. In designing rock-socketed H-pile sections we recommend drilling a rock socket a minimum of 1.0 feet below the calculated pile equivalent length in order to obtain pile behavior associated plastic stress redistribution and inelastic rotation in the pile. The designed rock socket may have a depth on the order of 1 to 3 feet. To provide a fixed condition at the pile tip, the bottom 1-foot of the rock socket should be tremie-filled with concrete with a minimum compressive strength of 5,000 pounds per square inch (psi). The upper portion of the rock socket is backfilled with granular soil to achieve the required free-length of pile.

- Short piles supporting integral abutments should be designed in accordance with AASHTO LRFD; the design example in *Integral Abutment Bridge Design Guidelines*, VTrans Structures Section, 2008; the design example found in Appendix B of Technical Report ME-01-7, UMaine, June 2005, *Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock – Phase I*; and Chapter 5 of Technical Report ME-01-7

- The need for bedrock sockets or special pile tips will be assessed by performing an L-Pile or FB-Pier analysis during final design. The geotechnical engineer should perform this evaluation. These analyses may either confirm the adequacy of a short pile or indicate the need to provide shallow bedrock sockets or a special pile tips at Abutment 2 in order to obtain adequate pile free length and a pinned or fixed condition at the pile tips.

- The L-Pile or FB-Pier analyses performed by the geotechnical engineer shall evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads and pile head displacements applied. The resulting bending moment in the pile would be

supplied to the structural engineer for evaluation. The structural resistance of the piles should be determined for compliance with the interaction equation by the structural engineer.

- If L-Pile analyses indicate the H-pile design does not achieve fixity and requires a pinned boundary condition at the pile tip, the piles may be fitted with Rock Injector HP-80500 Pile Points, manufactured by Associated Pile and Fitting (APF), LLC, or equivalent, to improve penetration and friction at the pile tips and support a pinned pile tip assumption. Special Provision 501 Foundation Piles – Rock Injector Pile Tip, is provided in Appendix D – Special Provisions.

- To assure the integrity of the abutment and bridge approach slopes, which provide lateral support to the pile groups, the stream velocity should be low, and the risk of scour action, wave action, storm surge and ice damage low.

In-place pile lengths for a span length of approximately 58 feet, considering a nominal 2-foot pile embedment in the pile cap, will range from approximately 12 to 17 feet. This data is summarized in Table 7-2 below:

Proposed Structure	Approximate Bedrock Elevation (feet)	Estimated Abutment/Pile Cap Bottom Elevation (feet)	Estimated Pile Embedment in Abutment (feet)	Estimated **In-place** Pile Lengths after cut-off (feet)
Abutment 1 BB-IMC-104	-19.7	4.0	2.0	26
Abutment 2 BB-IMC-101	-6.0	4.0	2.0	12

Table 7-2 Estimated In-place Pile Lengths for Piles

The pile lengths do not take into account consideration to accommodate locations where bedrock may be deeper than that encountered in the four borings, or the additional five feet of pile required for dynamic testing instrumentation or pile length needed to accommodate leads and driving equipment.

7.1.1 Strength Limit State Design

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

- compressive axial geotechnical resistance of individual piles bearing on bedrock
- structural resistance of individual piles in axial compression
- structural resistance of individual piles in combined axial loading and flexure

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

Since the H-piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. The analyses shall assign a free condition at the pile tip. As the proposed piles will be short and may not achieve fixity, the resistance for the piles should be determined for compliance with the interaction equation and checked for buckling.

The nominal compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD 6.9.4.1. Preliminary estimates of the factored structural axial compressive resistance of five H-pile sections were calculated and are provided in Table 7-3. It is the responsibility of the structural engineer to recalculate the nominal and factored pile structural compressive resistance (P_n) based on the "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e."

The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD 10.7.3.2.3 which states the *nominal* bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural resistance values obtained from LRFD 6.9.4.1 with a resistance factor for severe driving conditions applied. This limiting *nominal* bearing resistance is subsequently factored by a resistance factor, φ_{dyn}, of 0.65 considering a pile resistance determination method of dynamic pile testing with signal matching of at least two piles. The resulting, limiting factored pile geotechnical strength for the pile on rock is provided below in Table 7-3.

Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances were calculated using the resistance factor, $\varphi_{dyn,}$ of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of five H-piles sections for the strength limit state is provided in Table 7-3 below. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ϕ_c=0.50[1] (kips)	Controlling Geotechnical Resistance[2] φ_{dyn} = 0.65 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387	252	258	252
HP 12 x 74	544	354	403	354
HP 14 x 73	535	347	377	347
HP 14 x 89	652	424	416	416
HP 14 x 117	859	559	402	402

Table 7-3 Factored Axial Resistances for H-Piles for Strength Limit State Design

LRFD Article 10.7.3.2.3 states that the *nominal* axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for these site conditions the estimated factored axial pile resistances from the drivability analyses for the 14 x 89 and 14 x 117 sections are less than the controlling factored axial structural resistance per LRFD 10.7.3.2.3. Therefore, the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 7-3, above. The maximum applied factored axial pile load should not exceed the governing factored pile resistance shown in Table 7-3 above.

The piles shall also be checked for resistance against combined axial compression and flexure in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile group and pile group

[1] Calculated using a resistance factor, ϕ_c, for difficult driving conditions, an unbraced length (ℓ) of 0.5 feet and a K of 2.0. The piling may not achieve fixity, therefore the factored structural resistance may be controlled by combined the axial and flexural resistance of the pile.

[2] Based on guidance in LRFD 10.7.3.2.3., *Piles Driven to Hard Rock.*

movements/stability considering changes in soil conditions after scour due to the design flood event.

Extreme limit state design checks for the H-piles shall include pile axial bearing resistance, failure of the pile group by overturning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events.

Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

For the service and extreme limit states, resistance factors, ϕ, of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.1 and 10.5.5.3.

The nominal axial geotechnical piles resistance in the service and extreme limit state was calculated using the guidance in LRFD 10.7.3.2.3, *Piles Driven to Hard Rock*. The calculated factored axial structural, geotechnical and drivability resistances of five H-pile sections for the service and extreme limit states and are provided in Table 7-4. Supporting documentation is provided in Appendix C – Calculations.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance [3] (kips)	Controlling Nominal Geotechnical Resistance[4] (kips)	Drivability Resistance (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	774	387	397	387
HP 12 x 74	1089	544	620	544
HP 14 x 73	1069	535	580	535
HP 14 x 89	1304	652	640	640
HP 14 x 117	1719	859	618	618

Table 7-4 Factored Axial Resistance for H-Piles for Service and Extreme Limit State Design

[3] Calculated using a resistance factor of ϕ_c=1.0, an unbraced length (ℓ) of 0 feet and a K of 2.0. Short pile may not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

[4] Based on guidance in LRFD 10.7.3.2.3, *Piles Driven to Hard Rock*.

LRFD Article 10.7.3.2.3 states that the *nominal* axial compressive resistance of piles driven to hard rock shall not exceed the structural resistance with resistance factors for severe driving conditions. However, at this site the factored geotechnical pile resistances from the drivability analyses for the 14 x 89 and 14 x 117 pile sections are less than the nominal axial compressive resistance per LRFD 10.7.3.2.3. Therefore, it is recommended that the governing resistance used in service/extreme limit state design be those values provided in column "Governing Axial Pile Resistance (kips)" in Table 7-4.

7.1.3 Lateral Pile Resistance

In accordance with LRFD 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

Lateral loads will be reacted by plumb piles. We recommend the designer or geotechnical engineer perform a series of lateral pile resistance analyses to evaluate pile top deflections and bending stresses under strength limit state design lateral loads using L-Pile® software or FB-Pier software. These software programs analyze pile response under lateral loads where the nonlinear soil behavior is modeled using soil resistance (p-y) curves may be used. A secondary lateral pile analyses to determine maximum factored lateral loads permissible based on the allowable displacement criteria. The structural designer should evaluate the associated pile stresses under factored lateral loads.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 7-5 below. In general, the model developed should emulate the soil at the site by using the soil layers (referenced in Table 7-5 below by elevations) and appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. It is recommended that the analyses be conducted assuming a fixed pile-head boundary condition.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion lb/in^2 (lb/ft^2)	E_{50} for clays	Friction Angle
Misc. Fill	+2.0 to GS	Above	0.0723 (125)	60	-	-	32°
Recent Marine Sediments	-6.0 to 2.0	Below	0.0304 (53)	20	-	-	20°
Marine Clay (present only at Abutment 1)	-16 to -6	Below	0.0304 (53)	30	2.78 (400)	0.020	-
Glacial Till (present only at Abutment 1)	-16 to -19	Below	0.036 (63)	80	-	-	32°

Table 7-5 Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes will be not be required as part of the pile field quality control program unless pile behavior indicates the pile is not seated firmly on bedrock or if piles "walk" out of position. Any piles installed in bedrock sockets at Abutment 2 should be seated at the driving resistance specified for driven piles at Abutment 1.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

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

7.2 Integral Abutments

Integral abutments shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment 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.

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

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating the integral abutments will experience small movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.005, then the calculation of passive earth pressure should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. For designing the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6 below:

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

Table 7-6 Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. We recommend weep holes be constructed approximately 6 inches above Q1.1 (normal high water). Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT Bridge Design Guide (BDG).

Backfill within 10 feet of the abutments and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

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

7.3 Global Stability of Embankments

Proposed cross-section changes at the south and north bridge include widened and over-steepened roadway embankments. The designer has proposed 1.5H:1V slopes at some stations to minimize permanent wetland impacts. Stability analyses to determine factors of safety against global failure of the proposed 1.5H:1V and 1.75H:1V side slopes at Abutments 1 and 2 were conducted. The two critical cross sections selected for analysis and shown on Figures 1 and 2 of Appendix C – Calculations. The software used to conduct the stability analyses was GeoStudio Slope/W 6.20 which applied the Bishop method in the analyses. A minimum factor of safety of 1.3 is required in accordance with Federal Highway Administration (FHWA) Soils and Foundations Manual, 2006.

Evaluations of cross-section changes at both bridge approaches indicate that the minimum required factor of safety of 1.3 for slope stability is achievable with the use of 1.5H:1V and 1.75H:1V slopes as proposed in the preliminary plans. A more detailed discussion of the evaluations and recommendations follow:

7.3.1 Cross-Section Changes at Abutment 1

Slope stability analyses were conducted at the critical section behind Abutment 1 (Sta. 18+15). The borings encountered an approximately 7 to 11-foot thick layer of marine clay underlying the marine silt and embankment fills behind Abutment 1. The slope stability analysis at Sta. 18+15 is shown in Figure 3 of Appendix C – Calculations. The analysis indicates a factor of safety is 1.3 is achievable with 4-foot thick riprap slopes with a “toe-in” of the riprap of 5 feet below the streambed. To minimize permanent wetland impacts the “toe-in” may be overlain by native soils. The widened approaches and over steepened slopes will result in long-term consolidation/ settlement issues where a load greater than the existing overburden pressure is being applied. We therefore recommend that the widened approach embankments be constructed with heavy riprap at the toe of the slopes and then transition to plain riprap slope protection, as hydraulic conditions permit, to lessen the loading on the soft, weak foundation materials.

7.3.2 Cross-Section Changes at Abutment 2

The slope stability analysis at the critical section behind Abutment 2 (Sta. 19+00), is illustrated on Figure 4 of Appendix C. The analysis indicates the slope failure surfaces occur in the 1 to 2-foot thick layer of recent marine sediments left below the "toe-in" of the 4 foot thick riprap slope. The failure does not occur as sloughs in the over-steepened riprap section. Based on this analysis, we recommend that the toe of the riprap side slopes behind Abutment 2 be keyed-in an additional foot to the top of bedrock, at approximate Elev. -4.9 to -6.0 feet. The riprap illustrated on Figure 4 was modeled with a "toe-in" to elevation -5.0 feet. To minimize permanent wetland impacts the “toe-in” may be overlain by 6 to 12 inches of native streambed material.

A secondary, parametric analysis was run to determine if the use of heavy riprap contributes to the slope instability. GeoSlope analyses indicated that use of plain riprap will decrease the

loading on the soft, weak foundation soils and increase the factor of safety. We therefore recommend that the widened approach embankments be constructed with heavy riprap at the toe of the slopes and then transition to plain riprap slope protection, as hydraulic conditions permit, to lessen the loading on the soft, weak foundation materials.

7.4 Settlement

The recent marine sediments and the underlying marine clay are loose and/or soft in nature and are characterized as having a low bearing resistance. Furthermore, the materials are characteristically compressible and can be expected to consolidate where a load greater than the existing overburden pressure is being applied. Therefore, the cross-section changes consisting of widened and over-steepened slopes behind Abutments 1 and 2 and an approximate 2-foot grade raise of Mill Creek Road will result in some long-term consolidation of the soft foundation materials.

Elastic and consolidation settlement was computed at Station 18+50 which was selected as the critical cross section characterized by the thickest deposits of compressible materials. Post-construction settlements due to the widened slopes and raised approach fills are estimated to range from approximately 2 to 8 inches. These settlements will occur over a long period of time. The upper bounds of the computed settlements were computed at the toes of the widened slopes.

It is not recommended that any foundations be supported on shallow foundations bearing on the recent marine sediments or marine clay.

Any settlement of bridge abutments will be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch. The piles will be relatively short, therefore downdrag loads due to settlement of the compressible foundation soils are anticipated to be negligible.

7.5 Frost Protection

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

Foundations placed on fill side slopes should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Islesboro has a design freezing index of approximately 1300 F-degree days. An assumed water content of 20% was used for coarse grained soils within the intertidal zone or subject to groundwater seeps observed at the site. These components correlate to a frost depth of 5.3 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Islesboro was assigned a design freezing index of approximately 1256 F-degree days. An assumed water content of 20% was used for coarse grained soils above the water table. These components correlate to a frost depth of approximately 5.9 feet. We recommend

foundations be designed with an embedment of 5.5 feet for frost protection. See Appendix C – Calculations for supporting documentation.

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

7.6 Scour and Riprap

The PDR states that the stream velocities at the Mill Bridge are very slow and controlled and related to the flow and ebb of the tides. Therefore, scour has been determined by the designer to not be a concern and no further scour analysis in anticipated. Accordingly, grain size analyses were not specifically conducted for the purpose of generating grain size curves to determine parameters for scour analyses.

In accordance with Section 2.3.11.3 of the MaineDOT BDG, bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap, and bridges located immediately on the ocean should use heavy riprap. The top of the riprap should be located at a minimum elevation of 2 feet above the Mean Higher High Water (MHHW) elevation of 5.85 feet. Anecdotal information indicated a recent high tide event of Elev. 7.1 feet. In accordance with the BDG, consideration should be given to placing riprap higher than the MHHW due to waves and wave run-up.

Based on slope stability analyses we recommend that widened approach embankments be constructed with 4-foot thick riprap slope treatment with a 4-foot square "key-in" constructed 1 foot below the streambed. The slopes may be constructed with heavy riprap at the toe, then transition to plain riprap slope protection as soon as hydraulic conditions permit to lessen the loading on the soft, weak foundation materials.

Stone riprap shall conform to item number 703.26 and 703.28 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. 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).

7.7 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak Ground Acceleration coefficient (PGA) = 0.058g

- Site Class D (based on an average N-value for the upper 100 ft of the soil profile greater than 15 bpf and less than 50 bpf, using steps in LRFD Table C3.10.3.1-1)
- Acceleration coefficient (As) = 0.094g
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.207g
- Design spectral acceleration coefficient at 1.0-second period, (S_{D1})= 0.097g
- Seismic Zone 1, based on a $S_{D1} \leq 0.15$g

7.8 Construction Considerations

Construction of the abutments will require soil excavation and pile driving. Cofferdams or temporary lateral earth support systems may be required to permit construction of driven pile foundations and abutments.

The removal of fill soils will result in the exposure of naturally deposited pockets of potentially sensitive clays and silts. These soils will be susceptible to disturbance and rutting as a result of exposure to water or 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 or ¾ inch stone.

Furthermore, the recent marine sediments and marine clays 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. Water should be controlled by pumping from sumps.

8.0 CLOSURE

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

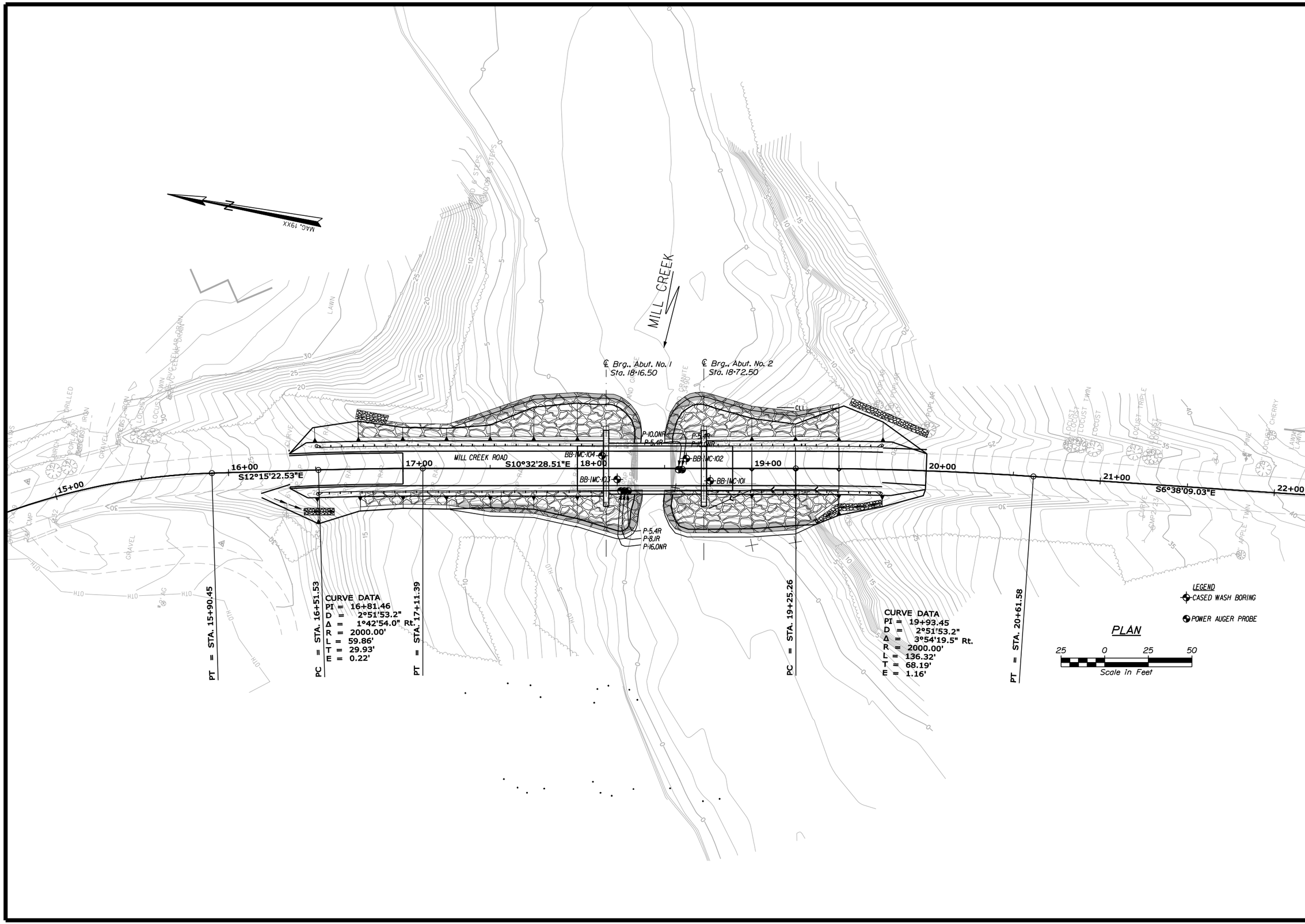
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map
Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Islesboro, Maine
Waldo County
WIN 19285.00 Federal No. AC-STP-1928(500)X
USGS 7.5' Series Topographic
Islesboro Quadrangle
DeLORME Map 14, Grid C5

Map Scale 1:24000

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



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-STP-1928(500)X

WIN 19285.00

BRIDGE NO. 3490

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	------	------	------
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	DEC 2011
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

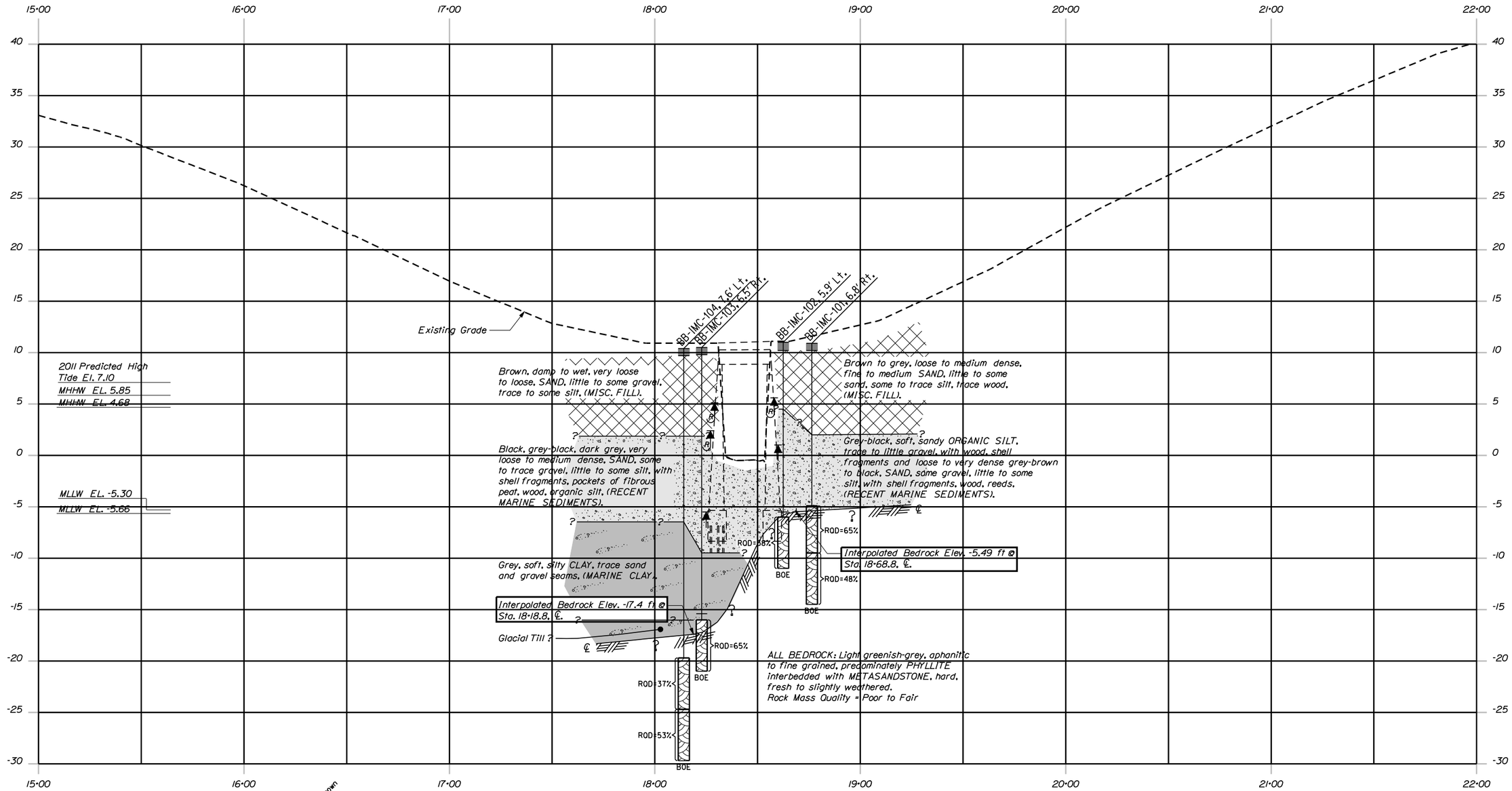
DATE

MILL BRIDGE
MILL CREEK
ISLESBORO WALDO COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\geotech\msla\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:3/30/2012



LEGEND

Boring No. Offset, if shown

Pavement Thickness if applicable

Strata Interface

Boring

ROD= Rock Quality Designation for Rock Core Sample

BOE= Bottom Of Exploration

No Refusal

Refusal

PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-STP-1928(500)X

WIN 19285.00

BRIDGE NO. 3490

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L.KRUSINSKI	DEC 2011
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MILL BRIDGE
MILL CREEK
ISLESBORO WALDO COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\00\geotech\msla\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:3/30/2012

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:3/30/2012

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

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-104
WIN: 19285.00

Driller:	Maine Test Borings	Elevation (ft.)	10.4	Auger ID/OD:	Solid Stem
Operator:	Enos/Barlow	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/8/11; 09:30-3:20	Drilling Methods:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	18+14.1, 7.4 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	7.0 ft on 11/10/11
Hammer Efficiency Factor: .6		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (Ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (Ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (Ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			ASPHALT.	
								9.70		0.70 Brown, damp, fine to medium SAND, some gravel, trace to little silt. (Granular Fill).	
	1D	24/6	2.00 - 4.00	4/3/3/5	6	6		7.40		3.00 Brown, damp, loose, fine SAND, some fine gravel, little silty, trace medium to coarse sand; appears reworked. (Till Fill).	
5	2D	24/13	5.00 - 7.00	1/1/11(12")	2	2		1.90		8.50 Brown, wet, very loose, fine to medium SAND, some fine gravel, little to some silt, trace coarse sand; appears reworked. (Till Fill).	G#12234c A-4, SM WC=16.8%
10	3D	24/5	10.00 - 12.00	36/15/4/1	19	19				Black, medium dense, fine to coarse SAND, some fine gravel, little silt with black, organic SILT, little fine to coarse sand in tip of spoon. Blow counts suggest driving gravel ahead of spoon to approximately 11 ft. (Recent Marine Sediments).	14 / 17 / 15 / 13 / 16
15	4D	24/18	15.00 - 17.00	3/1/2/2	3	3		-6.20		Dark grey, very loose, fine to medium SAND, little silt, trace coarse sand with fibrous peat pockets, shell fragments and wood. (Recent Marine Sediments).	19 / 15 / 12 / 30 / 17
								-16.10		26.50 Grey, SILT and CLAY, trace fine sand. (Marine Clay). Coarser material, likely sand and gravel seams, encountered between 17.0 and 25.0 ft based upon drilling behavior. Failed tube attempt; no recovery. Grey, Silty CLAY on exterior of tube and drill cuttings sand seams apparent from drilling behavior. (Marine Clay). Gravelly seams encountered when cleaning out from 20.0 to 23.0 ft, therefore no vane shear tests attempted at 23.0 ft. Grey, soft, Silty CLAY. (Marine Clay). Drilling behavior suggests change to gravelly material at about 26.5 ft. (likely Glacial Till).	MU 24/0 20.00 - 22.00 Hydraulic Push
25	5D	24/10	25.00 - 27.00	1-12"/2/5	2	2					G#12234d A-6, CL LL=30 PL=18 PI=12 WC=30.4%
	R1	60/60	30.10 - 35.10	RQD = 37%				-19.70		30.10 Top of Bedrock at Elev. -19.7 ft. Telescoped NW Casing into top of rock to 30.1 ft prior to coring. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Closely spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Core highly fractured with clay infilling from 30.8 to 31.0, 32.9 to 33.1 and 34.4-35.1 ft bgs. Rock Mass Quality: Poor. (ROCKS OF ISLESBORO) RI: Core Times (min:sec): 30.1-31.1 ft (2:10); 31.1-32.1 ft (1:45); 32.1-33.1 ft (1:30); 33.1-34.1 ft (1:35); 34.1-35.1 ft (1:20): 100% Recovery.	UCT$_{50}$=7.760 psi
35	R2	60/60	35.10 - 40.10	RQD = 53%				-29.70		R2: Bedrock: same as R1, except joints moderately spaced below 37.1 ft. Core highly fractured with clay infilling from 35.1 to 37.1 ft bgs. Rock Mass Quality: Fair. (ROCKS OF ISLESBORO) RI: Core Times (min:sec): 35.1-36.1 ft (1:35); 36.1-37.1 ft (1:25); 37.1-38.1 ft (1:25); 38.1-39.1 ft (1:25); 39.1-40.1 ft (1:30): 100% Recovery. Bottom of Exploration at 40.10 feet below ground surface.	
40											
45											
50											

Remarks:
UCT_{50} = Peak compressive strength from Unixial Compressive Strength Test.
Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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-IMC-104

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

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-103
WIN: 19285.00

Driller:	Maine Test Borings	Elevation (ft.)	10.5	Auger ID/OD:	Solid Stem
Operator:	Enos/Barlow	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/10/11; 10:10-15:20	Drilling Methods:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	18+22.4, 6.5 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	8.0 ft on 11/10/11
Hammer Efficiency Factor: .6		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (Ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (Ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (Ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			ASPHALT.	
								9.80		0.70 Brown, damp, fine to coarse SAND, some gravel, trace to little silt. (Granular Fill).	
	1D	24/13	2.00 - 4.00	4/5/4/6	9	9		7.40		3.10 Brown, damp, loose, fine to coarse SAND, some fine gravel, trace silt. (Granular Fill).	
5	2D	24/17	5.00 - 7.00	2/3/1/2	4	4				8.50 Brown, moist to wet, very loose, fine to medium SAND, some fine gravel, little to some silt, appears reworked. (Till Fill).	
10	3D	24/7	10.00 - 12.00	3/3/12/13	15	15		2.00		Grey-black, wet, medium dense, silty fine to medium SAND, little to some fine gravel with woody reeds and numerous shell fragments. (Recent Marine Sediments).	G#12234b A-4, SC LL=28 PL=20 PI=8 WC=21.1%
15	4D	24/6	15.00 - 17.00	4/4/3/5	7	7		-9.50		20.00 Grey-black, loose, fine to medium SAND, some fine gravel, some silt with shell fragments and occasional reeds and wood. (Recent Marine Sediments).	
20	5D	24/19	20.00 - 22.00	1/1/1/1	2	2				Grey, soft, Silty CLAY with black streaks in upper 6 inches of sample. (Marine Clay). Failed tube attempt; no recovery. Grey, silty CLAY on outside of tube.	G#12234d A-6, CL LL=32 PL=19 PI=14 WC=31.0%
	MU	24/0	22.00 - 24.00	Hydraulic Push							
25	6D R1	22.8/ 22.8	24.00 - 25.90	push through 24/36/11 blows				-15.40		Grey, Silty CLAY, trace fine sand with sand and gravel seams throughout. (Marine Clay). Gravelly sand seam at 24.5 ft; able to push through. Vane raw torque readings: V1: 240/42 in-lbs. Unable to push through.	
			25.90 - 26.50	RQD = 65%			NQ-2	-21.00		25.90 Top of Bedrock at Elev. -15.4 ft. Telescoped NW Casing into top of rock to 26.5 prior to coring. Spun NW Casing from 25.9 to 26.5 ft bgs. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Moderately spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding calcite veins and breaks. Rock Mass Quality: Fair. (ROCKS OF ISLESBORO) R1: Core Times (min:sec): 26.5-27.5 ft (1:35); 27.5-28.5 ft (1:20); 28.5-29.5 ft (1:15); 29.5-30.5 ft (1:15); 30.5-31.5 ft (2:10): 100% Recovery. Bottom of Exploration at 31.50 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:
Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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-IMC-103

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

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-102
WIN: 19285.00

Driller:	Maine Test Borings	Elevation (ft.)	11.0	Auger ID/OD:	Solid Stem
Operator:	Enos/Barlow	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/9/11; 08:50-12:20	Drilling Methods:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	18+42.5, 3.9 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	7.0 ft on 11/9/11
Hammer Efficiency Factor: .6		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (Ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (Ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (Ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			ASPHALT.	
								10.20		0.80 Brown-black, damp, fine to coarse SAND, little fine gravel, trace silt possibly decomposed asphalt. (Granular Fill).	
	1D	24/13	2.00 - 4.00	6/7/7/4	14	14		7.50		3.50 Grey, medium dense, fine to medium SAND, little fine gravel, little silt. (Granular Fill).	
5	2D	24/8	5.00 - 7.00	2/7/11/2	18	18		7.00		4.00 Grey, damp, fine to coarse SAND, little fine gravel, little silt. (Granular Fill).	
								4.00		7.00 Brown, medium dense, fine to medium SAND, some fine gravel, little silt with wood. (Till Fill). Grey, SILT and CLAY with shell fragments in tip of spoon. (Recent Marine Sediments).	
10	3D	24/7	10.00 - 12.00	2/2/3/6	5	5				Grey-brown, loose, fine to coarse SAND, some gravel, little to some silt, with wood, reeds and shell fragments zones of mostly sand and gravel and zones of mostly silt and clay. (Recent Marine Sediments).	G#12233a A-2-4, GM WC=17.6%
15	4D/4 R1	24/14 / 60/60	15.00 - 17.00 / 17.00 - 22.00	4/1/16/40 / RQD = 38%	17	17	SPUN / NQ-2	-5.50 / -6.00		40 (15.0-16.5 ft) Dark grey, medium dense, fine to coarse SAND, some gravel, little silt with wood, reeds and shell fragments; strong organic odor. (Recent Marine Sediments on Bedrock). 40-R1 (16.5-17.0 ft) Decomposed/weathered rock. Telescoped NW Casing to top of rock at 17.0 ft prior to coring. 17.00 Top of Intact Bedrock at Elev. -6.0 ft. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Closely spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding calcite veins and breaks. Core highly fractured with clay infilling from 18.1 to 19.0 and 19.6 to 20.5 ft bgs. Rock Mass Quality: Poor. (ROCKS OF ISLESBORO) R1: Core Times (min:sec): 17.0-18.0 ft (1:40); 18.0-19.0 ft (1:45); 19.0-20.0 ft (1:45); 20.0-21.0 ft (1:40); 21.0-22.0 ft (1:55): 100% Recovery. Bottom of Exploration at 22.00 feet below ground surface.	
20								-11.00			
25											
30											
35											
40											
45											
50											

Remarks:
Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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-IMC-102

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

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-101
WIN: 19285.00

Driller:	Maine Test Borings	Elevation (ft.)	10.9	Auger ID/OD:	Solid Stem
Operator:	Enos/Barlow	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Mobile B53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/9/11-11/10/11	Drilling Methods:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	18+76.2, 6.8 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: .60		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (Ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (Ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (Ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA			ASPHALT.	
								10.20		0.70 Brown, damp, fine to medium SAND, little gravel, trace silt. (Granular Fill).	
	1D	24/17	2.00 - 4.00	10/5/4/4	9	9		7.70		3.20 Grey-brown, damp, loose, fine to coarse SAND, some gravel, trace silt. (Granular Fill).	
5	2D	24/17	5.00 - 7.00	2/2/3/6	5	5				8.00 Brown, moist, loose, fine to medium SAND, little fine gravel, little silt. (Till Fill).	
10	3D	24/20	10.00 - 12.00	1-12"/2/1	2	2		1.90		Brown, moist, loose, fine to medium SAND, some fine gravel, little to some silt, appears reworked (Till Fill).	G#12233b A-4, ML WC=18.3%
15	4D R1	4/7 / 60/55	15.00 - 15.75 / 15.75 - 20.38	27/50(3") / RQD = 64%	---		SPUN / NQ-2	-4.90		9.00 Grey-black, very dense, fine to medium SAND, some gravel, little to some silt with significant shell and weathered rock fragments. (Recent Marine Sediments Bedrock). Telescope NW Casing into top of rock prior to coring. Top of Bedrock at Elev. -4.9 ft. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Moderately spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding calcite veins and breaks. Rock Mass Quality: Fair. (ROCKS OF ISLESBORO) R1: Core Times (min:sec): 15.8-16.8 ft (1:30); 16.8-17.8 ft (1:25); 17.8-18.8 ft (1:25); 18.8-19.8 ft (1:25); 19.8-20.8 ft (1:20): 100% Recovery. R2: Bedrock: Same as R1, except fresh. Rock Mass Quality: Poor. R2: Core Times (min:sec): 20.4-21.4 ft (1:55); 21.4-22.4 ft (1:35); 22.4-23.4 ft (1:15); 23.4-24.4 ft (1:10); 24.4-25.4 ft (1:10): 100% Recovery. Bottom of Exploration at 25.40 feet below ground surface.	UCT$_{50}$=6.770 psi
20	R2	60/60	20.40 - 25.40	RQD = 48%				-14.50			
25											
30											
35											
40											
45											
50											

Remarks:
UCT_{50} = Peak compressive strength from Unixial Compressive Strength Test
Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory testing.

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-IMC-101

State of Maine - Department of Transportation
Power Auger Probe Summary Sheet

Town(s): Islesboro **Project Number: 19285.00**

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Water Depth (Ft	Comments Date :11/10/2011
18+25	6.5 Rt.			16.0		
18+27	6.5 Rt.		8.1			
18+29	6.5 Rt.		5.4			
18+58	5.9 Lt.		5.4			
18+60	5.9 Lt.			10.0		

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-STP-1928(500)X
WIN 19285.00
BRIDGE NO. 3490
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L.KRUSINSKI	DEC 2011
CHECKED-REVIEWED	T.WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MILL BRIDGE
MILL CREEK
ISLESBORO WALDO COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs and Power Auger Probes

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-101
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.9	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/9/11-11/10/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+76.2, 6.8 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	10.20	ASPHALT. 0.70	
									Brown, damp, fine to medium SAND, little gravel, trace silt, (Granular Fill).	
	1D	24/17	2.00 - 4.00	10/5/4/4	9	9			Grey-brown, damp, loose, fine to coarse SAND, some gravel, trace silt, (Granular Fill).	
								7.70	3.20	
									Brown, moist, loose, fine to medium SAND, little fine gravel, little silt, (Till Fill).	
5	2D	24/17	5.00 - 7.00	2/2/3/6	5	5			Brown, moist, loose, fine to medium SAND, some fine gravel, little to some silt, appears reworked (Till Fill).	G#12233a A-4, ML WC=18.3%
								1.90	9.00	
10	3D	24/20	10.00 - 12.00	1-12"/2/1	2	2	7		Grey-black, soft, sandy ORGANIC SILT, trace to little fine gravel with wood and shell fragments, strong organic odor, (Recent Marine Sediments).	G#12233b A-4, ML LL=33 PL=25 PI=8 WC=34.3%
							8			
							17			
							51			
							46			
15	4D	9/7	15.00 - 15.75	27/50(3")	---		SPUN		Grey-black, very dense, fine to medium SAND, some gravel, little to some silt with significant shell and weathered rock fragments, (Recent Marine Sediments on Bedrock).	
	R1	55/55	15.80 - 20.38	RQD = 65%			NQ-2	-4.90	15.80	
									Telescope NW Casing into top of rock prior to coring. Top of Bedrock at Elev. -4.9 ft. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Moderately spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding breaks. Rock Mass Quality: Fair. [ROCKS OF ISLESBORO] R1:Core Times (min:sec) 15.8-16.8 ft (1:30) 16.8-17.8 ft (1:25) 17.8-18.8 ft (1:25) 18.8-19.8 ft (1:25) 19.8-20.4 ft (1:20) 100% Recovery R2: Bedrock: Same as R1, except fresh, Rock Mass Quality: Poor. R2:Core Times (min:sec)	UCTq_p=6,770 psi
20	R2	60/60	20.40 - 25.40	RQD = 48%						
25										

Remarks:

UCTq_p = Peak compressive strength from Uniaxial Compressive Strength Test

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory testing.

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-IMC-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-101
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.9	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/9/11-11/10/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+76.2, 6.8 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								-14.50		20.4-21.4 ft (1:55) 21.4-22.4 ft (1:35) 22.4-23.4 ft (1:15) 23.4-24.4 ft (1:10) 24.4-25.4 ft (1:10) 100% Recovery 25.40 **Bottom of Exploration at 25.40 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

$UCTq_p$ = Peak compressive strength from Uniaxial Compressive Strength Test

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory testing.

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: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-102
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	11.0	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/9/11; 08:50-12:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+62.5, 5.9 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	7.0 ft on 11/9/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	10.20	ASPHALT. 0.80	
									Brown-black, damp, fine to coarse SAND, little fine gravel, trace silt; possibly decomposed asphalt, (Granular Fill).	
	1D	24/11	2.00 - 4.00	6/7/7/4	14	14			Brown, damp, medium dense, fine to medium SAND, little fine gravel, little silt, (Granular Fill).	
								7.50	3.50	
								7.00	Grey, damp, fine to coarse SAND, little fine gravel, trace silt, (Granular Fill). 4.00	
5	2D	24/8	5.00 - 7.00	2/7/11/2	18	18			Brown, moist, medium dense, fine to medium SAND, some fine gravel, little silt with wood, (Till Fill).	
								4.00	7.00	
									Grey, SILT and CLAY with shell fragments in tip of spoon, (Recent Marine Sediments).	
10	3D	24/7	10.00 - 12.00	2/2/3/6	5	5	5		Grey-brown, loose, fine to coarse SAND, some gravel, little to some silt, with wood, reeds and shell fragments; zones of mostly sand and gravel and zones of mostly silt and clay. (Recent Marine Sediments).	G#12233c A-2-4, GM WC=17.6%
							6			
							20			
							36			
							16			
15	4D/A	24/14	15.00 - 17.00	4/1/16/40	17	17	SPUN		4D (15.0-16.5 ft) Dark grey, medium dense, fine to coarse SAND, some gravel, little silt with wood, reeds and shell fragments; strong organic odor, (Recent Marine Sediments on Bedrock).	
							SPUN	-5.50	16.50	
	R1	60/60	17.00 - 22.00	RQD = 38%			NQ-2	-6.00	4D-A (16.5-17.0 ft) Decomposed/weathered rock. Telescoped NW Casing to top of rock at 17.0 ft prior to coring. 17.00	
20								-11.00	Top of intact Bedrock at Elev. -6.0 ft. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Closely spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding calcite veins and breaks. Core hightly fractured with clay infilling from 18.1 to 19.0 and 19.6 to 20.5 ft bgs. Rock Mass Quality: Poor. [ROCKS OF ISLESBORO] R1:Core Times (min:sec) 17.0-18.0 ft (1:40) 18.0-19.0 ft (1:45) 19.0-20.0 ft (1:45)	
25										

Remarks:

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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-IMC-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-102
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	11.0	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/9/11; 08:50-12:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+62.5, 5.9 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	7.0 ft on 11/9/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										20.0-21.0 ft (1:40) 21.0-22.0 ft (1:55) 100% Recovery 22.00 **Bottom of Exploration at 22.00 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-103
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.5	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/10/11; 10:10-15:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+22.8, 6.5 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	8.0 ft on 11/10/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.80	ASPHALT. 0.70	
									Brown, damp, fine to coarse SAND, some gravel, trace to little silt, (Granular Fill).	
	1D	24/13	2.00 - 4.00	4/5/4/6	9	9			Brown, damp, loose, fine to coarse SAND, some fine gravel, trace silt, (Granular Fill).	
								7.40	3.10	
									Brown, moist, loose, fine to medium SAND, little to some fine gravel, little silt; appears reworked, (Till Fill).	
5	2D	24/17	5.00 - 7.00	2/3/1/2	4	4			Brown, moist to wet, very loose, fine to medium SAND, some fine gravel, little to some silt, appears reworked (Till Fill).	
								2.00	8.50	
10	3D	24/7	10.00 - 12.00	3/3/12/13	15	15	35		Grey-black, wet, medium dense, silty fine to medium SAND, little to some fine gravel with wood, reeds and numerous shell fragments, (Recent Marine Sediments).	G#12234a A-4, SC LL=28 PL=20 PI=7 WC=21.1%
							31			
							19			
							26			
							20			
15	4D	24/6	15.00 - 17.00	4/4/3/5	7	7	24		Grey-black, loose, fine to medium SAND, some fine gravel, some silt with shell fragments and occasional reeds and wood, (Recent Marine Sediments).	
							23			
							34			
							50			
							34			
20	5D	24/19	20.00 - 22.00	1/1/1/1	2	2		-9.50	20.00	
									Grey, soft, Silty CLAY with black streaks in upper 6 inches of sample. (Marine Clay).	G#12234b A-6, CL LL=32 PL=19 PI=14 WC=31.0%
	MU	24/0	22.00 - 24.00	Hydraulic Push					Failed tube attempt, no recovery. Grey, silty CLAY on outside of tube.	
25	6D	22.8/22.8	24.00 - 25.90	push through					Grey, Silty CLAY, trace fine sand with sand and gravel seams throughout. (Marine Clay).	

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-103
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.5	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/10/11; 10:10-15:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+22.8, 6.5 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	8.0 ft on 11/10/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	V1		24.63 - 25.00	Su=636/111 psf						Gravelly sand seam at 24.5 ft; able to push through. Vane raw torque readings: V1: 240/42 in-lbs Vane fetches up at 25.7 ft; unable to push through.	
								-15.40		25.90	
	R1	60/60	26.50 - 31.50	RQD = 65%			NQ-2			Top of Bedrock at Elev. -15.4 ft. Telescope NW Casing into top of rock to 26.5 prior to coring. Spun NW Casing from 25.9 to 26.5 ft bgs. R1:Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Moderately spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Some cross-bedding calcite veins and breaks. Rock Mass Quality: Fair. [ROCKS OF ISLESBORO] R1:Core Times (min:sec) 26.5-27.5 ft (1:35) 27.5-28.5 ft (1:20) 28.5-29.5 ft (1:15) 29.5-30.5 ft (1:15) 30.5-31.5 ft (2:10) 100% Recovery	
30								-21.00			
35										31.50	
40										**Bottom of Exploration at 31.50 feet below ground surface.**	
45											
50											

Remarks:

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-104
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.4	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/8/11; 09:30-3:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+14.1, 7.6 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	7.0 ft on 11/10/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.70	ASPHALT. 0.70	
									Brown, damp, fine to medium SAND, some gravel, trace to little silt, (Granular Fill).	
	1D	24/6	2.00 - 4.00	4/3/3/5	6	6		7.40	3.00	
									Brown, damp, loose, fine SAND, some fine gravel, little silt, trace medium to coarse sand; appears reworked, (Till Fill).	
5	2D	24/13	5.00 - 7.00	1/1/1(12")	2	2			Brown, wet, very loose, fine to medium SAND, some fine gravel, little to some silt, trace coarse sand; appears reworked, (Till Fill).	G#12234c A-4, SM WC=16.8%
								1.90	8.50	
10	3D	24/5	10.00 - 12.00	36/15/4/1	19	19	14		Black, medium dense, fine to coarse SAND, some fine gravel, little silt with black, organic SILT, little fine to coarse sand in tip of spoon. Blow counts suggest driving gravel ahead of spoon to approximately 11 ft, (Recent Marine Sediments).	
							17			
							15			
							13			
							16			
15	4D	24/18	15.00 - 17.00	3/1/2/2	3	3	19		Dark grey, very loose, fine to medium SAND, little silt, trace coarse sand with fibrous peat pockets, shell fragments and wood, (Recent Marine Sediments).	
							15	-6.20	16.60	
							12		Grey, SILT and CLAY, trace fine sand, (Marine Clay). Coarser material, likely sand and gravel seams, encountered between 17.0 and 25.0 ft based upon drilling behavior.	
							30			
							17			
20	MU	24/0	20.00 - 22.00	Hydraulic Push					Failed tube attempt, no recovery. Grey, Silty CLAY on exterior of tube and drill cuttings; sand seams apparent from drilling behavior, (Marine Clay).	
									Gravelly seams encountered when cleaning out from 20.0 to 23.0 ft, therefore no vane shear tests attempted at 23.0 ft.	
25										

Remarks:

UCTq_p = Peak compressive strength from Uniaxial Compressive Strength Test.

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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: Mill Bridge #3490 carries Mill Creek Road over Mill Creek
Location: Islesboro, Maine

Boring No.: BB-IMC-104
WIN: 19285.00

Driller:	Maine Test Borings	**Elevation (ft.)**	10.4	**Auger ID/OD:**	Solid Stem
Operator:	Enos/Barlow	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/8/11; 09:30-3:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	18+14.1, 7.6 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	7.0 ft on 11/10/11
Hammer Efficiency Factor:	.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/10	25.00 - 27.00	1-12"/2/5	2	2				Grey, soft, Silty CLAY, (Marine Clay).	G#12234d A-6, CL LL=30 PL=18 PI=12 WC=30.4%
								-16.10		26.50	
										Drilling behavior suggests change to gravelly material at about 26.5 ft, likely Glacial Till.	
30	R1	60/60	30.10 - 35.10	RQD = 37%			NQ-2	-19.70		30.10	
										Top of Bedrock at Elev. -19.7 ft. Telescoped NW Casing into top of rock to 30.1 ft prior to coring. R1: Bedrock: Light greenish-grey, aphanitic to fine grained, predominately PHYLLITE interbedded with METASANDSTONE, hard, fresh to slightly weathered. Some banding of minerals apparently aligned with original bedding. Closely spaced, typically moderate to high angle, planar, slightly rough, fresh to slightly discolored, tight to partially open joints also aligned with original bedding. Core highly fractured with clay infilling from 30.8 to 31.0, 32.9 to 33.1 and 34.4-35.1 ft bgs. Rock Mass Quality: Poor. [ROCKS OF ISLESBORO] R1:Core Times (min:sec) 30.1-31.1 ft (2:10) 31.1-32.1 ft (1:45) 32.1-33.1 ft (1:30) 33.1-34.1 ft (1:35) 34.1-35.1 ft (1:20) 100% Recovery	$UCTq_p$=7,760 psi
35	R2	60/60	35.10 - 40.10	RQD = 53%							
										R2: Bedrock: same as R1, except joints moderately spaced below 37.1 ft. Core highly fractured with clay infilling from 35.1 to 37.1 ft bgs. Rock Mass Quality: Fair. R2:Core Times (min:sec) 35.1-36.1 ft (1:35) 36.1-37.1 ft (1:25) 37.1-38.1 ft (1:25) 38.1-39.1 ft (1:25) 39.1-40.1 ft (1:30) 100% Recovery	
40								-29.70		40.10	
45											
50											

Bottom of Exploration at 40.10 feet below ground surface.

Remarks:

$UCTq_p$ = Peak compressive strength from Uniaxial Compressive Strength Test.

Field visual descriptions of soil samples were not altered to reflect the results of subsequent laboratory tests.

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

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

State of Maine - Department of Transportation

Power Auger Probe Summary Sheet

Town(s): Islesboro Project Number: 19285.00

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Water Depth (Ft)	Comments Date :11/10/2011
18+25	6.5 Rt.			16.0		
18+27	6.5 Rt.		8.1			
18+29	6.5 Rt.		5.4			
18+58	5.9 Lt.		5.4			
18+60	5.9 Lt.			10.0		

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Islesboro Work Number: 19285.00

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-IMC-101, 2D	18+76.2	6.8 Rt.	5.0-7.0	12233a		18.3			ML	A-4	IV
BB-IMC-101, 3D	18+76.2	6.8 Rt.	10.0-12.0	12233b		34.3	33	8	ML	A-4	IV
BB-IMC-102, 3D	18+62.5	5.9 Lt.	10.0-12.0	12233c		17.6			GM	A-2-4	II
BB-IMC-103, 3D	18+22.8	6.5 Rt.	10.0-12.0	12234a		21.1	28	7	SC	A-4	III
BB-IMC-103, 5D	18+22.8	6.5 Rt.	20.0-22.0	12234b		31.0	32	14	CL	A-6	III
BB-IMC-104, 2D	18+14.1	7.6 Lt.	5.0-7.0	12234c		16.8			SM	A-4	III
BB-IMC-104, 5D	18+14.1	7.6 Lt.	25.0-27.0	12234d		30.4	30	12	CL	A-6	III

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

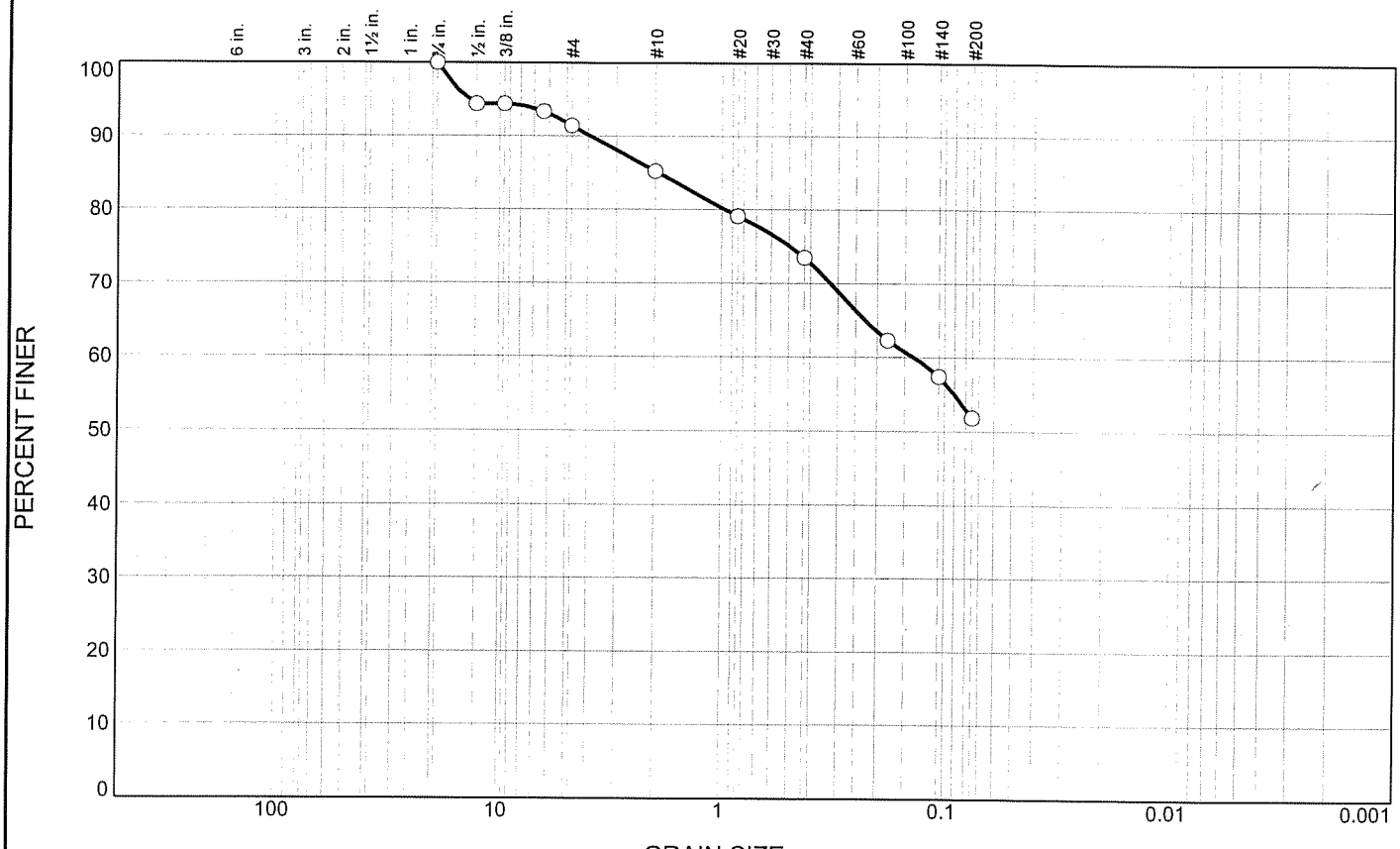
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.6	6.2	11.7	21.6	51.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	94.4		
3/8"	94.4		
1/4"	93.4		
#4	91.4		
#10	85.2		
#20	79.1		
#40	73.5		
#80	62.3		
#140	57.4		
#200	51.9		

* (no specification provided)

Soil Description

sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.9391 D_{60}= 0.1374 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content: 18.3%

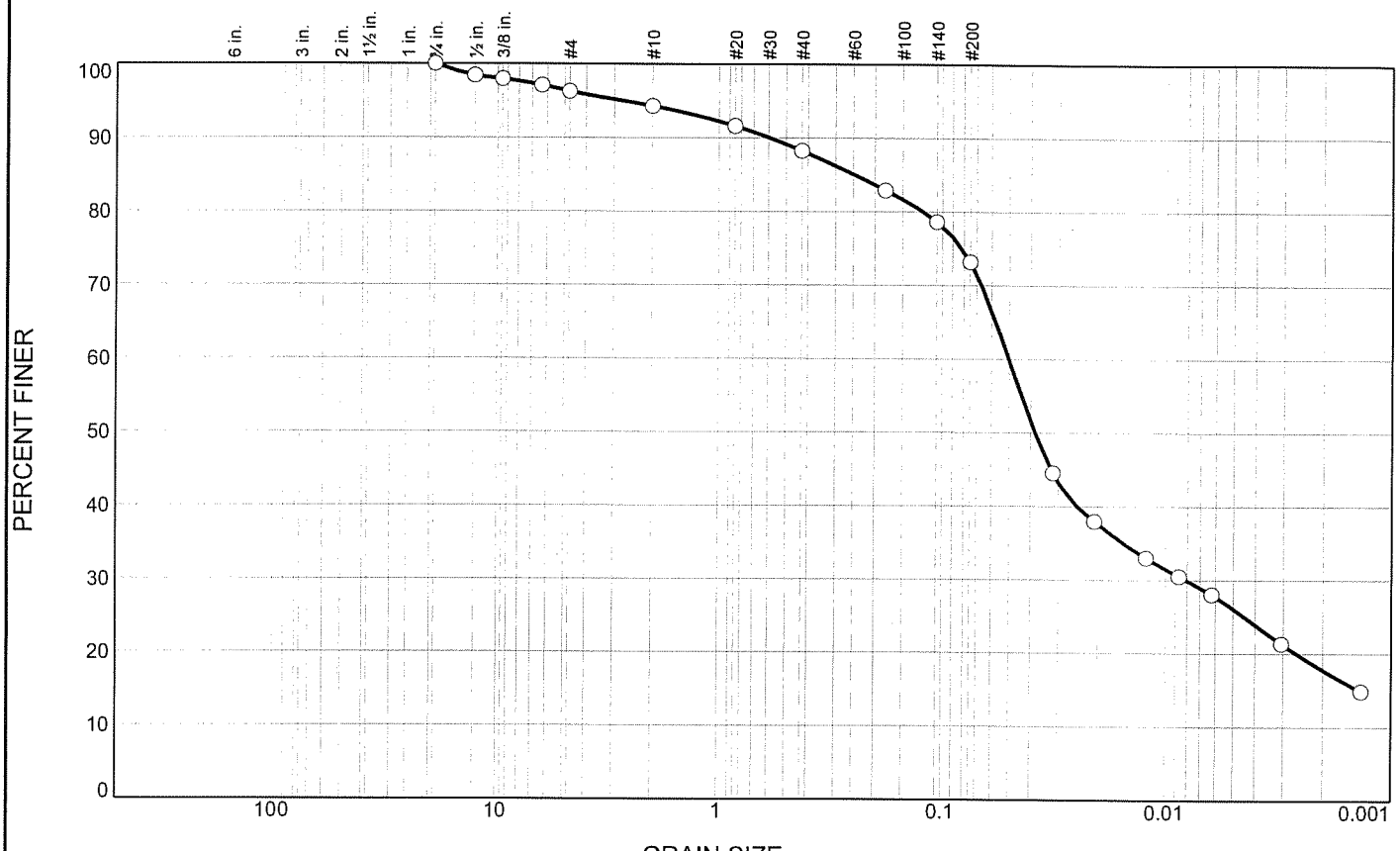
Sample No.: 2D **Source of Sample:** BB-IMC-101 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 5'-7'

R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12233a

Tested By: DCH/JJH **Checked By:** MTG MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.7	2.0	6.0	15.1	47.1	26.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	98.4		
3/8"	97.9		
1/4"	97.1		
#4	96.3		
#10	94.3		
#20	91.6		
#40	88.3		
#80	83.0		
#140	78.6		
#200	73.2		
0.0318 mm.	44.6		
0.0207 mm.	38.0		
0.0122 mm.	33.0		
0.0087 mm.	30.5		
0.0062 mm.	28.1		
0.0031 mm.	21.3		
0.0014 mm.	14.9		

* (no specification provided)

Soil Description

silt with sand

Atterberg Limits

PL= 24.9 LL= 33.1 PI= 8.2

Coefficients

D_{85}= 0.2453 D_{60}= 0.0502 D_{50}= 0.0382
D_{30}= 0.0081 D_{15}= 0.0014 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(5)

Remarks

Moisture Content: 34.3%

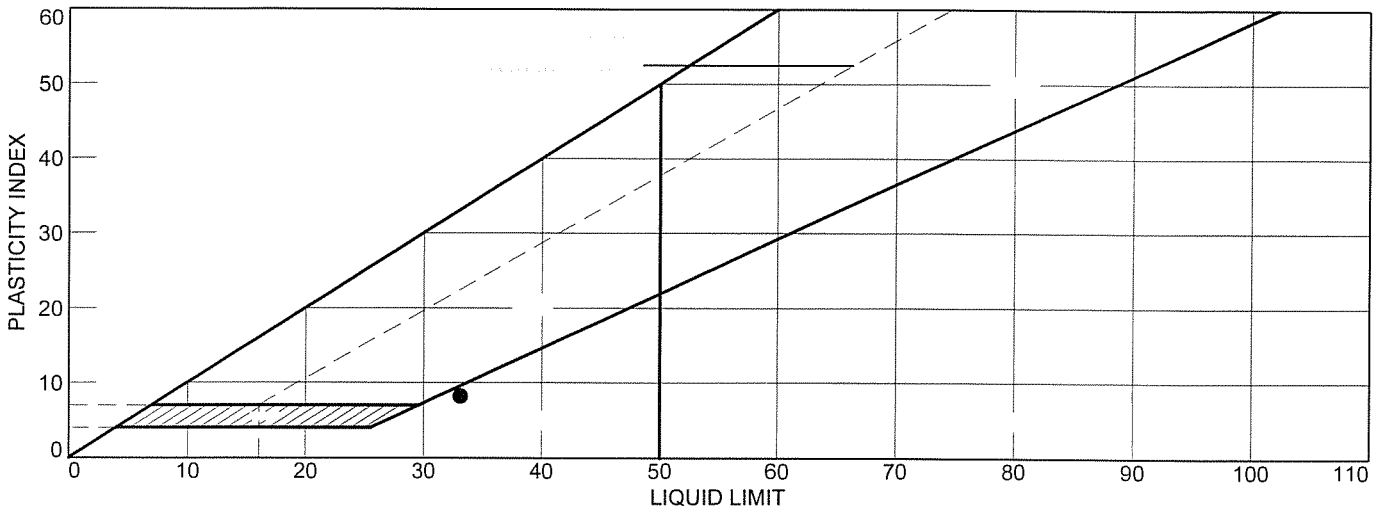
Sample No.: 3D **Source of Sample:** BB-IMC-101 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 10'-12'

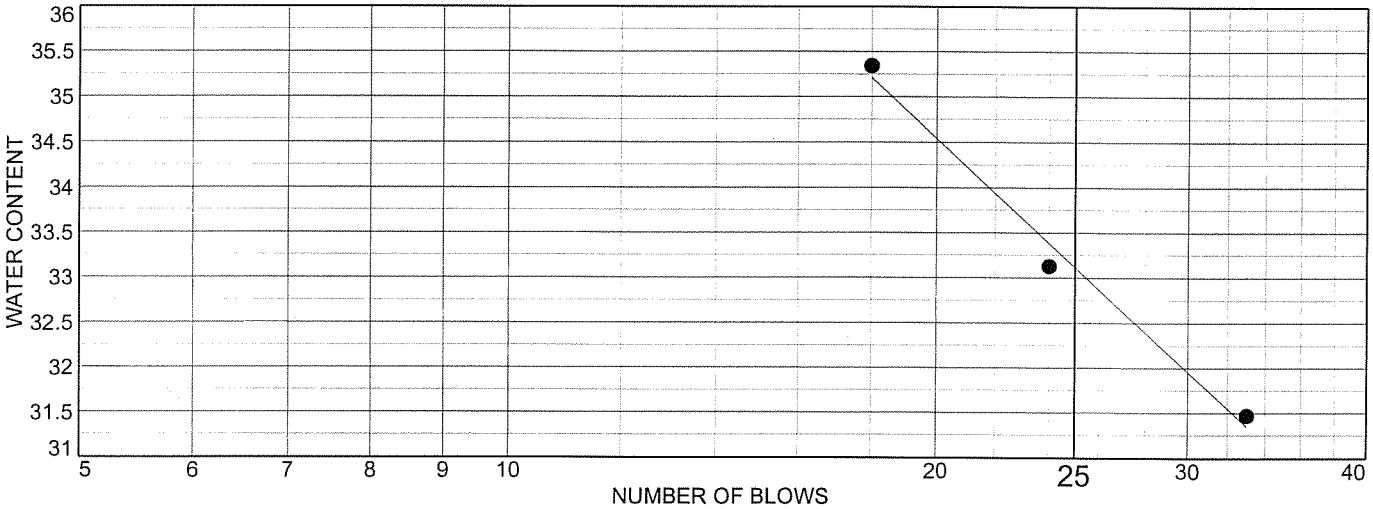
R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12233b

Tested By: DCH/JJH **Checked By:** MTG MTG

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	silt with sand	33.1	24.9	8.2	88.3	73.2	ML

Project No. 1368-001 **Client:** Schonewald Engineering Associates, Inc.

Project: MaineDOT Isleboro Mill Creek PIN 19285.00

Schonewald Job No. 11-015

● **Source of Sample:** BB-IMC-101 **Depth:** 10'-12' **Sample Number:** 3D

Remarks:

● Moisture Content: 34.3%

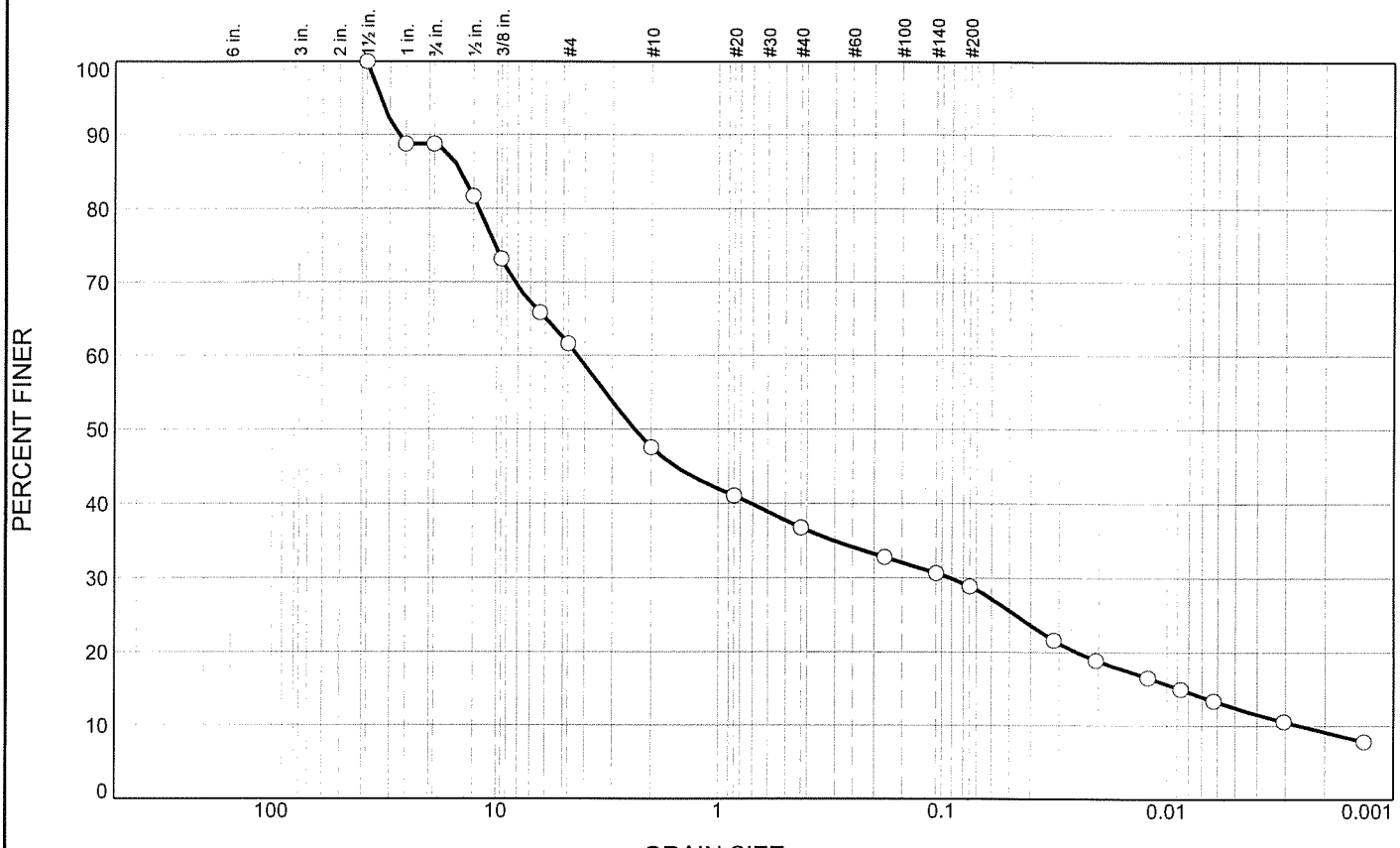
R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 12233b

Tested By: DCH **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.2	27.2	14.0	10.8	7.9	16.5	12.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	88.8		
3/4"	88.8		
1/2"	81.7		
3/8"	73.2		
1/4"	65.8		
#4	61.6		
#10	47.6		
#20	41.1		
#40	36.8		
#80	32.8		
#140	30.6		
#200	28.9		
0.0317 mm.	21.6		
0.0206 mm.	18.9		
0.0121 mm.	16.5		
0.0087 mm.	14.9		
0.0062 mm.	13.4		
0.0031 mm.	10.5		
0.0013 mm.	7.9		

* (no specification provided)

Soil Description

silty gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 14.4061 D_{60}= 4.2952 D_{50}= 2.3772
D_{30}= 0.0919 D_{15}= 0.0088 D_{10}= 0.0026
C_u= 1648.85 C_c= 0.76

Classification

USCS= GM AASHTO= A-2-4(0)

Remarks

Moisture Content: 17.6%

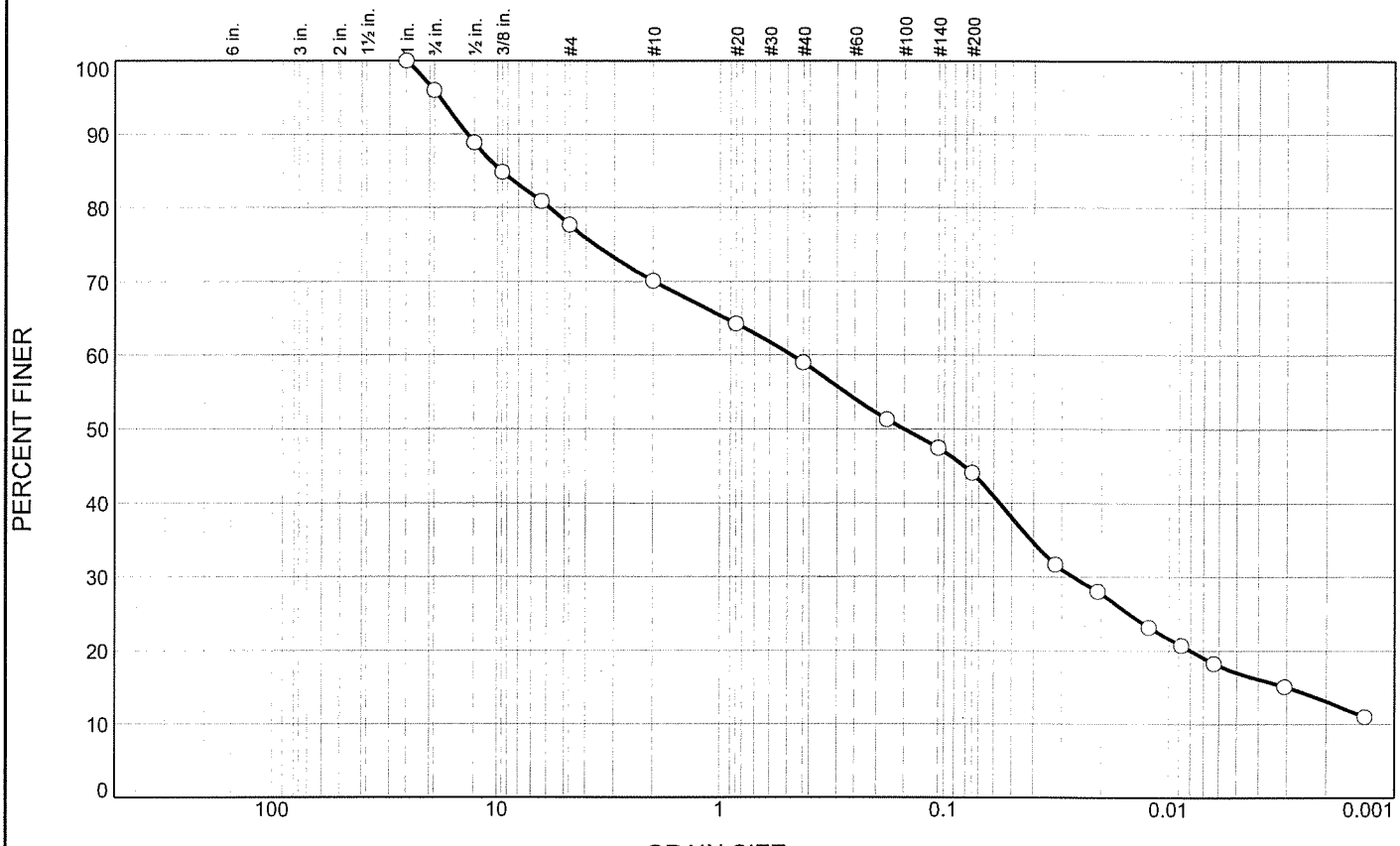
Sample No.: 3D **Source of Sample:** BB-IMC-102 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 10'-12'

R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12233c

Tested By: DCH/JJH **Checked By:** MTG MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.0	18.3	7.6	11.1	14.9	27.1	17.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	96.0		
1/2"	88.9		
3/8"	84.9		
1/4"	80.9		
#4	77.7		
#10	70.1		
#20	64.3		
#40	59.0		
#80	51.3		
#140	47.5		
#200	44.1		
0.0320 mm.	31.7		
0.0207 mm.	28.0		
0.0123 mm.	23.1		
0.0088 mm.	20.7		
0.0063 mm.	18.3		
0.0031 mm.	15.1		
0.0014 mm.	11.0		

* (no specification provided)

Soil Description

clayey sand with gravel

Atterberg Limits

PL= 20.3 LL= 27.6 PI= 7.3

Coefficients

D_{85}= 9.6415 D_{60}= 0.4800 D_{50}= 0.1509
D_{30}= 0.0267 D_{15}= 0.0030 D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-4(1)

Remarks

Moisture Content: 21.1%

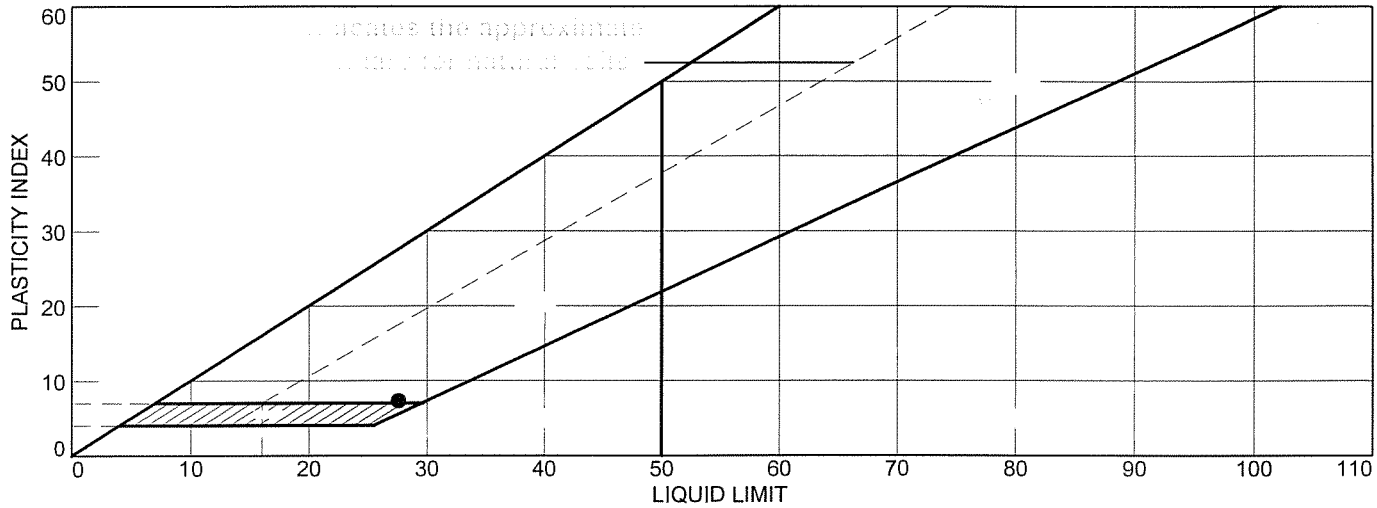
Sample No.: 3D **Source of Sample:** BB-IMC-103 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 10'-12'

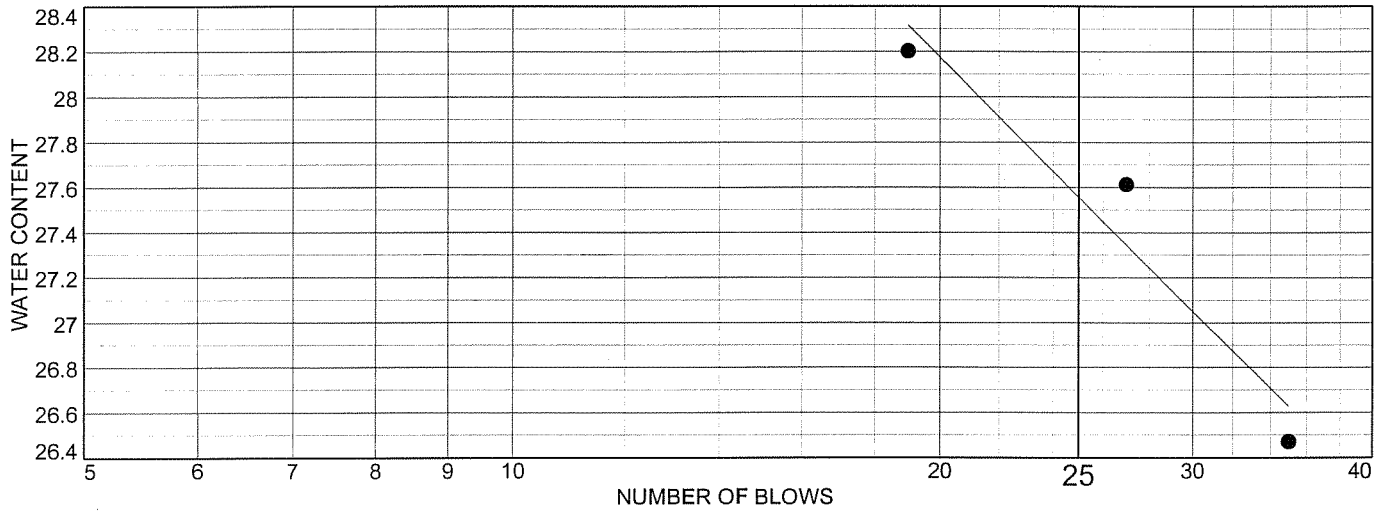
R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12234a

Tested By: DCH/JJH **Checked By:** MTG MTG

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	clayey sand with gravel	27.6	20.3	7.3	59.0	44.1	SC

Project No. 1368-001 **Client:** Schonewald Engineering Associates, Inc.

Project: MaineDOT Isleboro Mill Creek PIN 19285.00

Schonewald Job No. 11-015

● **Source of Sample:** BB-IMC-103 **Depth:** 10'-12' **Sample Number:** 3D

Remarks:

● Moisture Content: 21.1%

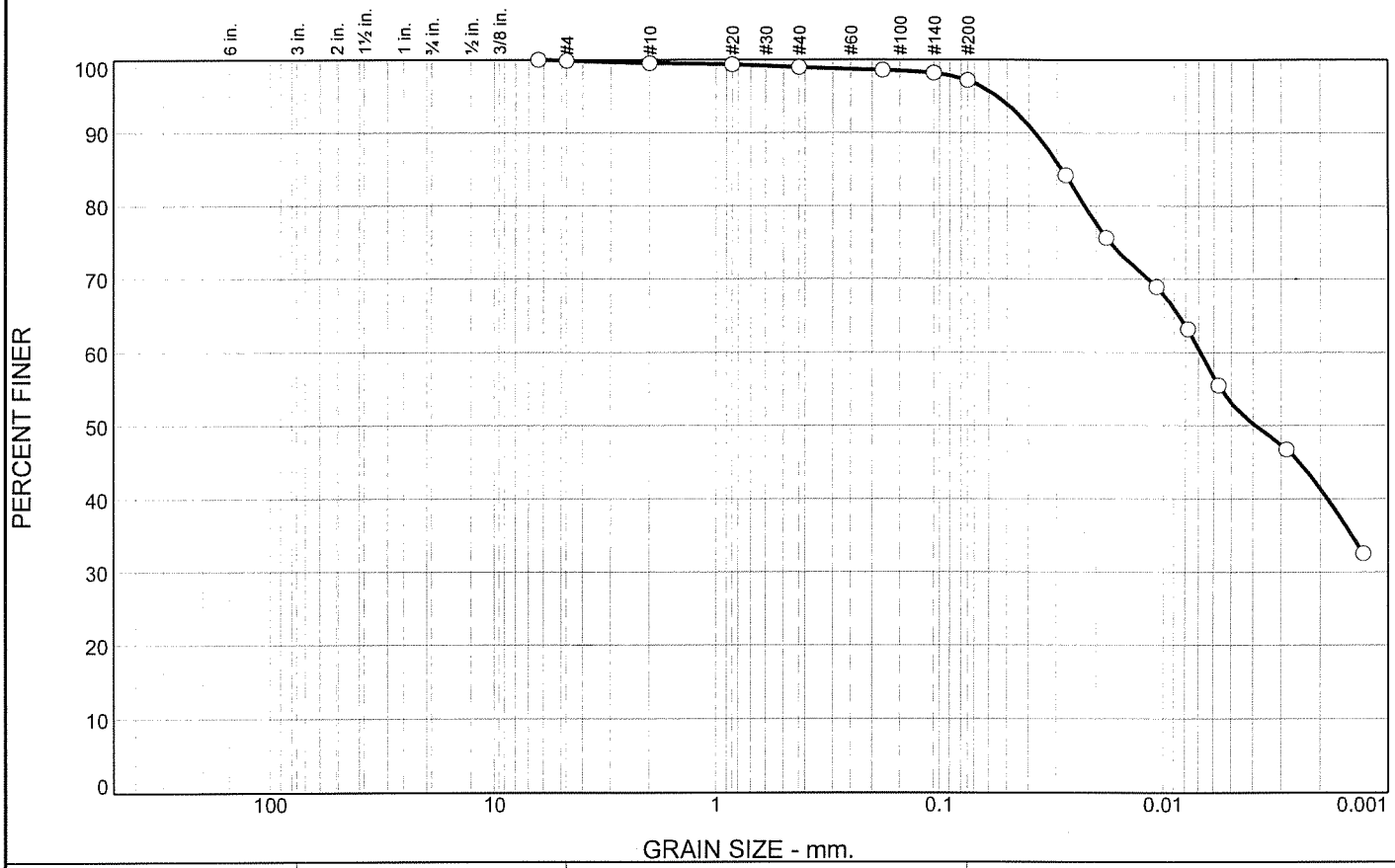
R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 12234a

Tested By: DCH **Checked By:** MTG MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.4	0.6	1.7	44.1	53.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/4"	100.0		
#4	99.9		
#10	99.5		
#20	99.3		
#40	98.9		
#80	98.6		
#140	98.2		
#200	97.2		
0.0273 mm.	84.2		
0.0181 mm.	75.6		
0.0108 mm.	68.9		
0.0078 mm.	63.2		
0.0057 mm.	55.5		
0.0028 mm.	46.8		
0.0013 mm.	32.5		

* (no specification provided)

Soil Description

lean clay

Atterberg Limits

PL= 18.5 LL= 32.1 PI= 13.6

Coefficients

D_{85}= 0.0284 D_{60}= 0.0069 D_{50}= 0.0039
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(13)

Remarks

Moisture Content: 31.0%

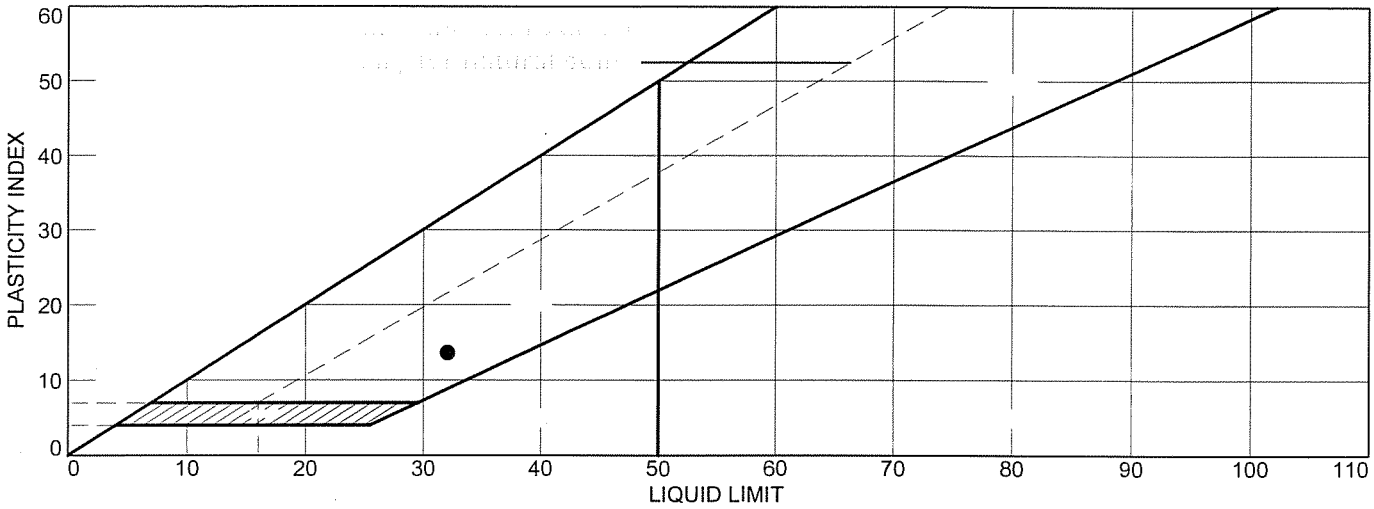
Sample No.: 5D **Source of Sample:** BB-IMC-103 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 20'-22'

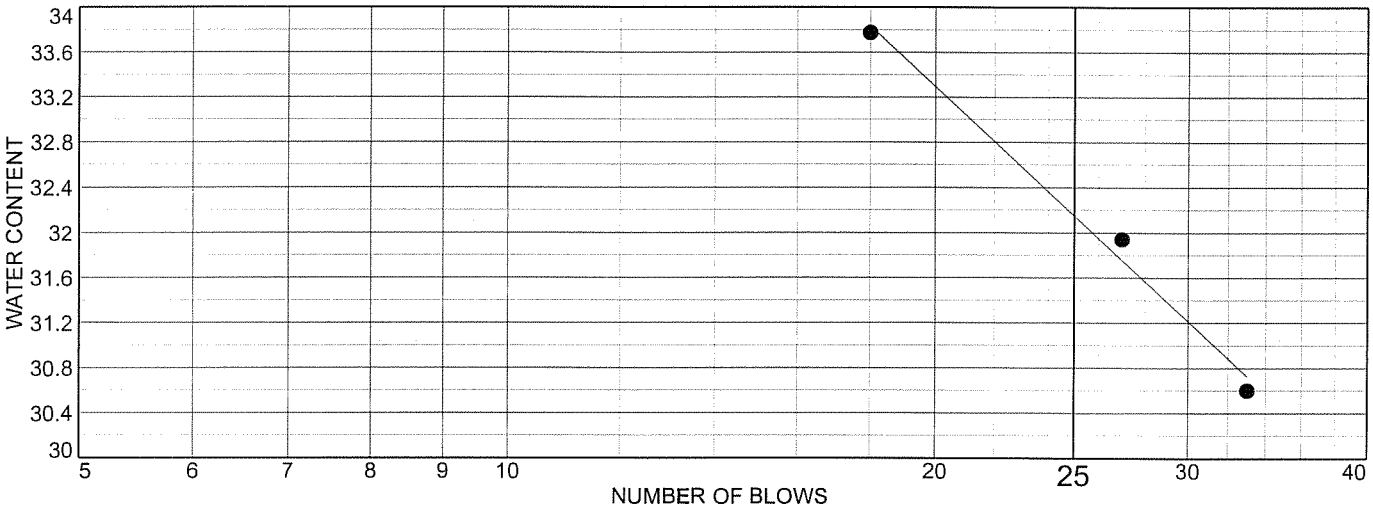
R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12234b

Tested By: DCH/JJH **Checked By:** MTG

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	lean clay	32.1	18.5	13.6	98.9	97.2	CL

Project No. 1368-001 **Client:** Schonewald Engineering Associates, Inc.

Project: MaineDOT Isleboro Mill Creek PIN 19285.00

Schonewald Job No. 11-015

● **Source of Sample:** BB-IMC-103 **Depth:** 20'-22' **Sample Number:** 5D

Remarks:

● Moisture Content: 31.0%

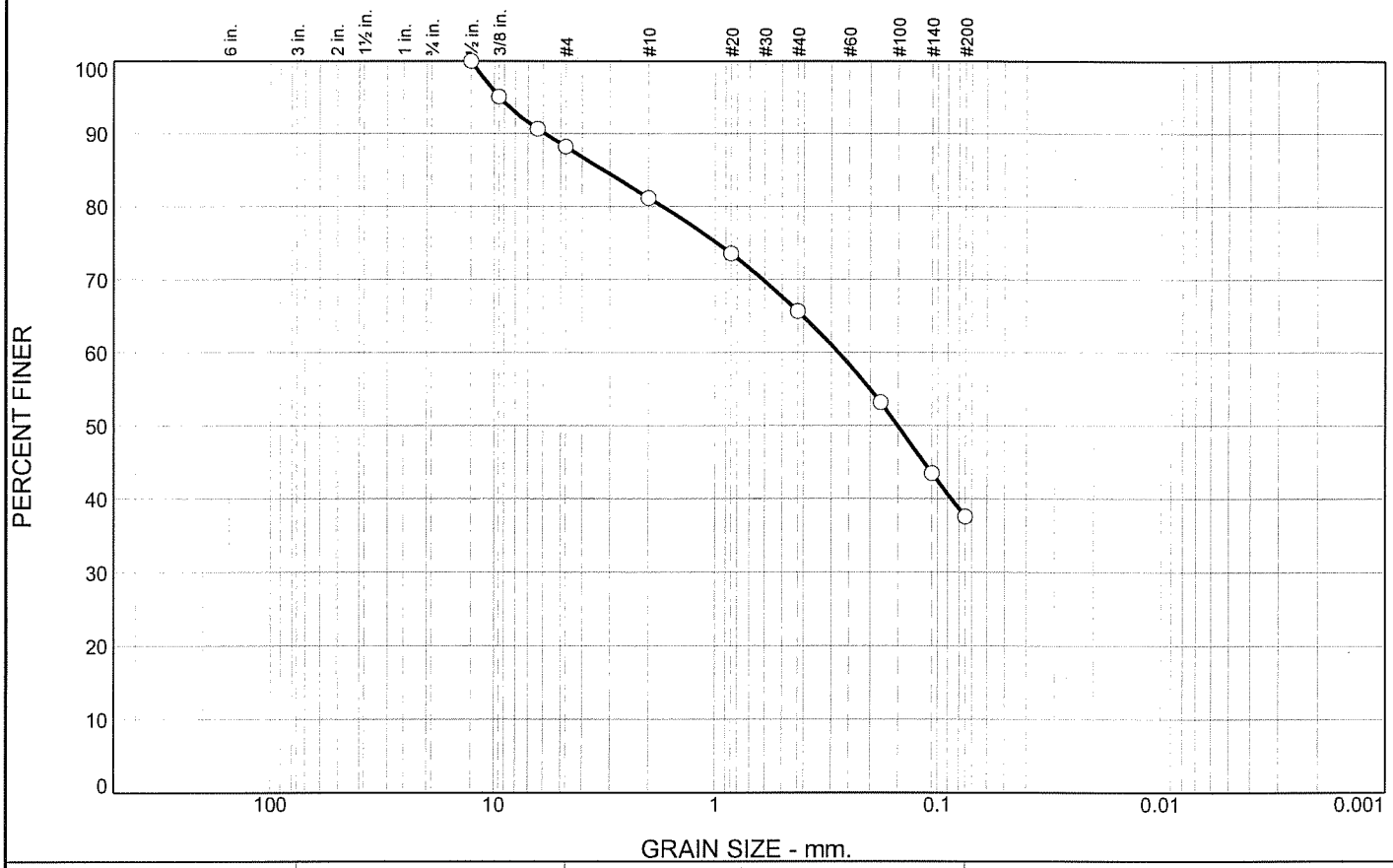
R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 12234b

Tested By: DCH **Checked By:** MTG MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.8	7.0	15.6	28.0	37.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	95.1		
1/4"	90.6		
#4	88.2		
#10	81.2		
#20	73.6		
#40	65.6		
#80	53.2		
#140	43.5		
#200	37.6		

* (no specification provided)

Soil Description

silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.2076 D_{60}= 0.2778 D_{50}= 0.1508
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 16.8%

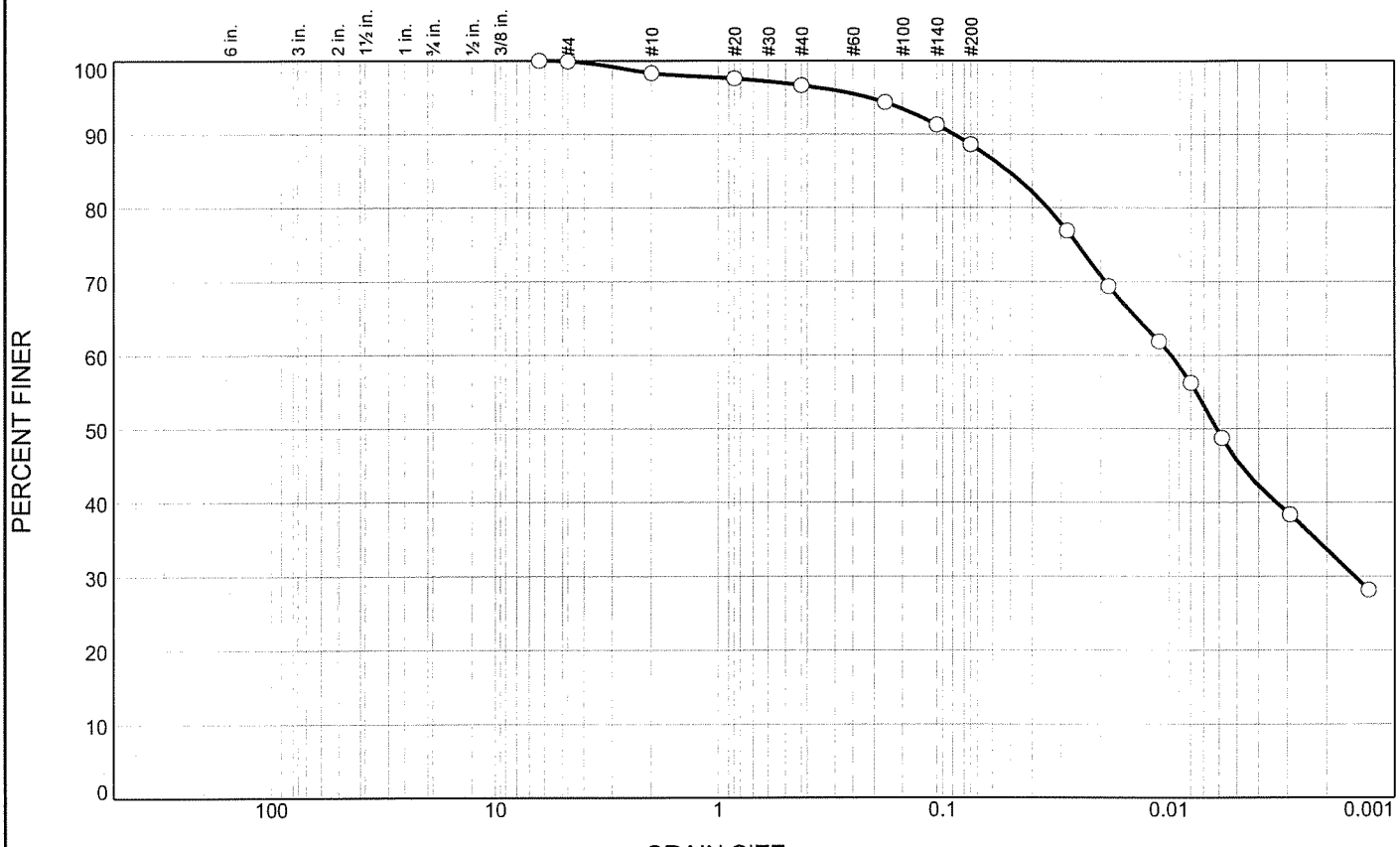
Sample No.: 2D **Source of Sample:** BB-IMC-104 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 5'-7'

R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12234c

Tested By: DCH/JJH **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	1.6	1.7	8.0	42.9	45.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/4"	100.0		
#4	99.9		
#10	98.3		
#20	97.6		
#40	96.6		
#80	94.3		
#140	91.3		
#200	88.6		
0.0282 mm.	76.9		
0.0185 mm.	69.4		
0.0111 mm.	61.9		
0.0080 mm.	56.3		
0.0058 mm.	48.8		
0.0029 mm.	38.3		
0.0013 mm.	28.1		

* (no specification provided)

Soil Description

lean clay

Atterberg Limits

PL= 18.1 LL= 29.8 PI= 11.7

Coefficients

D_{85}= 0.0511 D_{60}= 0.0098 D_{50}= 0.0062
D_{30}= 0.0015 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(10)

Remarks

Moisture Content: 30.4%

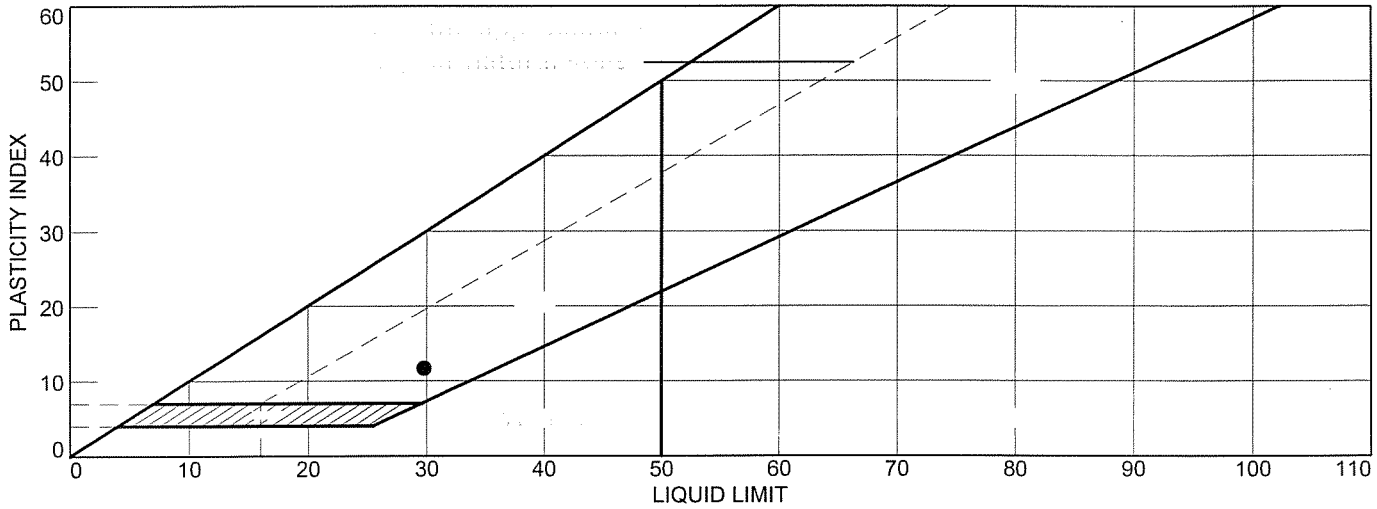
Sample No.: 5D **Source of Sample:** BB-IMC-104 **Date:** 12/23/2011
Location: Isleboro, ME **Elev./Depth:** 25'-27'

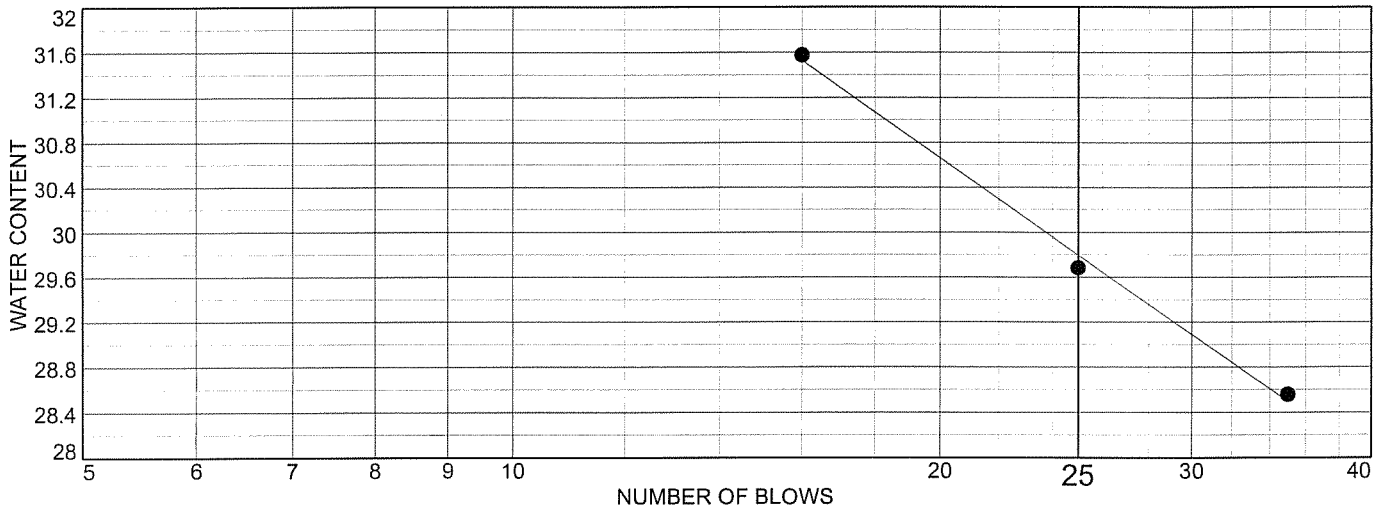
R.W. Gillespie & Associates, Inc.
Saco, Maine

Client: Schonewald Engineering Associates, Inc.
Project: MaineDOT Isleboro Mill Creek PIN 19285.00
Schonewald Job No. 11-015
Project No: 1368-001 **Lab No.** 12234d

Tested By: DCH/JJH **Checked By:** MTG MTG

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	lean clay	29.8	18.1	11.7	96.6	88.6	CL

Project No. 1368-001 **Client:** Schonewald Engineering Associates, Inc.

Project: MaineDOT Isleboro Mill Creek PIN 19285.00

Schonewald Job No. 11-015

● **Source of Sample:** BB-IMC-104 **Depth:** 25'-27' **Sample Number:** 5D

Remarks:

● Moisture Content: 30.4%

R.W. Gillespie & Associates, Inc.

Saco, Maine

Lab No. 12234d

Tested By: DCH **Checked By:** MTG MTG

LABORATORY TESTING DATA SHEET

Project Name MaineDOT Isleboro Mill Creek Bridge
Location Isleboro, ME
Reviewed By
Schonewald Engineering Assoc. No. 11-015
Assigned By B. Schonewald
MaineDOT PIN 19285.00
Report Date 11/29/2011
Date Reviewed 11/30/2011

Boring	Sample No.	Depth Ft.	Lab No.	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests (ASTM D7012 D) (3) Other Tests	(4) Strength KSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ KSI	Rock Formation or Description or Remarks
BB-IMC-101	R-1	19.8-20.2	1	1.988	4.559	172.2			U	6.77	0.21		2.42	0.14		Sample Failed on Foliation Plane
BB-IMC-104	R-1	31.1-31.5	2	1.984	3.976	174.3			U	7.76	0.47		1.23	0.11		

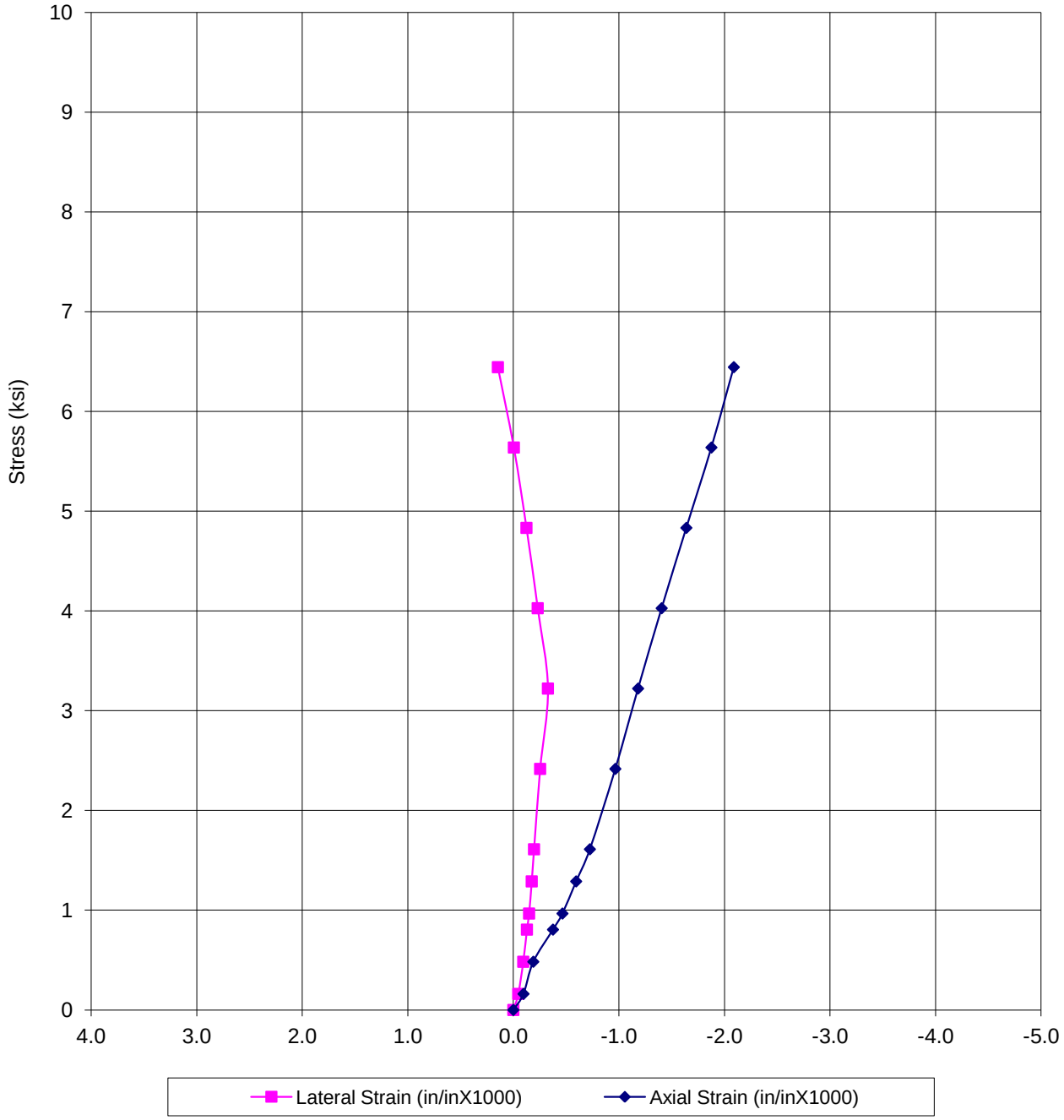
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical), PLA= Point Load (Axial) RST= Splitting Tensile U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

THIELSCH ENGINEERING
106 South Street
Hopkinton, MA 01748
508-435-9244

MaineDOT Isleboro Mill Creek Bridge
Isleboro, ME

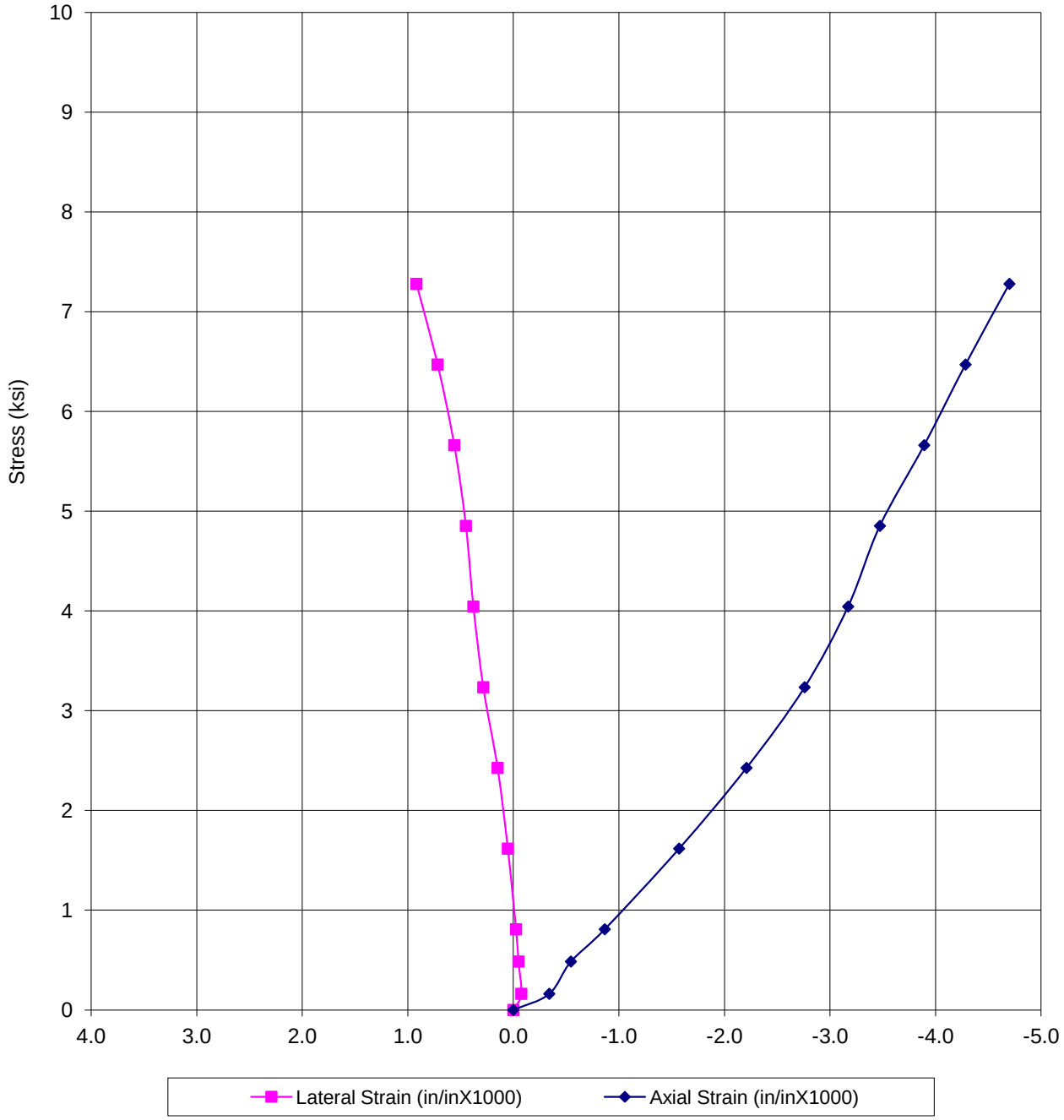


Rock Testing

Schonewald EA	11-015	MaineDOT PIN	19285.00
Boring No.	BB-IMC-101	File No.	CTS-74-11-0051.18
Sample No.	R-1	Date:	11/29/11
Depth:	19.8-20.2'	Test No.	U 1

THIELSCH ENGINEERING

MaineDOT Isleboro Mill Creek Bridge
Isleboro, ME



Rock Testing

Schonewald EA	11-015	MaineDOT PIN	19285.00
Boring No.	BB-IMC-104	File No.	CTS-74-11-0051.18
Sample No.	R-1	Date:	11/29/11
Depth:	31.1-31.5'	Test No.	U 2

THIELSCH ENGINEERING

Appendix C

Calculations

Bedrock Properties at the Site

RQD of bedrock cores
Abutment #1 Pile Group for span of 62 feet: BB-IMC-104, R1=37%, UCT Qp = 7,760 psi
R2=53%

Abutment #1 Pile Group for span of 50 feet: BB-IMC-103, R1= 65%

Abutment #2 Pile Group for span of 50 feet BB-IMC-102, R1=38%

Abutment #2 Pile Group for span of 62 feet: BB-IMC-101, R1=65%, UCT Qp = 6,770 psi
R2=48%
Rock Type: Sedimentary PHYLLITE and META-SANDSTONE

ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1);

uniaxial compressive strength = Co= 3500 to 35,000 psi - AASHTO TABLE 4.4.8.1.2.B (17th Edition, 2002)

Average of ALL upper bedrock cores: 51.3%

For Design Purposes: RQD = 51% and an Unconfined Compressive Strength of 6,770 psi

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 148.168 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

Nominal and Factored Structural Compressive Resistance of HP piles

Use LRFD Equation 6.9.2.1-1

$$F_y := 50 \cdot ksi$$

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QFyAs (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

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

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

Determine elastic critical buckling resistance Pe, LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 (assume rotation free at pile tip

l = unbraced length $l_{unbraced} := .5 \cdot ft$

r s = radius of gyration

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

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 251999 \\ 369452 \\ 518084 \\ 646435 \\ 881216 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 325.16 \\ 338.946 \\ 484.19 \\ 495.353 \\ 512.335 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then

$$P_n = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

this applies to all the pile sizes

use Po

If Pe/Po < 0.44, then:

$$P_{n1} := \overrightarrow{(0.877 \cdot P_e)}$$

not :

$$P_{n1} = \begin{pmatrix} 221003 \\ 324009 \\ 454359 \\ 566924 \\ 772827 \end{pmatrix} \cdot kip$$

Factored Axial Structural Resistance of an H-pile for the Strength Limit State

Resistance factor for H-pile in compression, severe driving condtions LRFD 6.5.4.2 for the case of tip damage

$$\phi_c := 0.5$$

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

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

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 387 \\ 544 \\ 535 \\ 652 \\ 859 \end{pmatrix} \cdot kip$$

Note : the structural resistance of the pile in combined axial and flexural resistance of the pile may be less; this is calculation is the responsibililty of the structural engineer.

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock - Nominal Axial Geotechnical Resistance

Article 10.7.3.2.3 states "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The NOMINAL bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for SEVERE driving conditoins. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore the approach used is:

1. Forgo a strictly static geotechnical pile resistance calculation (the Canadian Geotech. Society Method is for "end bearing in rock" which is not applicable if rock sockets are not installed),
2. Limit the NOMINAL axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.
3. Then compute the FACTORED axial geotechical pile resistance by applying a resistance factor of 0.65 considering a pile capacity determination method of dynamic testing with signal matching of at least 2 piles (ref: LRFD Table 10.5.5.2.3-1)

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5 \qquad P_r := \phi_c \cdot P_n$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 387 \\ 544 \\ 535 \\ 652 \\ 859 \end{pmatrix} \cdot kip$$

Resistance factor for wave equation analysis and dynamic test which will be required for construction.

$$\phi_{dyn} := 0.65$$

Factored Geotechnical Axial Pile Resistance (Rr) - **Strength Limit State**

$$R_{r_1} := \phi_{dyn} \cdot P_r \qquad R_{r_1} = \begin{pmatrix} 252 \\ 354 \\ 347 \\ 424 \\ 559 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Based on past practice, limit the factored geotechnical resistance to:

Based on recent LRFD driven pile projects, limit the factored geotechnical resistance to 75% of the nominal structural pile resistance with a resistance factor for severe driving condtions, of 0.50, applied. Then compute the nominal resistance by dividing by a resistance factor of 0.65 for a pile resistance determination method consisting of at least 2 dynamic load tests with signal matching

$$R_{geo} := P_r \cdot 0.75 \qquad R_{geo} = \begin{pmatrix} 290 \\ 408 \\ 401 \\ 489 \\ 644 \end{pmatrix} \cdot kip$$

Pile Load Test to the following "nominal" resitance

$$R_{nominal} := \frac{1}{\phi_{dyn}} \cdot R_{geo} \qquad R_{nominal} = \begin{pmatrix} 447 \\ 628 \\ 617 \\ 752 \\ 991 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing **IN** rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Phyllite $q_{u_1} := 6770 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3

$dd = 1$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p_1} := 3 \cdot q_{u_1} \cdot K_{sp} \cdot dd \qquad q_{p_1} = \begin{pmatrix} 661 \\ 660 \\ 650 \\ 649 \\ 649 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p_1} := \overrightarrow{\left(q_{p_1} \cdot A_s\right)} \qquad R_{p_1} = \begin{pmatrix} 71 \\ 100 \\ 97 \\ 118 \\ 155 \end{pmatrix} \cdot kip \qquad \phi = 1.0$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

$$\phi_{stat} := 0.45$$

LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r_p1} := \phi_{stat} \cdot R_{p_1} \qquad R_{r_p1} = \begin{pmatrix} 32 \\ 45 \\ 43 \\ 53 \\ 70 \end{pmatrix} \cdot kip$$

CGS method appropriate for pile bearing **IN** bedrock as LRFD Table 10.5.5.2.3-1 implies - do not use the values above

Drivability Analyses

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of fy

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ driving stress cannot exceed 45 ksi

Limit driving stress to 45 ksi or limit blow count to 6-10 bpi which is optimal for diesel hammers

Compute the resistance that can be achieved in a drivablity analysis:

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

$\phi_{dyn} := 0.65$ Reference LRFD Table 10.5.5.2.3-1

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
12 x 53 Delmag D19-42

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	21.59	0.00	1.1	5.39	14.86
200.0	29.93	0.01	2.5	6.57	13.55
300.0	37.19	0.50	3.9	7.11	13.25
350.0	41.56	0.86	4.7	7.38	13.28
370.0	43.12	0.80	5.1	7.47	13.40
390.0	44.45	0.87	5.5	7.53	13.42
400.0	45.23	1.00	5.7	7.60	13.52
450.0	49.15	1.52	6.6	8.04	14.39

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	15.50	in2

Limiting driving stress to 45 ksi:

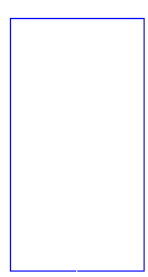
$$R_{ndr} := \left(\frac{45 - 44.45}{45.23 - 44.45}\right)\cdot(400\cdot kip - 390\cdot kip) + 390\cdot kip$$

$$R_{ndr} = 397.1\cdot kip$$

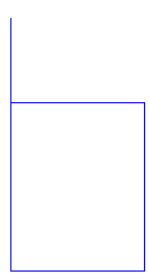
$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 258\cdot kip}$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 12 x 74

The 12x 74 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
12 x 74 Delmag D19-42

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	23.52	0.13	2.7	6.59	13.71
300.0	30.49	0.29	4.4	7.10	13.43
400.0	35.43	0.78	6.6	7.50	13.60
500.0	39.90	0.90	9.0	8.10	14.59
550.0	42.00	1.75	10.6	8.39	15.03
590.0	43.69	1.88	12.0	8.62	15.41
600.0	44.14	1.88	12.3	8.69	15.52
620.0	45.00	1.83	13.1	8.82	15.78

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	21.80	in2

Limit driving stress to 45 ksi

$R_{ndr} := 620 \cdot kip$

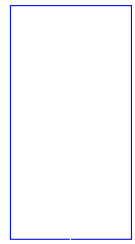
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

For a resistance factor for dynamic test of 0.65:

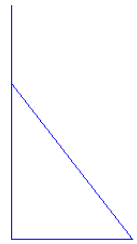
$R_{fdr} = 403 \cdot kip$

use this

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 73

14 x 73 with Delmag 19-42 and 2.7 kip helmet

State of Maine Dept. Of Transportation
14 x 73 Delmag D19-42

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	24.95	0.00	2.6	6.57	13.39
400.0	36.31	0.52	5.6	7.56	13.46
550.0	43.57	0.96	8.1	8.44	14.81
550.0	43.57	0.96	8.1	8.44	14.81
570.0	44.67	0.80	8.5	8.59	15.04
580.0	45.01	0.83	8.8	8.56	14.93
590.0	45.51	0.82	9.1	8.62	15.02
600.0	46.05	0.82	9.3	8.69	15.13
620.0	47.05	0.93	9.7	8.81	15.33

Limiting driving stress to 45 ksi

$R_{ndr} := 580 \cdot kip$

For a resistance factor for dynamic test of 0.65:

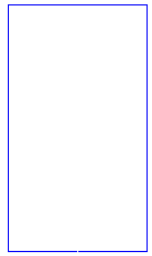
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$R_{fdr} = 377 \cdot kip$

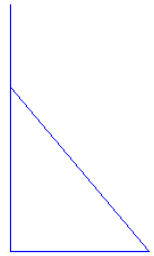
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 89 Delmag D19-42

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	27.21	0.11	4.2	7.05	12.98
400.0	31.40	0.46	5.7	7.36	12.92
570.0	38.04	0.88	8.3	8.28	14.20
580.0	38.26	0.87	8.6	8.26	14.09
600.0	38.99	0.88	9.0	8.37	14.24
640.0	40.35	1.05	9.9	8.58	14.52
660.0	41.14	1.13	10.2	8.75	14.84
680.0	41.62	1.25	10.9	8.76	14.74

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	26.10	in2

Limiting driving resistance to 10 bpi

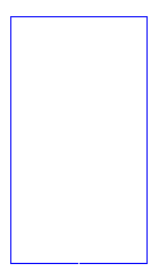
$R_{ndr} := 640 \cdot kip$

For a resistance factor for dynamic test of 0.65:

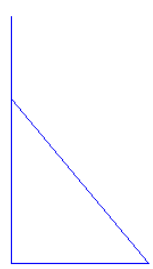
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$R_{fdr} = 416 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

The 14 x 89 pile can be driven to the resistances below with a D 36-32 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 89 Delmag D 36-32

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	33.06	0.00	1.7	6.60	30.95
470.0	44.49	0.10	3.0	7.42	29.23
480.0	45.10	0.09	3.0	7.47	29.18
500.0	46.26	0.10	3.2	7.56	29.25
520.0	47.37	0.15	3.3	7.64	29.17
530.0	48.28	0.21	3.4	7.68	29.32

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	26.10	in2

Limiting driving stress to 45 ksi:

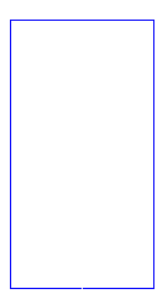
$R_{ndr} := 480 \cdot kip$

For a resistance factor for dynamic test of 0.65:

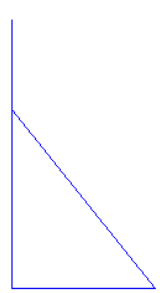
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$R_{fdr} = 312 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 3 and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 117 Delmag D36-32

01-Feb-2012
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	28.93	0.00	1.7	6.61	30.87
400.0	34.41	0.00	2.5	7.20	29.30
500.0	39.60	0.00	3.3	7.56	28.69
600.0	44.23	0.23	3.9	7.94	28.90
680.0	47.70	0.71	4.5	8.26	29.72
690.0	47.98	0.74	4.6	8.30	29.88
700.0	48.41	0.77	4.7	8.34	29.90

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	15.00	ft
Pile Penetration	10.00	ft
Pile Top Area	34.40	in2

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.23}{47.7 - 44.23}\right)\cdot(680\cdot kip - 600\cdot kip) + 600\cdot kip$$

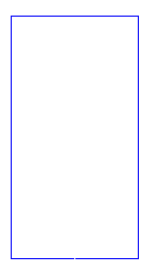
$$R_{ndr} = 617.8\cdot kip$$

For a resistance factor for dynamic test of 0.65:

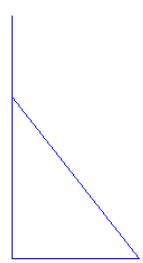
$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 402\cdot kip}$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Integral Abutments Passive Earth Pressure - Coulomb Theory

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

$\phi_1 := 32 \cdot deg$

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

For cases where interface friction is considered (this is for gravity shaped structures), use Coulomb.

For IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1 - because of the interface of the integral abutment backface and backfill soil

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta := 19.5 \cdot deg$

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

$K_{p_coul} = 6.73$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when B>0.

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot deg$

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

$K_{p_rank} = 3.255$

Pp is oriented at an angle of β to the vertical plane

Global Stability Analyses

Figure 1 – Cross Section at Station 18+15 (Abutment 1)
Figure 2 – Cross Section at Station 19+00 (Abutment 2)
Figure 3 – GeoSlope Results for slope stability at Station 18+15
Figure 4 - GeoSlope Results for slope stability at Station 19+00

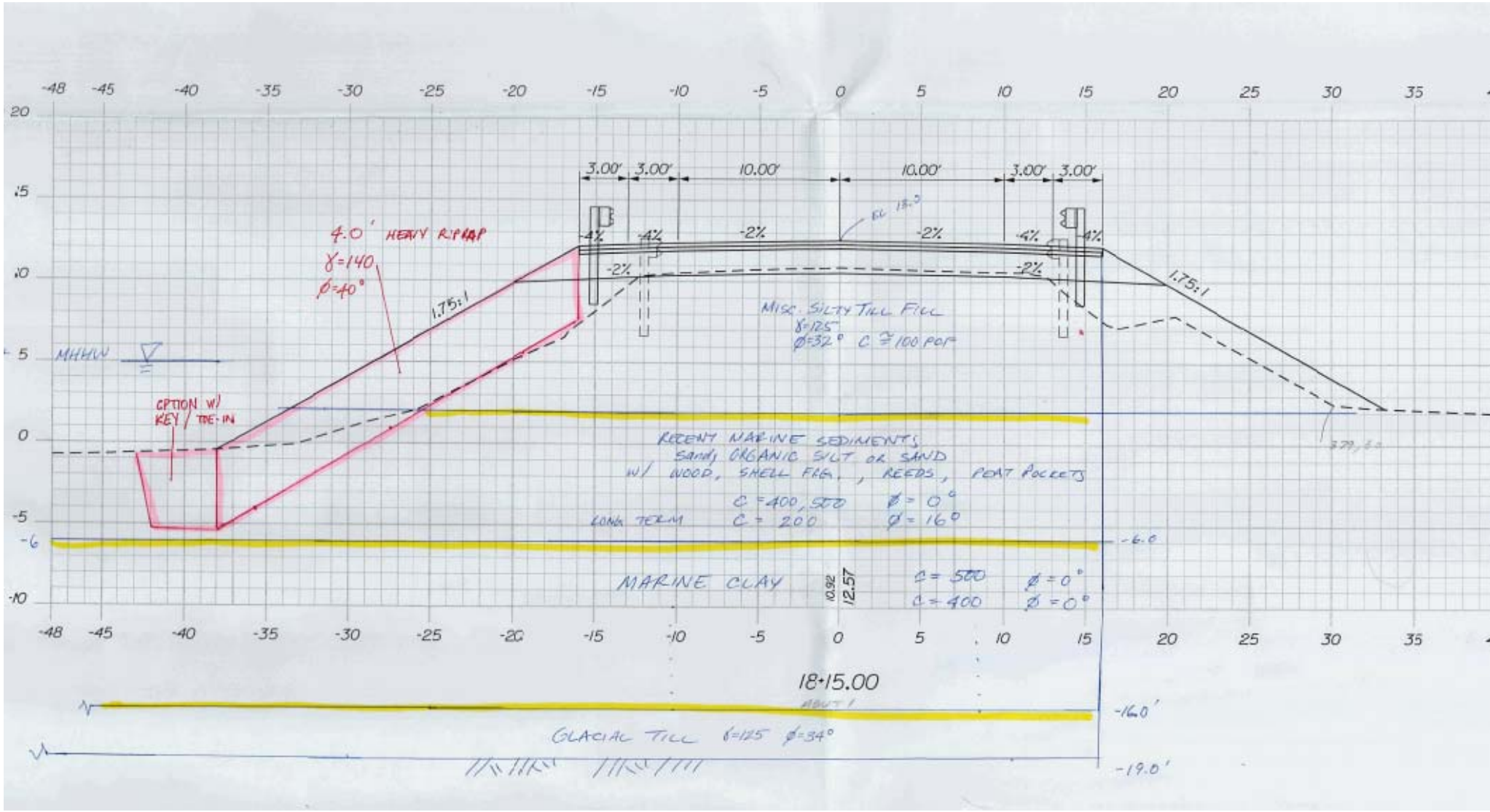

4.0' HEAVY RIPRAP
γ=140
φ=40°
1.75:1
3.00'
3.00'
10.00'
10.00'
3.00'
3.00'
-4%
-4%
-2%
-2%
-2%
-2%
-4%
-4%
MISC. SILTY TILL FILL
γ=125
φ=32° C ≅ 100 PSF
MHHW
OPTION W/ KEY / TRE-IN
RECENT MARINE SEDIMENTS
Sandy ORGANIC SILT OR SAND
W/ WOOD, SHELL FRG., REEDS, PEAT POCKETS
C=400, 500 φ=0°
LONG TERM C=200 φ=16°
-6.0
MARINE CLAY
10.92
12.57
C=500 φ=0°
C=400 φ=0°
18+15.00
-16.0'
GLACIAL TILL γ=125 φ=34°
-19.0'

FIGURE 1

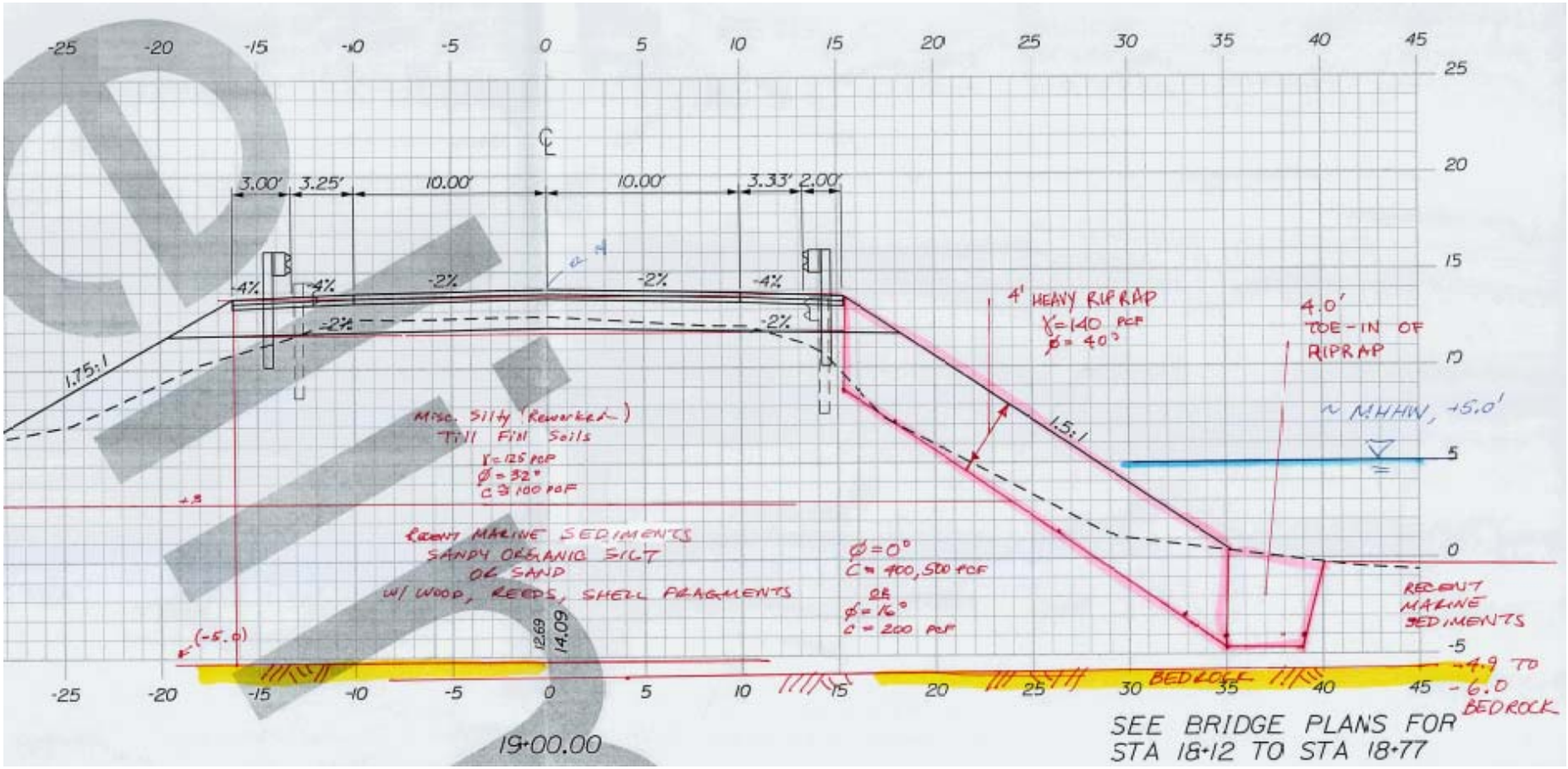

3.00'
3.25'
10.00'
10.00'
3.33'
2.00'
-4%
-4%
-2%
-2%
-4%
-2%
-2%
1.75:1
4' HEAVY RIPRAP
γ=140 PCF
φ=40°
4.0'
TOE-IN OF
RIPRAP
1.5:1
~ MHHW, +5.0'
Misc. Silty (Reworked)
Till Fill Soils
γ=125 PCF
φ=32°
C≅100 PCF
RECENT MARINE SEDIMENTS
SANDY ORGANIC SILT
OR SAND
W/ WOOD, REEDS, SHELL FRAGMENTS
φ=0°
C=400,500 PCF
OR
φ=16°
C=200 PCF
RECENT
MARINE
SEDIMENTS
12.69
14.09
(-5.0)
BEDROCK
-4.9 TO
-6.0
BEDROCK
19+00.00
SEE BRIDGE PLANS FOR
STA 18+12 TO STA 18+77

FIGURE 2

Title: Isleboro, soil profile at Abutment 1
Comments: 4 feet heavy rip rap over Recent Marine Deposits
Date: 2/3/2012
File Name: Islesboro Sta 18+00 rev0.gsz

Description: 4 feet heavy riprap STA 19+00 Right 1.5:1 slope

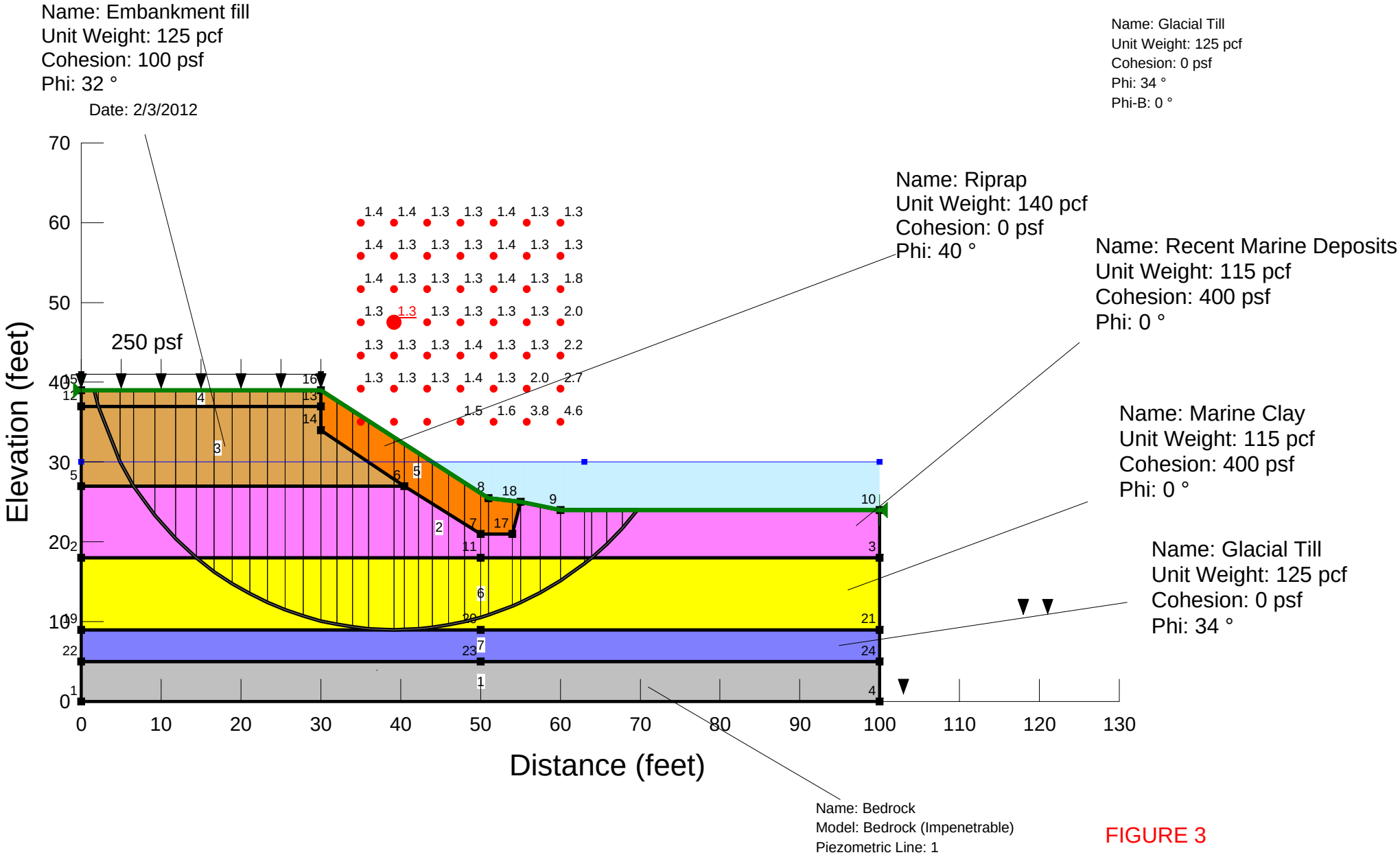


FIGURE 3

Title: Isleboro, soil profile at Abutment 2
Comments: 4 feet heavy rip rap over Recent Marine Deposits
Date: 2/3/2012
File Name: Islesboro Sta 19 R rev2.gsz

Description: 4 feet heavy riprap STA 19+00 Right 1.5:1 slope

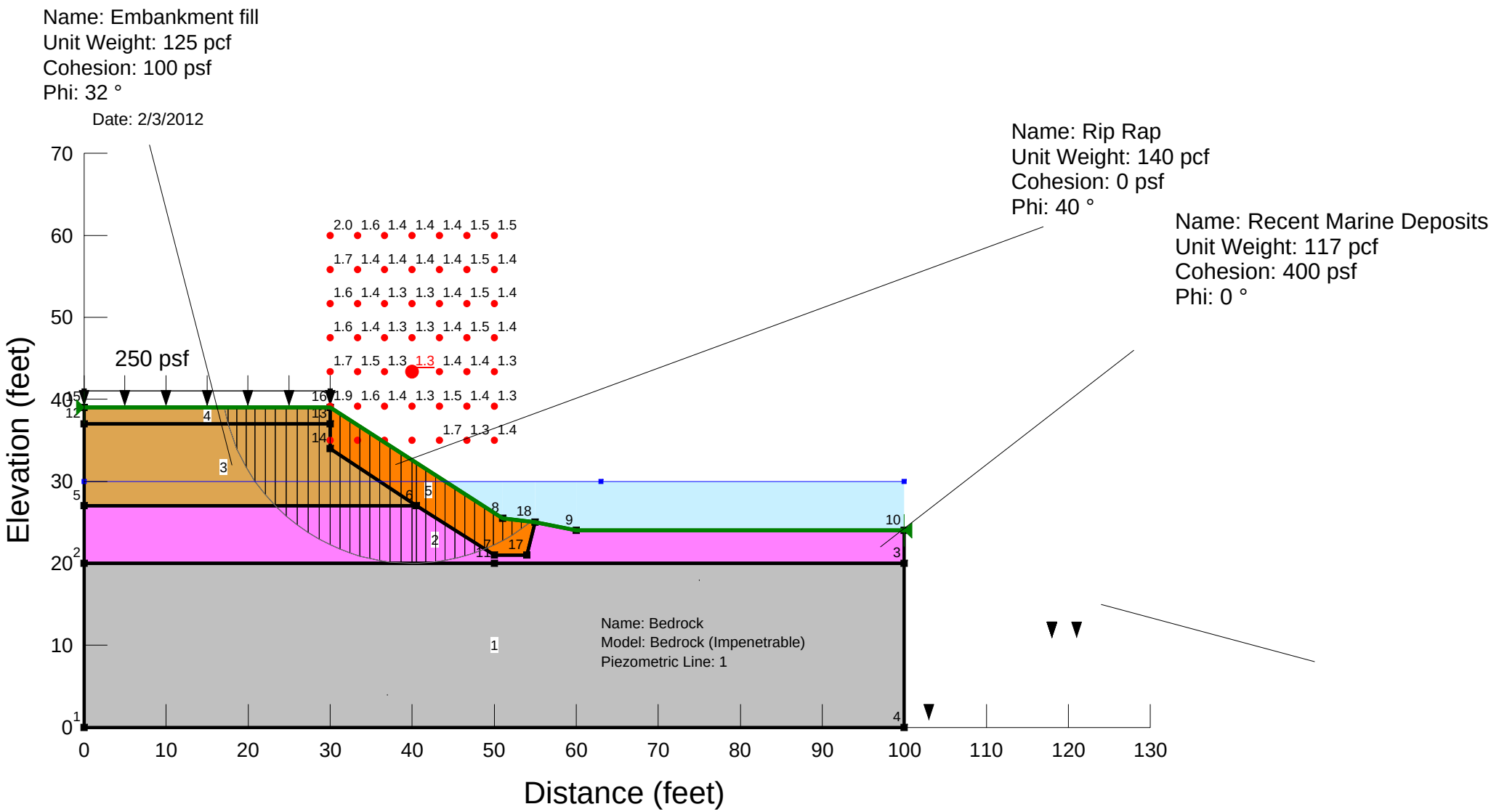


FIGURE 4

Settlement Evaluations

Islesboro Mill Bridge

PROJECT IDENTIFICATION

Title: Islesboro Mill Bridge
Project Number: 19285.00 -
Client:
Designer: Laura Krusinski
Station Number: Check by: KM 3/2012

Description:

Company's information:

Name: MaineDOT
Street: Bridge Program
,

Telephone #:
Fax #:
E-Mail:

Original file path and name: C:\Program Files\ADAMA\FoSSA(1.0)\Islesboro Abut 1 rev1.F2S
Original date and time of creating this file: Feb. 28 2012

GEOMETRY: Analysis of a 2D geometry

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Estimate Cc and Cr based on LL correlations

Correlations

Cc = 0.18 - 0.34	Bangor Area Clayey Silt, Andrews (1986)
Cc = -.5506 + 2.8801 x LL	Bangor Area Samples, Young (1966)
Cc = 0.009(LL-10%)	Terzaghi and Peck
Cr = 8 -10% Cc	

Recent Marine Sediments:

$$LL := \begin{pmatrix} 33 \\ 28 \end{pmatrix} \qquad Cc := .009 \cdot (LL - 10) \qquad Cc = \begin{pmatrix} 0.207 \\ 0.162 \end{pmatrix}$$

Use Terzaghi Correlation

$$Cr := Cc \cdot 0.10 \qquad Cr = \begin{pmatrix} 0.021 \\ 0.016 \end{pmatrix}$$

Marine Clay:

$$LL := \begin{pmatrix} 30 \\ 32 \end{pmatrix} \qquad Cc := .009 \cdot (LL - 10) \qquad Cc = \begin{pmatrix} 0.18 \\ 0.198 \end{pmatrix}$$

Use Terzaghi Correlation

$$Cr := Cc \cdot 0.10 \qquad Cr = \begin{pmatrix} 0.018 \\ 0.02 \end{pmatrix}$$

Input Parameters for Settlement Analyses:

Recent Marine Sediments, OCR = 1, Cc = 0.20 Cr =0.02

Marine Silt Clay, Cc= 0.2, Cr =0.02

INPUT DATA – FOUNDATION LAYERS – 4 layers

	Wet Unit Weight, γ [lb/ft³]	Poisson's Ratio μ	Description of Soil
1	125.00	0.30	Existing Embankment Fills
2	115.00	0.35	Recent Marine Deposits, organic silts, silty sands
3	115.00	0.45	Marine Clay
4	130.00	0.10	Bedrock or Till

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INPUT DATA – EMBANKMENT LAYERS – 1 layers

	Wet Unit Weight, γ [lb/ft³]	Description of Soil
1	125.00	

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INPUT DATA OF WATER

Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]
1	290.00	330.00
2	311.68	330.00
3	330.00	330.00
4	340.00	330.00
5	360.00	330.00

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INPUT DATA FOR CONSOLIDATION -- $\alpha = 1/2$

Layer #	Underging Consolidation [Yes/No]	OCR = Pc / Po	Cc	Cr	e0	Cv [ft ²/day]	Drains at :
1	No	N/A	N/A	N/A	N/A	N/A	N/A
2	Yes	1.00	0.20	0.02	1.00	0.2153	Top
3	Yes	1.00	0.20	0.02	1.00	0.2153	Bottom
4	No	N/A	N/A	N/A	N/A	N/A	N/A

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IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section: X [ft.]	Y [ft.]	Layer #	Young's Modulus, E [lb/ft ²]	Poisson's Ratio, μ	Si(k) [ft.]	Z initial [ft.]	Z final [ft.]
1	297.00	0.00	1	522117	0.3000	0.0000	327.00	327.00
			2	208800	0.3500	-0.0005		
			3	313270	0.4500	-0.0001		
			4	104423	0.1000	0.0005		
2	306.44	0.00	1	522117	0.3000	0.0000	329.18	329.16
			2	208800	0.3500	0.0087		
			3	313270	0.4500	0.0061		
			4	104423	0.1000	0.0010		
3	315.89	0.00	1	522117	0.3000	0.0030	333.62	333.60
			2	208800	0.3500	0.0136		
			3	313270	0.4500	0.0090		
			4	104423	0.1000	0.0013		
4	325.33	0.00	1	522117	0.3000	0.0034	338.00	337.98
			2	208800	0.3500	0.0065		
			3	313270	0.4500	0.0058		
			4	104423	0.1000	0.0012		
5	334.78	0.00	1	522117	0.3000	0.0032	338.00	337.99
			2	208800	0.3500	0.0062		
			3	313270	0.4500	0.0043		
			4	104423	0.1000	0.0011		
6	344.22	0.00	1	522117	0.3000	0.0033	338.00	337.99
			2	208800	0.3500	0.0063		
			3	313270	0.4500	0.0042		
			4	104423	0.1000	0.0010		
7	353.67	0.00	1	522117	0.3000	0.0033	338.00	337.98
			2	208800	0.3500	0.0064		
			3	313270	0.4500	0.0046		
			4	104423	0.1000	0.0011		
8	363.11	0.00	1	522117	0.3000	0.0039	337.48	337.46
			2	208800	0.3500	0.0086		
			3	313270	0.4500	0.0062		
			4	104423	0.1000	0.0011		
9	372.56	0.00	1	522117	0.3000	0.0016	333.03	333.02
			2	208800	0.3500	0.0086		
			3	313270	0.4500	0.0053		
			4	104423	0.1000	0.0009		
10	382.00	0.00	1	522117	0.3000	0.0000	330.00	330.00
			2	208800	0.3500	-0.0002		
			3	313270	0.4500	0.0004		
			4	104423	0.1000	0.0005		

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ULTIMATE SETTLEMENT, Sc

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z * [ft.]
1	297.00	0.00	327.00	0.23	326.77
2	306.44	0.00	329.18	0.65	328.53
3	315.89	0.00	333.62	0.40	333.22
4	325.33	0.00	338.00	0.20	337.80
5	334.78	0.00	338.00	0.19	337.81
6	344.22	0.00	338.00	0.18	337.82
7	353.67	0.00	338.00	0.18	337.82
8	363.11	0.00	337.48	0.21	337.26
9	372.56	0.00	333.03	0.32	332.72
10	382.00	0.00	330.00	0.21	329.79

*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

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TABULTAED GEOMETRY INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	297.00	327.00	Existing Embankment Fills
	2	310.00	330.00	
	3	323.00	338.00	
	4	362.00	338.00	
	5	379.00	330.00	
2	1	287.00	327.00	Recent Marine Deposits, organic silts, silty sands
	2	297.00	327.00	
	3	310.00	330.00	
	4	370.00	330.00	
	5	400.00	330.00	
3	1	287.00	322.00	Marine Clay
	2	379.00	322.00	
	3	400.00	322.00	
4	1	287.00	312.00	Bedrock or Till
	2	400.00	312.00	

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TABULTAED GEOMETRY INPUT OF EMBANKMENT SOILS

Embank. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	366.00	341.00	
	2	3712.46	341.00	

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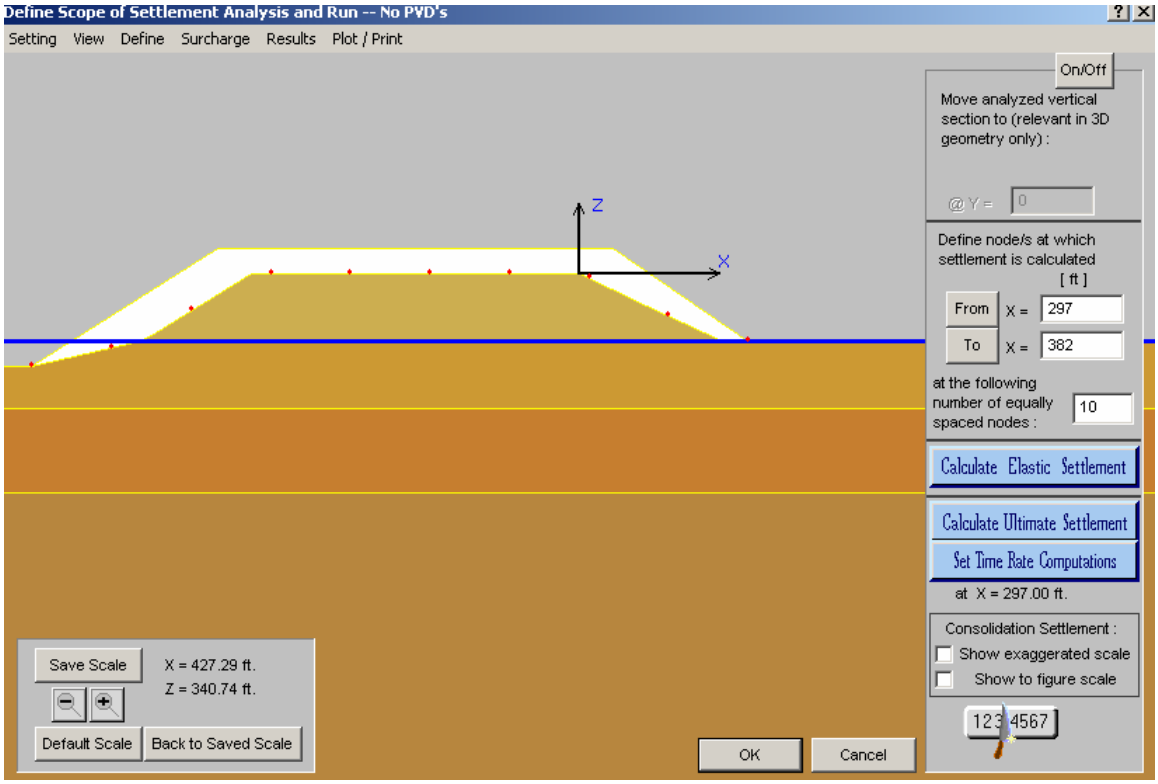
Settlement evaluated at 10 nodes (red)



Maximum Elastic Settlement occurs at Node 3

Immediate Settlement Si -- Node # 3

Settlement is calculated along section (Z-axis) for X = 315.89 and Y = 0.00 ft.

$$\varepsilon_z = \frac{1}{E}\left[\sigma_z - \mu\left(\sigma_x + \sigma_y\right)\right]$$

$$(S_i)_i = \int_{Z_0}^{Z_f} \varepsilon_z \, dz = \sum_{i=0}^{m} (\varepsilon_z \cdot \Delta z)_i \quad \text{=====>} \quad (S_i)_k = \sum_{j=1}^{n} (S_i)_j$$

Specified the Zf-ordinate at which settlement integration is terminated, Hi = 312.00

Total settlement at ground surface (Si)k : 0.03 ft.

Ground elevation:: Inial = 333.62 ft.
Final = 333.60 ft.

Layer #	Young's Modulus, E [lb/ft²]	Poisson's Ratio μ	Approximate elastic (immediate) settlement of each layer, (Si) k : [ft]
1	522117	0.3000	0.0030
2	208800	0.3500	0.0136
3	313270	0.4500	0.0090
4	104423	0.1000	0.0013

Back < Display results for different nodes > Next

RETURN

Combined Elastic and Consolidation Settlement

Results of Ultimate Consolidation Settlements -- 1D

#	X [ft]	Y [ft]	Original Z [ft]	Settlement Sc [ft]	Final Z [ft]
1	297.000	0.000	327.0000	0.2320	326.7680
2	306.444	0.000	329.1795	0.6478	328.5316
3	315.889	0.000	333.6239	0.4044	333.2195
4	325.333	0.000	338.0000	0.2012	337.7988
5	334.778	0.000	338.0000	0.1857	337.8143
6	344.222	0.000	338.0000	0.1833	337.8167
7	353.667	0.000	338.0000	0.1846	337.8154
8	363.111	0.000	337.4771	0.2144	337.2627
9	372.556	0.000	333.0327	0.3164	332.7163
10	382.000	0.000	330.0000	0.2114	329.7886

Ultimate Settlement Equation

RETURN

X (ft)	Y (ft)	Settlement (inches)
279.0	0.00	2.78
306.44	0.00	8.88
315.89	0.00	4.85
325.33	0.00	2.41
334.78	0.00	2.23
344.22	0.00	2.20
353.67	0.00	2.22
363.11	0.00	2.57
372.56	0.00	3.80
382.00	0.00	2.57

Frost Depth Calculation

Seismic Parameters

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

From Design Freezing Index Map:
Islesboro, Maine

DFI = 1300 degree-days

Case I - Medium to coarse grained fill soils -WC=20%.

Depth of Frost Penetration = 63 inch

$d := 63 \cdot in$ $d = 63 \cdot in$ $d = 5.25 \cdot ft$

Method 2 - ModBerg Software

Examine coarse grained soils without 4 inches of asphalt

--- 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	70.3	20.0	125.0	34	46	3.8	1.9	3,600

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

Total Depth of Frost Penetration = 5.86 ft = 70.3 in.

Recommendation: 5.5 feet for design of spread footings constructed on soil

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04848
Zip Code Latitude = 44.306800
Zip Code Longitude = -068.903200
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.058	PGA - Site Class B
0.2	0.129	Ss - Site Class B
1.0	0.040	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04848
Zip Code Latitude = 44.306800
Zip Code Longitude = -068.903200
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.094	As - Site Class D
0.2	0.207	SDs - Site Class D
1.0	0.097	SD1 - Site Class D

BB-101					BB-102					BB-103					BB-104				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
3	9		3.2	0.355556	3	14		4	0.285714	3	9		3.1	0.344444	3	6		9	1.5
6	5		5.8	1.16	6	18		3	0.166667	6	4		5.4	1.35	6	2		6	3
11	2		6.8	3.4	11	5		8	1.6	11	15		6.5	0.433333	11	19		6.5	0.342105
					16	17		2	0.117647	16	7		5	0.714286	16	3		2	0.666667
										21	2		5.9	2.95	21	1		8.4	8.4
															26	2		2	1
															26.5	20		3.6	0.18
15.8	100	**bedrock**	84.2	0.842	17	100	**bedrock**	83	0.83										
										25.9	100	**Bedrock**	74.1	0.741	37.5	100	**bedrock**	62.5	0.625
SUM			**100**	**5.757556**				**100**	**3.000028**				**100**	**6.533063**				**100**	**15.71377**

di/di/N **17.36848** **di/di/N** **33.33302** **di/di/N** **15.30675** **di/di/N** **6.363844**

SUM	Nav.	18.09303

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

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock Injector Pile Tip)

Subsection 501.10 Prefabricated Pile Tips of the Standard Specifications is amended as follows:

Pile tips for use on all piles shall be Rock Injector HP-80500 Pile Point, manufactured by Associated Pile and Fitting or approved equal. Material specifications, attachment of pile tips and seating of the piles shall be in accordance with Manufacturer's recommendations and in accordance with the Standard Specifications.

Payment will be made under:

Pay Item	Description	Pay Unit
501.903	Pile Tips – Rock Injector Point	Each