MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

NORTON BRIDGE
FULLER ROAD OVER BLACK STREAM
CARMEL, 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 Norton Bridge which carries Fuller Road over Black Stream, in Carmel, Maine. The proposed replacement bridge will be a 60-foot single span, integral bridge. The proposed replacement structure will have a centerline approximately matching the existing bridge centerline. The roadway profile will be raised approximately 1 foot.

Preliminary geotechnical evaluations identified two foundations alternatives: pile-supported integral abutments, and full-height, cantilever type abutments founded on spread footings constructed directly on bedrock or seal concrete founded on bedrock. Subsequent evaluations by the designer have identified the more effective foundation type to be integral abutments founded on approximately 15-foot long piles. The following design recommendations for pile supported integral abutments are discussed in detail in this report.

Integral Pile Design - The piles should be end bearing and driven to the required resistance on, or within, bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design pile loads. Piles should be 50 ksi, Grade A572 steel. Driven piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

The structural designer shall design H-piles for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral and flexural resistance. Our analysis indicates the factored axial drivability pile resistances control.

The maximum factored axial pile load should not exceed the calculated factored drivability pile resistances provided in this report. Integral pile design is discussed in Section 7.2 of this report.

The top of the piles should be checked for resistance against combined axial load and flexure, per Article 6.15.2 of the AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, with 2008 Interims (herein referred to as LRFD). As the proposed integral H-piles will be short and not achieve fixity, the resistance of the piles should be analyzed for combined axial compression and flexure resistance and evaluated for structural compliance with the interaction equation.

For strength limit state load combinations, a resistance factor of 0.70 for axial resistance (ϕ_c) and 1.0 for flexural resistance (ϕ_f) should be applied to the combined axial and flexural resistance of the pile in the interaction equation.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Driven Pile Quality Control - The contractor is required to perform a wave equation analysis of the proposed pile-hammer system. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. 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. LRFD Article 10.5.5.2.3 specifies that the resistance factor, ϕ_{dyn}, of 0.65 be reduced by 20 percent when there are less than five (5) piles in the group, in which case a resistance factor, ϕ_{dyn}, of 0.52 should be used. The maximum factored pile load and the appropriate resistance factor should be shown on the plans.

Integral Stub Abutment Design - Integral abutment sections shall be designed for all relevant strength, service and extreme limit states specified in LRFD Articles 3.4.1 and 11.5.5. Integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure calculated using a passive pressure coefficient, K_p, of 3.3, calculated using Rankine Theory. Wingwall sections that are integral with the abutment, should also be designed to withstand a maximum earth pressure equal to the Rankine passive earth pressure state. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT Bridge Design Guide (BDG) for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of the surcharge loads is permitted per LRFD 3.11.6.5.

Scour and Riprap - For scour protection, bridge approach slopes and slopes at abutments and wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1-foot thick layer of bedding material.

If short pile-supported abutments are used, the stream velocity should be low and there should be low potential for scour action, wave action, storm surge and ice damage. This is to maintain the integrity of the bridge approach fills and riprap abutment slopes, which provide the lateral support to the approach embankments and pile groups.

Settlement - The grades of bridge approaches will be raised approximately 1 foot. Post-construction settlement due to consolidation of the glaciomarine foundation soils is calculated to be approximately 1.5 inches near the mid to lower fill extension areas. Any settlement of the bridge abutments will be due to elastic settlement of the bedrock or piles, which is assumed to occur during construction and will be negligible.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Frost Protection - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection. Any retaining wall foundations placed on granular fill soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations – In conformance with LRFD 4.7.4.2., seismic analysis is not required for single-span bridges, regardless of seismic zone, however superstructure connections and bridge seat dimensions shall be satisfied per the seismic requirements in LRFD 3.10.9 and 4.7.4.4., respectively.

Construction Considerations – Construction of the abutments will require soil excavation. Construction activities may require internally braced cofferdams and earth support systems. The silt clay soils at the integral abutment subgrade will be susceptible to rutting as a result of exposure to water or construction traffic.

The subgrade native silt-clay soils within the project area are both poorly drained and moderately to highly frost susceptible. In some locations, these soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The contractor should control groundwater, surface water infiltration, and soil erosion.

The existing abutments, wingwalls and timber piles may obstruct installation of piles. Removal of all or some of the existing substructures may be necessary. The pile foundation area may require placement and compaction of granular fill up to the abutment subgrade level.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Norton Bridge which carries Fuller Road over the Black Stream, in Carmel, Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for replacement bridge foundations.

Norton Bridge was built in approximately 1925 and is a 36-foot two-span, concrete slab superstructure, supported on full height, concrete gravity abutments and a mass pier. The abutments and pier are founded on timber piles. The substructure concrete is unreinforced with the exception of K-bars at the abutment wingwall junctions and the bridge seat and the footings. The wingwalls are constructed at flares to the abutments, and consist of concrete gravity walls, supported on timber piles. The pre-existing bridge was a 1-span bridge with abutments constructed of dry laid, field stone.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate substructure distress in areas in the form of concrete spall, scaling and section loss. There is some concrete loss at the water line and a horizontal crack in the pier. Bridge inspection records note undermining and scour at the pier. 2007 MaineDOT Bridge Maintenance inspection reports assign the substructure a condition rating of 6 – satisfactory, and channel protection a rating of 5 – bank protection is eroded. The bridge has Bridge Sufficiency Rating of 52.2.

The MaineDOT Bridge Program is currently proposing a replacement structure consisting of a 60-foot span, voided slab superstructure founded on pile-supported integral abutments. The proposed replacement structure will have a centerline approximately matching the existing bridge centerline. The roadway profile will be raised approximately 1.0 foot on the west bridge approach and 0.58 feet on the east bridge approach.

2.0 GEOLOGIC SETTING

Norton Bridge on Fuller Road in Carmel, Maine crosses the Black Stream as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey “Surficial Geology of Stetson Quadrangle, Maine, Open-file No. 86-39” (1986) and “Surficial Geology of Bangor Quadrangle, Maine, Open-file No. 77-24” (1977) indicates that the Black Steam in Carmel is flanked by a surficial glacial marine deposit. The glacial marine geologic unit consists of silt, clay and sand. The unit is commonly a clayey silt, but sand is very abundant at the surface in some places. The unit may include small areas of till, sand and gravel that are not completely covered by the marine sediment. The glacial marine unit is composed of sediment that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor during the most recent glacial period, when the relative sea level was higher than present and seawater flooded portions of coastal and interior Maine.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, the bedrock at the project site is the Vassalboro Formation and consists of interbedded calcareous sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three test borings. Two borings were terminated with bedrock cores. Test borings BB-CBS-101 and BB-CBS-101A were drilled behind the existing west abutment. Test boring BB-CBS-102 was drilled behind the existing east abutment. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report.

The borings were drilled on August 21 and 29, 2008 using the Maine Department of Transportation (MaineDOT) drill rig. 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 MaineDOT drill rig is newly equipped with a CME automatic hammer. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor, 0.77, and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in two borings using an NQ-2 core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team Member logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of two (2) standard grain size analyses, six (6) grain size analyses with hydrometer, eight natural water contents, and one (1) one-dimensional consolidation test. The results of soil laboratory tests are included as Appendix B - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test borings BB-CBS-101, BB-CBS-101A and BB-CBS-102 generally consisted of coarse-grained and fine-grained fill soils, overlying a silt clay glacial marine deposit, all underlain by glacial till and metamorphic bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. The boring logs are provided in Appendix A – Boring Logs. A brief summary description of the strata encountered follows:

5.1 Fill

A layer of fill was encountered in borings drilled within the approach fills leading to the abutments. The encountered fill layer is approximately 8.8 to 10 feet thick. The upper fill subunit generally consisted of brown, dry to damp, sand, little to some gravel, trace to no silt. The lower fill subunit is comprised of reworked native soils and consisted of grey-brown, mottled, damp to moist, medium stiff, silt, some to little clay, some to little sand, trace to little gravel. Traces of wood fragments were detected in the silt unit in both borings.

SPT N-values in the granular fill unit ranged from 13 to 15 blows per foot (bpf), indicating a soil that is medium dense in consistency. SPT N-values in the cohesive fill unit ranged from 5 to 8 bpf, indicating the soil is medium stiff in consistency.

Grain size analyses were conducted on two (2) samples from the coarse-grained fill subunit. Grain size analyses resulted in the soil being classified as A-1-b under the AASHTO Soil Classification System and as SM under the Unified Soil Classification System. Grain size analyses conducted on two (2) samples from the silt fill resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and CL-ML or ML under the Unified Soil Classification System.

5.2 Glacial Marine Deposit

An glacial marine deposit, known as the Presumpscot Formation, was encountered below approach embankment fill deposits. The encountered thickness of the unit was approximately 13.9 to 16.2 feet thick. The deposit consisted of predominately grey, wet, clayey silt, and silt, some clay, trace to no sand, trace to no gravel.

SPT N-values in silt clay deposit ranged from weight of rods (WOR) to 9 bpf, indicating that the silt deposit is very soft to stiff in consistency.

Grain size analyses were conducted on four (4) samples from the glacial marine unit. Grain size analyses resulted in the soil being classified as an A-4 or A-6 under the AASHTO Soil Classification System and as ML or CL-ML under the Unified Soil Classification System.

Five undrained vane shear tests, conducted within the glacial marine deposit, measured undrained shear strengths ranging from approximately 312 to 491 psf while the remolded shear strengths ranged from 45 to 89 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the glacial marine unit is determined to have a sensitivity ranging from 3.5 to 10, which correlates to a soil that is "moderately sensitive" to "very sensitive" to disturbance. Atterberg Limits tests on samples from the deposit determined moisture contents ranged from approximately 22 to 36 percent and plastic limits ranged from 22 to 30. The natural water contents of three of the four tested samples exceed the liquid limits. Disturbance by construction activity can cause remolding in these soils and has the potential to transform this type of soil into a viscous liquid. The calculated values of liquidity index for the soils tested where greater than 1 for the three (3) soil samples. Therefore, this soil has a high potential to become a viscous liquid if disturbed by construction activity. Conversion can be localized, such as in response to pile driving, or involve a larger area.

The following table summarizes the results of Atterberg Limits test made from samples of the silt-clay unit:

Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-CBS-101, 4D	clayey SILT	29.9	28	22	6	1.31
BB-CBS-101, 1U	SILT, some clay	34.0	26	23	3	3.67
BB-CBS-102, 3D	SILT, some clay	21.8	37	30	7	-1.17
BB-CBS-102, 5D	clayey SILT	36.1	31	25	6	1.85

Table 1. Atterberg Limits Test Results

Laboratory test results can be found in Appendix B - Laboratory Data. This testing information is also shown on the boring logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

5.3 Glacial Till

A thin layer of weathered glacial till was encountered in the borings. The encountered thickness was approximately 1.4 to 2.6 feet thick at the boring locations. The glacial till unit has a high portion of fine grained soil and weathered bedrock. The glacial till generally consisted of brown, mottled, silty gravel and grey, moist, sandy silt layered with weathered bedrock.

SPT N-values in the glacial till unit were greater than 50 bpf in boring BB-CBS-102 and would likely be greater than 50 bpf in boring BB-CBS-101A (encountered refusal). This indicates a soil of very dense consistency.

5.4 Bedrock

Bedrock at the site was encountered and cored beginning at a depth of 25.6 feet bgs and Elevation 111.10 feet in boring BB-CBS-101. Bedrock was encountered and cored beginning at a depth of 26.4 feet bgs and Elevation 110.20 feet in boring BB-CBS-102.

The bedrock at the site is identified as grey to green-grey, fine grained, metasedimentary greenschist, moderately hard to friable and soft, very slightly weathered to severely weathered, joint set along foliation, dipping at steep angles, very closely spaced, tight to open, fresh to silt infilled, with occasional quartz seams. The rock quality designation (RQD) of the bedrock was determined to range from 0 to 77 percent, correlating to a rock mass quality of very poor to good.

The following table summarizes top of bedrock elevations at the exploration locations.

Proposed Substructure	Boring	Station	Depth to Bedrock (feet)	Elevation of Bedrock Surface (feet)
Abutment 1	BB-CBS-101	4+69.2	25.6	111.1
Abutment 2	BB-CBS-102	5+25.1	26.4	110.2

Table 2. Elevation of Bedrock Surface at Exploration Locations

5.5 Groundwater

Groundwater observations were not recorded in the logs. However, groundwater level is inferred to be at a depth of approximately 10 feet bgs or approximately Elevation 127 feet. Groundwater levels will fluctuate with seasonal changes, runoff, and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

The following foundations were considered for the replacement bridge substructures and evaluated for practicality and effectiveness during preliminary design:

- Full height, cantilever-type concrete abutments founded on new spread footings supported on bedrock or seal concrete founded on bedrock.
- Integral abutments supported on short piles, with piles driven behind the existing abutments. The existing gravity abutments may be partially demolished and the remaining portion left in place as protection for the new pile-supported abutments.

Preliminary design phase evaluations have resulted in a proposed replacement bridge consisting of a 60-foot span, precast/prestressed concrete voided slab superstructure founded on integral H-pile supported abutments. This report addresses this selected foundation alternative.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

7.1 General - Integral Abutment Founded on Driven H-piles

For a 60-foot span integral structure, we anticipate a voided slab superstructure depth of approximately 2.5 feet, and abutment breastwall height of approximately 7 feet. This implies a depth of approximately 10 feet may be required to accommodate the superstructure and stub abutment. Bedrock was encountered at a depth of approximately 25.6 feet bgs below the west bridge approach and approximately 26.4 feet bgs below the east bridge approach. This results in estimated pile lengths of approximately 17 feet. This data is summarized in Table 3.

Proposed Structure	Approximate Bedrock Elevation (feet)	Estimated Pile Cap Elevation (feet)	Estimated Integral Pile Lengths (feet)
Abutment 1	111.1	128	17
Abutment 2	110.2	127.5	17

Table 3. Estimated Pile Lengths

The MaineDOT and the University of Maine (UMO) have investigated the performance of integral abutment bridges at sites with shallow bedrock and have instrumented and monitored the piles at the Nash Stream Bridge in Coplin Plantation, Maine. Preliminary evaluation of the field data from the research study indicate 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 an H-pile embedded in 14 feet of soil.

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

- In consideration of (a) the consequences of scour and pile exposure, (b) the need to limit pile tip movement, and (c) obtaining pile behavior associated with plastic stress redistribution and inelastic rotation in the pile, a minimum pile length of 10 feet is recommended. The UMO research indicates a pinned pile tip condition, and some rotation at the pile tip is acceptable.

- Piles supporting integral abutments should be designed in accordance AASHTO LRFD criteria.

- Since the abutment 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. An L-Pile analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, moments and pile head displacements applied. Achievement of an assumed pinned condition at the pile tip should be also confirmed with an L-Pile analysis. As the proposed piles for this project will be short and will not achieve fixity, the resistance for the piles should be determined for compliance with the interaction equation.

- Driven piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

- The stream velocity should be low and there should be low potential for scour action, wave action, storm surge and ice damage. This is to ensure the long-term integrity of the bridge approach fills and riprap abutment slopes, which provide lateral support to pile groups.

- The existing abutments may be left in place as protection for the pile supported abutments with 1.75H:1V slopes constructed to the tops of the partially demolished, existing abutments. Slopes should be protected with riprap over an erosion control geotextile.

7.2 Integral Pile Design

The piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design axial loads. 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.

The designer shall design H-piles at the strength limit states considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.2.1 below.

The design of H-piles at the service limit state shall consider tolerable horizontal movement of the piles, and overall stability of the pile group and displacements considering scour at the design flood event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the design flood can support the unfactored Strength Limit States loads with a resistance factor of 1.0. The design flood for scour is defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

7.2.1 Strength Limit State Design

The nominal compressive resistance (P_n) in the structural limit state for piles loaded in compression shall be as specified in LRFD 6.9.4.1. For preliminary analysis, the H-piles were assumed fully embedded, and the column slenderness factor, λ, was taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections presented in this report were calculated using a resistance factor, ϕ_c, of 0.60 and a λ of 0. It is the responsibility of the Structural Designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced length and K-values from project specific L-Pile analyses and recalculate structural resistances.

For the portion of the pile which is theoretically in pure compression, i.e. below the point of fixity, the factored structural axial resistances of four H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60. Short pile will not achieve a fixed condition, therefore the factored structural axial resistance will be controlled by the combined axial and flexural resistance of the pile. This analysis is the responsibility of the Structural Designer.

The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ_{stat}, of 0.45. The calculated factored geotechnical resistances of four H-pile sections were calculated and are provided in Table 4, below.

Drivability analyses of the four proposed H-pile sections were conducted. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be no more that 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is $\phi_{dyn} = 0.65$. Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. When a pile group is nonredundant, i.e., there are less than five (5) piles, LRFD Article 10.5.5.2.3 dictates a 20 percent reduction of the resistance factor value of 0.65. The factored pile resistances provided in this report assume a four-pile group, and therefore are factored by resistance factor, ϕ_{dyn}, of 0.52. If the final design calls for a five (5) pile group, the factored geotechnical resistance of the piles should be reevaluated by the Geotechnical Engineer using a resistance factor, ϕ_{dyn}, of 0.65.

For the strength limit state, the calculated factored axial compressive structural, geotechnical and drivability resistances of four (4) proposed H-piles sections are summarized in Table 4 below. Supporting calculations can be found in Appendix C – Calculations, at the end of this report.

	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance ϕ_c=0.60[1]	Geotechnical Resistance φ_{stat} = 0.45	Drivablity Resistance	Governing Pile Resistance
HP 12 x 53	465	47	227	227
HP 14 x 73	642	64	275	275
HP 14 x 89	783	78	309	309
HP 14 x 117	1032	103	364	364

Table 4. Factored Axial Structural Resistances for Four H-Pile Sections for Strength Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, the factored axial drivablity resistance is less than the factored axial structural resistance, and local experience supports the estimated factored resistance from the drivability analyses. Therefore, the recommended governing resistance for pile design should be the factored drivability resistance in Table 4. The maximum factored axial pile load should not exceed the calculated factored drivability pile resistances in Table 4.

The top of the piles should be checked for resistance against combined axial load and flexure, per LRFD Article 6.15. This axial load will govern the design. The upper portion of the pile is defined per LRFD Figure C6.15.2-1 as that portion of the pile above the point of second inflection in the moment vs. pile depth curve, or at the lowest point of zero deflection. For strength limit state load combinations, resistance factors of 0.70 for axial resistance (ϕ_c) and 1.0 for flexural resistance (ϕ_f) should be applied to the combined axial and flexural resistance of the pile in the interaction equation. The resistance of the pile in the lower zone need only be checked against axial load, but only if the piles are fully fixed.

[1] Assuming $\lambda = 0$ and ϕ_c =0.60. Short pile will not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

7.2.2 Service and Extreme Limit State Design

The design of piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and deflections resulting from scour at the design flow event. For the service and extreme limit states, a resistance factor of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.2 and 10.5.5.3.

The extreme limit state design shall determine that there is adequate nominal foundation resistance remaining after scour due to the design flood to resist the unfactored Strength Limit States loads with a resistance factor of 1.0. The unfactored Strength Limit State loads shall include any debris loads occurring during the flood event.

The calculated factored axial structural, geotechnical and drivablity resistances of four (4) H-pile sections were calculated for the service and extreme limit states and are provided below in Table 5. Supporting documentation is provided in Appendix C – Calculations.

	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance [2]	Geotechnical Resistance	Drivability Resistance	Governing Pile Resistance
HP 12 x 53	775	105	436	436
HP 14 x 73	1070	143	528	528
HP 14 x 89	1305	174	595	595
HP 14 x 117	1720	229	700	700

Table 5. Factored Axial Pile Resistance for H-Piles for Service and Extreme Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, the factored axial drivability resistance is less than the factored axial structural resistance, and local experience supports the estimated factored resistance from the drivablity analyses. Therefore, it is recommended that the governing resistance used in design be the factored drivability resistance in the Table 5. The maximum factored axial pile loads for the service and extreme limit states should not exceed the calculated factored drivablity pile resistance in Table 5.

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

[2] Assuming $\lambda = 0$. Short pile will not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

7.2.3 Driven Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock, pile splices should not be permitted.

Contract documents should 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 substructure. The first pile driven at each abutment should be dynamically tested to confirm capacity 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.

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 provided that a minimum of three to four piles out of the total number of piles driven at the project site are dynamically tested, in accordance with LRFD Tables 10.5.5.2.3-1 and -3. LRFD Article 10.5.5.2.3 further specifies that the resistance factor, ϕ_{dyn}, of 0.65 be reduced by 20 percent when applied to nonredundant pile groups, i.e. pile groups with less than five (5) piles. This results in a resistance factor, ϕ_{dyn}, of 0.52. The factored pile resistances provided in this report assume a four-pile group, and therefore are factored by ϕ_{dyn} of 0.52. With the use of a reduced resistance factor, the η_R factor provided in LRFD Article 1.3.4 should not be increased to address the lack of foundation redundancy. If the final design calls for a five (5) pile group, the factored geotechnical resistance of the piles should be reevaluated by the Geotechnical Engineer using a resistance factor, ϕ_{dyn}, of 0.65.

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 $0.90\phi_{da}$ F_y, where ϕ_{da} is equal to 1.0, 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 5 to 15 blows per inch (bpi). 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.

7.3 Integral Stub Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments at the strength limit state shall consider pile group failure and structural failure. Strength limit state shall also consider the foundation/pile group resistance after scour due to the design flood, using unfactored loads and nominal pile/foundation resistances. The design of cantilevered, in-line wingwalls at the strength limit state shall consider overturning, lateral sliding and structural failure.

A resistance factor of 1.0 shall be used for abutment design at the service limit state, including: settlement, excessive horizontal movement and movement resulting from scour at the design flood. The overall global 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 shall also check that the nominal foundation resistance remaining after scour due to the design flood can support the unfactored Strength Limit States loads with a resistance factor of 1.0. The unfactored Strength Limit State loads shall include any debris loads occurring during the flood event.

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure. The Rankine passive earth pressure coefficient, K_p, of 3.3 is recommended.

In-line wingwall sections that are integral with the abutment, should also be designed to withstand a maximum earth pressure equal to the passive earth pressure state. A Rankine passive earth pressure coefficient, $K_{p,}$ of 3.3 is recommended.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and walls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads is permitted per LRFD 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) taken from Table 6 below:

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

Table 6. Equivalent Height of Soil for Estimating Live Load Surcharge

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

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

Slopes in front of pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile and not exceed 1.75H:1V.

7.4 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at all limits states. Design at the strength limit state should consider loss of lateral and vertical support to due to scour. Design at the extreme limit state should check that the nominal foundation resistance after the design flood event is no less than the unfactored Strength Limit State loads. At the service limit state the design shall limit movements and overall stability considering scour at the design flood. These changes in foundation conditions shall be investigated at abutments and wingwalls.

In general, for scour protection, any footings which are constructed on soil deposits should be embedded at least 2 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to BDG Section 2.3.11 for information regarding scour design.

For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Stone riprap shall conform to item number 703.26 Plain and Hand Laid Riprap of the Standard Specification and be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material conforming to item number 703.19, of the Standard Specification. Riprap may be placed at the toes of abutments, wingwalls and retaining walls, as required.

7.5 Settlement

The roadway profile of bridge approaches will be raised approximately 1 foot. Post-construction settlement due to consolidation of the glacial marine foundation soil is calculated to be approximately 1.5 inches near the mid to lower fill extension areas. Any settlement of the bridge abutments will be due to elastic settlement of the bedrock or piles, which is assumed to occur during construction will be negligible.

7.6 Frost Protection

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

It is anticipated that return wingwalls at the corners of the abutments will be straight extension wings. However, should any walls be founded on spread footings on compacted granular borrow, the foundations should be designed with an appropriate embedment for frost protection. According to the BDG, Carmel, Maine has a design freezing index of approximately 1760 F-degree days. An assumed water content of 10% was used for coarse grained granular fill soil above the water table. These components correlate to a frost depth of approximately 7.4 feet. Modberg, a computer program, developed by U.S. Army Cold Regions Research and Engineering Laboratory, was used to check the calculated maximum depth of frost penetration. The calculated depth of frost according to the Modberg solution, which is based on the Modified Berggren Equation, is 6.1 feet.

We recommend that foundations placed on granular fill soil should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

7.7 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of seismic zone. Norton Bridge is not on the National Highway System, and is therefore not classified as functional important. Furthermore, the bridge is not classified as a major structure, since the bridge construction costs will not exceed $10 million. These criteria eliminate the BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and bridge seat dimensions shall be satisfied per LRFD 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) = 0.069g
- Short-term (0.2-second period) spectral acceleration coefficient, (SDs)= 0.370g
- Long-term (1.0-second period) spectral acceleration coefficient, (SD1) = 0.156g
- Site Class E (site soils with an average blow count less than 15 bpf or an undrained shear strength less than 1000 psf)
- Seismic Zone 2 (based on a SD1 between 0.15g and 0.30g)

7.8 Construction Considerations

Construction of the abutments will require soil excavation. Construction activities may require internally braced cofferdams and earth support systems. The silt-clay soils at the integral abutment subgrade will be susceptible to rutting as a result of exposure to water or construction traffic. The contractor shall protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that he contractor remove and replace the disturbed materials with compacted MaineDOT Standard Specification 703.20, Gravel Borrow.

The subgrade native silt-clay soils within the project area are both poorly drained and moderately to highly frost susceptible. In some locations, these soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The contractor should control groundwater, surface water infiltration, and soil erosion.

Using the excavated glacial marine silt-clay or native silt fill soils as structural backfill should not be permitted, and may only be used as common borrow in accordance with MaineDOT Standard Specifications Sections 203 and 703.

The contractor will have to excavate the existing subbase gravel and the subgrade fill soils. These materials should not be used to re-base the new bridge approaches, but excavated subbase sand and gravel may be used as fill below subgrade level in fill embankments provided all other requirements of MaineDOT Sections 203 and 703 are met.

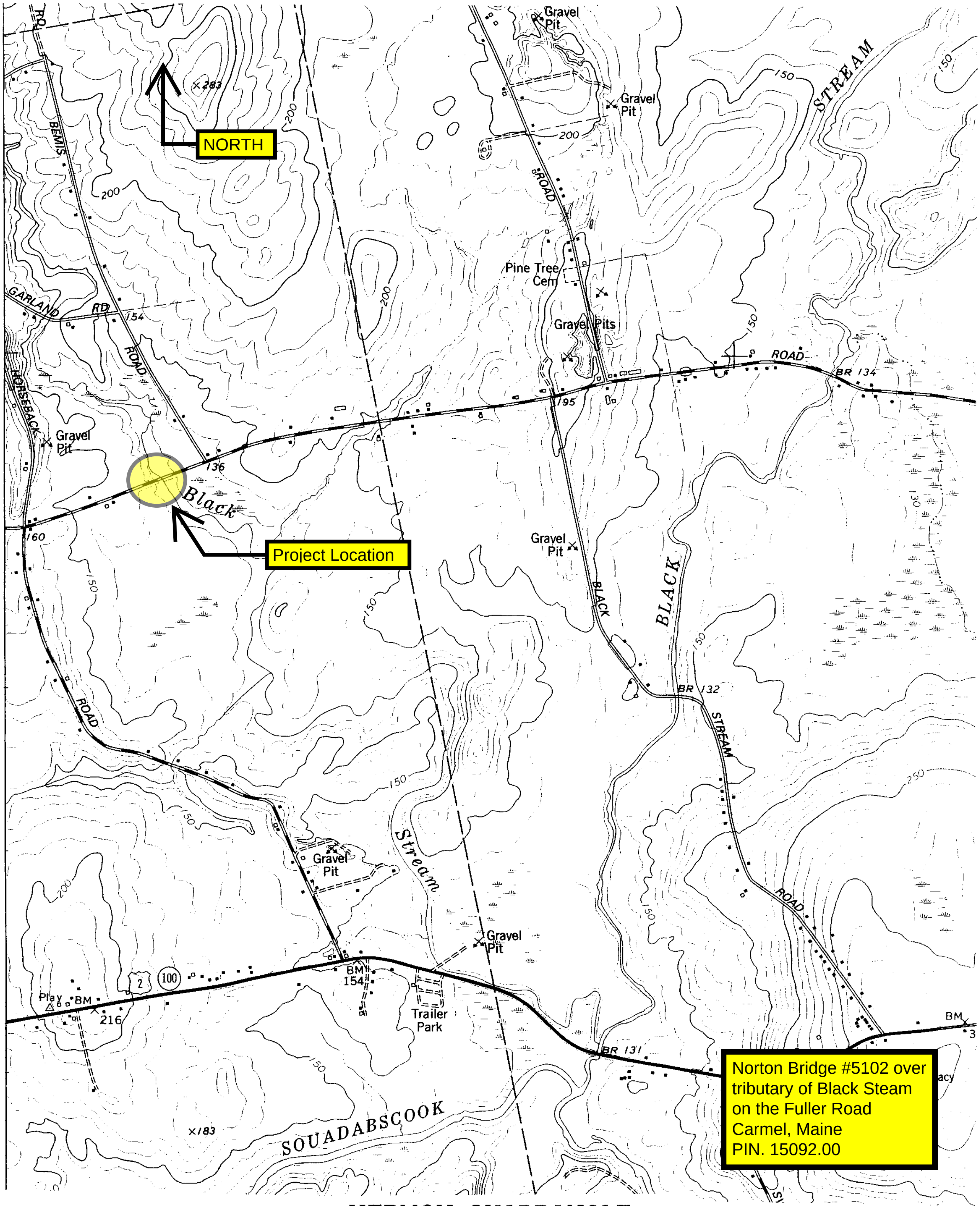
The existing abutments, wingwalls and pile groups may obstruct installation of piles. Removal of all or some of the existing substructures may be necessary. This may also necessitate the replacement of excavated backfill soils with compacted granular fill before pile driving can commence.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Norton Bridge in Carmel, 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 plans and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



HERMON QUADRANGLE
MAINE—PENOBSCOT CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
SW/4 BANGOR 15′ QUADRANGLE

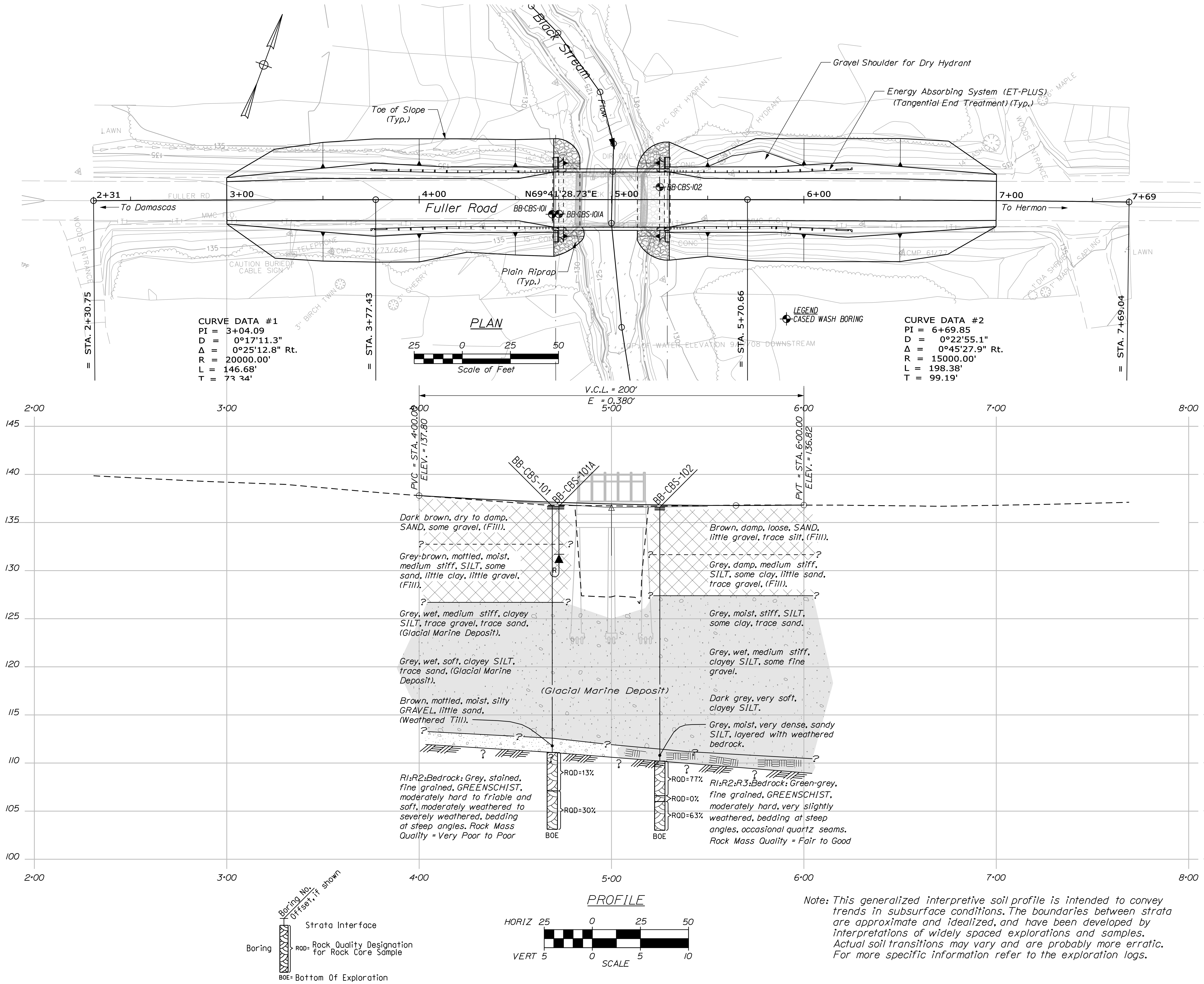
SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:4/30/2009



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

BH-1509(200)X

PIN 15092.00

BRIDGE NO. 5102

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L. KRUSINSKI	
CHECKED-REVIEWED	T. WHITE	NOV 2008
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

NORTON BRIDGE
Tributary of BLACK STREAM
CARMEL PENOBSCOT COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:4/28/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Norton Bridge #5102 over a tributary of Black Stream on	Location:	Carmel, Maine	Boring No.:	BB-CBS-101
				PIN:	15092.00
Driller:	MaineDOT	Elevation (ft.)	136.7	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	L. Krusinski/B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/21/08; 07:45-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	4+69.2, 7.6 Rt.	Casing ID/OD:	HW	Water Level*:	Not recorded
Hammer Efficiency Factor: 0.77		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	136.40	PAVEMENT. (0.30) Dark brown, dry to damp, fine to coarse SAND, some gravel, trace silt, (Fill).	
								132.70	(4.00)	
5	2D	24/18	5.00 - 7.00	2/3/3/2	6	8	17		Grey-brown, mottled, moist, medium stiff, SILT, some sand, little clay, little gravel, (Fill).	G#210021 A-4, CL-ML WC=26.2%
							17			
							17			
							33			
							55		Wood from 9.5' to 10.0' bgs.	
10	3D	24/4	10.00 - 12.00	4/3/2/3	5	6	28	126.70	(10.00) Grey, wet, medium stiff, clayey SILT, trace gravel, trace fine sand, (Glaciomarine Deposit).	
							24			
							45			
							63			
							67			
15	4D/MV	24/12	15.00 - 17.00	WOH/WOH/2/2	2	3	45		Grey, wet, soft, clayey SILT, trace sand, slightly plastic, homogeneous, (Glaciomarine Deposit). Failed 55x110 mm vane attempt, could not push.	G#210022 A-4, CL-ML WC=29.9% LL=28 PL=22 PI=6
							50			
							31			
							30			
							39			
20	1U	24/24	20.00 - 22.00	Hydraulic Push			31		Grey, wet, soft, SILT, some clay, trace sand.	G,C#210023 A-4, ML WC=34.0% LL=26 PL=23 PI=3
							38			
							34			
	V1		23.00 - 23.37	Su=312/89 psf			55		55x110 mm vane raw torque readings: V1: 7.0/2.0 ft-lbs	
	MV						76	112.80	(23.90) Failed 55x110 mm vane attempt, could not push.	
25	5D / R1	7.2/4 / 48/48	25.00 - 25.60 / 25.60 - 29.60	24/19(1.2") / RQD = 13%	---		a85 / NQ-2	111.10	a85 blows for 0.6'. Brown, mottled, moist, silty fine to coarse angular GRAVEL, little sand, (Weathered Glacial Till). (25.60)	
	R2	48/36	29.60 - 33.60	RQD = 30%						
								103.10	(33.60)	

Top of Bedrock at Elev. 111.1'
R1: Bedrock: Oxidized, stained, grey-green, fine grained metasedimentary (GREENSCHIST), moderately hard to friable in upper 1.7', severely weathered to slightly weathered, bedding/foliation at steep angles, tight, stained/weathered, upper 1.2' hightly fractured and weathered. (Vassalboro Formation). Rock Mass Quality: very poor.
R1:Core Times (min:sec)
25.6-26.6' (2:42)
26.6-27.6' (2:50)
27.6-28.6' (2:01)
28.6-29.6' (4:20) 100% Recovery
R2: Bedrock: Grey, stained and weathered, fine grained, GREENSCHIST, moderately hard to soft weathered seams, moderately weathered, bedding close at steep angles, tight to open, fresh to silt infilled, (Vassalboro Formation). Rock Mass Quality: Poor.
R2:Core Times (min:sec)
29.6-30.6' (3:47)
30.6-31.6' (3:26)
31.6-32.6' (3:15)
32.6-33.6' (5:53) 75% Recovery
Core Block at 4.0'

Bottom of Exploration at 33.60 feet below ground surface.

Depth (ft.): 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Remarks:

No sample 1D. Soil description interpolated from BB-CBS-101A.

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-CBS-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Norton Bridge #5102 over a tributary of Black Stream on	Location:	Carmel, Maine	Boring No.:	BB-CBS-101A
				PIN:	15092.00
Driller:	MaineDOT	Elevation (ft.)	136.7	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/21/08; 07:30-07:45	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	4+72.7, 7.5 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.77		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	136.40	PAVEMENT. (0.30)	
	1D	24/18	1.00 - 3.00	9/8/4/6	12	15			Brown, damp, medium dense, fine to coarse SAND, some silt, trace gravel, (fILL).	G#210020 A-1-b, SM WC=8.0%
5								131.70	(5.00) Bottom of Exploration at 5.00 feet below ground surface. Hit Boulder, moved to BB-CBS-101.	

Depth (ft.): 0, 5, 10, 15, 20, 25

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CBS-101A

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Norton Bridge #5102 over a tributary of Black Stream on	Location:	Carmel, Maine	Boring No.:	BB-CBS-102
				PIN:	15092.00
Driller:	MaineDOT	Elevation (ft.)	136.6	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	L. Krusinski	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/29/08; 07:30-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+25.1, 6.6 Lt.	Casing ID/OD:	HW	Water Level*:	Not noted
Hammer Efficiency Factor: 0.77		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	136.30	PAVEMENT. (0.30)	
	1D	24/19	1.00 - 3.00	6/5/5/5	10	13			Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt. (Fill)	G#210026 A-1-b, SM WC=6.5%
5	2D	24/24	5.00 - 7.00	2/2/2/3	4	5	4	131.60	(5.00) Grey, damp, medium stiff, SILT, some clay, little sand, trace gravel, occasional thin lense/partings of wood slivers and stained, weathered silt seams, otherwise homogeneous.	G#210027 A-4, ML WC=30.3% LL=32 PL=27 PI=5
							12			
							23			
							43		Change to brown wash water at 8.8' bgs.	
							59			
10	3D	24/24	10.00 - 12.00	3/4/3/5	7	9	31		Grey, moist, medium stiff, mottled SILT, some clay, trace sand, homogeneous, (Glaciomarine Deposit).	G#210028 A-4, CL-ML WC=21.8% LL=37 PL=30 PI=7
							45			
							54			
							63			
							78			
15	4D	24/4	15.00 - 17.00	2/3/2/2	5	6	47		Grey, wet, medium stiff, mottled clayey SILT, some fine gravel.	
							46		Change at 16.0' bgs. to soft PF, washed ahead to 18.0' bgs.	
							50		Changed to grey glaciomarine clay-silt based on wash water at 17.4' bgs.	
	5D / V1	24/24	18.00 - 20.00 / 18.00 - 18.37	Hydraulic Push / Su=469/89 psf			50		Dark grey, wet, soft clayey SILT, trace sand, homogeneous, dilatent, (Glaciomarine Deposit). Pushed thru vane test from 18.0-19.0' bgs. 55x110 mm vane raw torque readings: V1: 10.5/2.0 ft-lbs V2: 11.0/1.1 ft-lbs	G#210029 A-4, ML WC=36.1% LL=31 PL=25 PI=6
	V2		19.00 - 19.37	Su=491/49 psf			48			
20	MD/MU	24/0	20.00 - 22.00	WOR/WOR/WOR/WOR	---		77		Failed one split spoon and tube attempt at 20.0' bgs. Dark grey, very soft, clayey SILT, (Glaciomarine Deposit).	
							56			
							48			
	V3		23.00 - 23.37	Su=339/45 psf			42		55x110 mm vane raw torque readings: V3: 7.6/1.0 ft-lbs	
	V4		24.00 - 24.37	Su=424/49 psf			64		V4: 9.5/1.1 ft-lbs	
25	6D	16.8/13	25.00 - 26.40	13/8/64(4.8")	---		126		Grey-brown, moist, very dense, sandy SILT, layered with seams of weathered (Greenschist) bedrock. b47 blows for 0.1'.	
	R1	43.2/ 43.2	26.40 - 30.00	RQD = 77%			b47 / NQ-2	110.20	(26.40)	
30	R2 / R3	7.2/7.2 / 36/36	30.00 - 30.60 / 30.60 - 33.60	RQD = 0% / RQD = 63%						
								103.00	(33.60)	

Top of Bedrock at Elev. 110.2'
R1: Bedrock: Green-grey, fine grained metasedimentary (GREENSCHIST), moderately hard, very slightly weathered, bedding very close at steep angles, bedding surfaces tight, silty, (Upper 0.7' softer, more fractured and weathered). (Vassalboro Formation). Rock Mass Quality: Good. R1:Core Times (min:sec)
26.4-27.4' (2:23)
27.4-28.4 (2:00)
28.4-29.4' (2:12)
29.4-30.0' (1:42) 100% Recovery
Core Blocked at 3.6'.
R2:Bedrock: GREENSCHIST with 1.5" quartz vein. Core block at 0.6'.
R3: Bedrock: Green-grey, fine grained, metasedimentary (GREENSCHIST), moderately hard, slightly weathered jointing along bedding at steep angles, close, tight, surfaces fresh to weathered, soft stained seam at 2.4' bgs. (Vassalboro Formation). Rock Mass Quality: Fair.
R3: Core Times (min:sec)
30.6-31.6' (1:22)
31.6-32.6' (2:59)
32.6-33.6' (3:47) 100% Recovery
Core Blocked at 3.6'.

Bottom of Exploration at 33.60 feet below ground surface.

Depth (ft.): 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Remarks:

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

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

Page 1 of 1

Boring No.: BB-CBS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1509(200)X

BRIDGE NO. 5102 | PIN 15092.00 | BRIDGE PLANS

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	NOV 2008
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

NORTON BRIDGE
Tributary of BLACK STREAM
CARMEL PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER

3

OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Norton Bridge #5102 over a tributary of Black Stream on Fuller Road
Location: Carmel, Maine

Boring No.: BB-CBS-101A
PIN: 15092.00

Driller:	MaineDOT	**Elevation (ft.)**	136.7	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/21/08; 07:30-07:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	4+72.7, 7.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	136.40		PAVEMENT. 0.30	
	1D	24/18	1.00 - 3.00	9/8/4/6	12	15				Brown, damp, medium dense, fine to coarse SAND, some silt, trace gravel, (fILL).	G#210020 A-1-b, SM WC=8.0%
5								131.70		5.00	
										Bottom of Exploration at 5.00 feet below ground surface. Hit Boulder, moved to BB-CBS-101.	
10											
15											
20											
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Norton Bridge #5102 over a tributary of Black Stream on Fuller Road
Location: Carmel, Maine

Boring No.: BB-CBS-101
PIN: 15092.00

Driller:	MaineDOT	**Elevation (ft.)**	136.7	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/21/08; 07:45-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+69.2, 7.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Not recorded
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	136.40		PAVEMENT. -0.30- Dark brown, dry to damp, fine to coarse SAND, some gravel, trace silt, (Fill).	
								132.70		-4.00-	
5	2D	24/18	5.00 - 7.00	2/3/3/2	6	8	17			Grey-brown, mottled, moist, medium stiff, SILT, some sand, little clay, little gravel, (Fill).	G#210021 A-4, CL-ML WC=26.2%
							17				
							17				
							33				
							55			Wood from 9.5' to 10.0' bgs.	
10	3D	24/4	10.00 - 12.00	4/3/2/3	5	6	28	126.70		-10.00- Grey, wet, medium stiff, clayey SILT, trace gravel, trace fine sand. (Glaciomarine Deposit).	
							24				
							45				
							63				
							67				
15	4D/MV	24/12	15.00 - 17.00	WOH/WOH/2/2	2	3	45			Grey, wet, soft, clayey SILT, trace sand, slightly plastic, homogeneous. (Glaciomarine Deposit). Failed 55x110 mm vane attempt, could not push.	G#210022 A-4, CL-ML WC=29.9% LL=28 PL=22 PI=6
							50				
							31				
							30				
							39				
20	1U	24/24	20.00 - 22.00	Hydraulic Push			31			Grey, wet, soft, SILT, some clay, trace sand.	G,C#210023 A-4, ML WC=34.0% LL=26 PL=23 PI=3
							38				
							34				
	V1		23.00 - 23.37	Su=312/89 psf			55	112.80		55x110 mm vane raw torque readings: V1: 7.0/2.0 ft-lbs -23.90-	
	MV						76			Failed 55x110 mm vane attempt, could not push.	
25											

Remarks:

No sample 1D. Soil description interpolated from BB-CBS-101A.

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:	Norton Bridge #5102 over a tributary of Black Stream on Fuller Road	**Boring No.:**	BB-CBS-101		
Location:	Carmel, Maine	**PIN:**	15092.00		
Driller:	MaineDOT	**Elevation (ft.)**	136.7	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski/B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/21/08; 07:45-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	4+69.2, 7.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Not recorded
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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	5D	7.2/4	25.00 - 25.60	24/19(1.2")	---		a85	111.10		a85 blows for 0.6'. Brown, mottled, moist, silty fine to coarse angular GRAVEL, little sand, (Weathered Glacial Till). 25.60	
	R1	48/48	25.60 - 29.60	RQD = 13%			NQ-2			Top of Bedrock at Elev. 111.1' R1: Bedrock: Oxidized, stained, grey-green, fine grained metasedimentary (GREENSCHIST), moderately hard to friable in upper 1.7', severely weathered to slightly weathered, bedding/foliation at steep angles, tight, stained/weathered, upper 1.2' hightly fractured and weathered. (Vassalboro Formation). Rock Mass Quality: very poor. R1:Core Times (min:sec) 25.6-26.6' (2:42) 26.6-27.6' (2:50) 27.6-28.6' (2:01) 28.6-29.6' (4:20) 100% Recovery	
	R2	48/36	29.60 - 33.60	RQD = 30%						R2: Bedrock: Grey, stained and weathered, fine grained, GREENSCHIST, moderately hard to soft weathered seams, moderately weathered, bedding close at steep angles, tight to open, fresh to silt infilled, (Vassalboro Formation). Rock Mass Quality: Poor. R2:Core Times (min:sec) 29.6-30.6' (3:47) 30.6-31.6' (3:26) 31.6-32.6' (3:15) 32.6-33.6' (5:53) 75% Recovery Core Block at 4.0' 33.60	
30											
								103.10		**Bottom of Exploration at 33.60 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

No sample 1D. Soil description interpolated from BB-CBS-101A.

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: Norton Bridge #5102 over a tributary of Black Stream on Fuller Road
Location: Carmel, Maine

Boring No.: BB-CBS-102
PIN: 15092.00

Driller:	MaineDOT	**Elevation (ft.)**	136.6	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/29/08; 07:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+25.1, 6.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Not noted
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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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)
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N-uncorrected = Raw field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	136.30		PAVEMENT. 0.30	
	1D	24/19	1.00 - 3.00	6/5/5/5	10	13				Brown, damp, medium dense, fine to coarse SAND, little gravel, trace silt. (Fill)	G#210026 A-1-b, SM WC=6.5%
5	2D	24/24	5.00 - 7.00	2/2/2/3	4	5	4	131.60		5.00 Grey, damp, medium stiff, SILT, some clay, little sand, trace gravel, occasional thin lense/partings of wood slivers and stained, weathered silt seams, otherwise homogeneous.	G#210027 A-4, ML WC=30.3% LL=32 PL=27 PI=5
							12				
							23				
							43			Change to brown wash water at 8.8' bgs.	
							59				
10	3D	24/24	10.00 - 12.00	3/4/3/5	7	9	31			Grey, moist, stiff, mottled SILT, some clay, trace sand, homogeneous, (Glaciomarine Deposit).	G#210028 A-4, CL-ML WC=21.8% LL=37 PL=30 PI=7
							45				
							54				
							63				
							78				
15	4D	24/4	15.00 - 17.00	2/3/2/2	5	6	47			Grey, wet, medium stiff, mottled clayey SILT, some fine gravel.	
							46			Change at 16.0' bgs. to soft PF, washed ahead to 18.0' bgs.	
							50			Changed to grey glaciomarine clay-silt based on wash water at 17.4' bgs.	
	5D V1	24/24	18.00 - 20.00 ~~18.00 - 18.37~~	Hydraulic Push ~~Su=469/89 psf~~			50			Dark grey, wet, soft clayey SILT, trace sand, homogeneous, dilatent, (Glaciomarine Deposit). Pushed thru vane test from 18.0-19.0' bgs.	G#210029 A-4, ML WC=36.1% LL=31 PL=25 PI=6
	V2		19.00 - 19.37	Su=491/49 psf			48			55x110 mm vane raw torque readings: V1: 10.5/2.0 ft-lbs	
20	MD/MU	24/0	20.00 - 22.00	WOR/WOR/WOR/ WOR	---		77			V2: 11.0/1.1 ft-lbs Failed one split spoon and tube attempt at 20.0' bgs. Dark grey, very soft, clayey SILT, (Glaciomarine Deposit).	
							56				
							48				
	V3		23.00 - 23.37	Su=339/45 psf			42			55x110 mm vane raw torque readings: V3: 7.6/1.0 ft-lbs	
25	V4		24.00 - 24.37	Su=424/49 psf			64			V4: 9.5/1.1 ft-lbs	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Norton Bridge #5102 over a tributary of Black Stream on Fuller Road
Location: Carmel, Maine

Boring No.: BB-CBS-102
PIN: 15092.00

Driller:	MaineDOT	**Elevation (ft.)**	136.6	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	L. Krusinski	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/29/08; 07:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+25.1, 6.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Not noted
Hammer Efficiency Factor:	0.77	**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	6D	16.8/13	25.00 - 26.40	13/8/64(4.8")	---		126			Grey-brown, moist, very dense, sandy SILT, layered with seams of weathered (Greenschist) bedrock. b47 blows for 0.1'.	
	R1	43.2/43.2	26.40 - 30.00	RQD = 77%			b47 NQ-2	110.20		26.40 Top of Bedrock at Elev. 110.2' R1: Bedrock: Green-grey, fine grained metasedimentary (GREENSCHIST), moderately hard, very slightly weathered, bedding very close at steep angles, bedding surfaces tight, silty. (Upper 0.7' softer, more fractured and weathered). (Vassalboro Formation). Rock Mass Quality: Good. R1:Core Times (min:sec) 26.4-27.4' (2:23) 27.4-28.4 (2:00) 28.4-29.4' (2:12) 29.4-30.0' (1:42) 100% Recovery Core Blocked at 3.6'.	
30	R2	7.2/7.2	30.00 - 30.60	RQD = 0%						R2:Bedrock: GREENSCHIST with 1.5" quartz vein. Core block at 0.6'.	
	R3	36/36	30.60 - 33.60	RQD = 63%				103.00		R3: Bedrock: Green-grey, fine grained, metasedimentary (GREENSCHIST), moderately hard, slightly weathered jointing along bedding at steep angles, close, tight, surfaces fresh to weathered, soft stained seam at 2.4' bgs. (Vassalboro Formation). Rock Mass Quality: Fair. R3: Core Times (min:sec) 30.6-31.6' (1:22) 31.6-32.6' (2:59) 32.6-33.6' (3:47) 100% Recovery Core Blocked at 3.6'. 33.60	
35										**Bottom of Exploration at 33.60 feet below ground surface.**	
40											
45											
50											

Remarks:

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

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Carmel Project Number: 15092.00

Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.	W.C.	L.L.	P.I.	Classification		
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%			Unified	AASHTO	Frost
BB-CBS-101A, 1D	4+72.7	7.5 Rt.	1.0-3.0	210020	1	8.0			SM	A-1-b	II
BB-CBS-101, 2D	4+69.2	7.6 Rt.	5.0-7.0	210021	1	26.2			CL-ML	A-4	IV
BB-CBS-101, 4D	4+69.2	7.6 Rt.	15.0-17.0	210022	1	29.9	28	6	CL-ML	A-4	IV
BB-CBS-101, 1U	4+69.2	7.6 Rt.	20.0-22.0	210023	1	34.0	26	3	ML	A-4	IV
BB-CBS-101, 1U	4+69.2	7.6 Rt.	20.0-22.0	210023	1	Consolidation Test (T 216)					
BB-CBS-102, 1D	5+25.1	6.6 Lt.	1.0-3.0	210026	2	6.5			SM	A-1-b	II
BB-CBS-102, 2D	5+25.1	6.6 Lt.	5.0-7.0	210027	2	30.3	32	5	ML	A-4	IV
BB-CBS-102, 3D	5+25.1	6.6 Lt.	10.0-12.0	210028	2	21.8	37	7	CL-ML	A-4	IV
BB-CBS-102, 5D	5+25.1	6.6 Lt.	18.0-20.0	210029	2	36.1	31	6	ML	A-4	IV

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

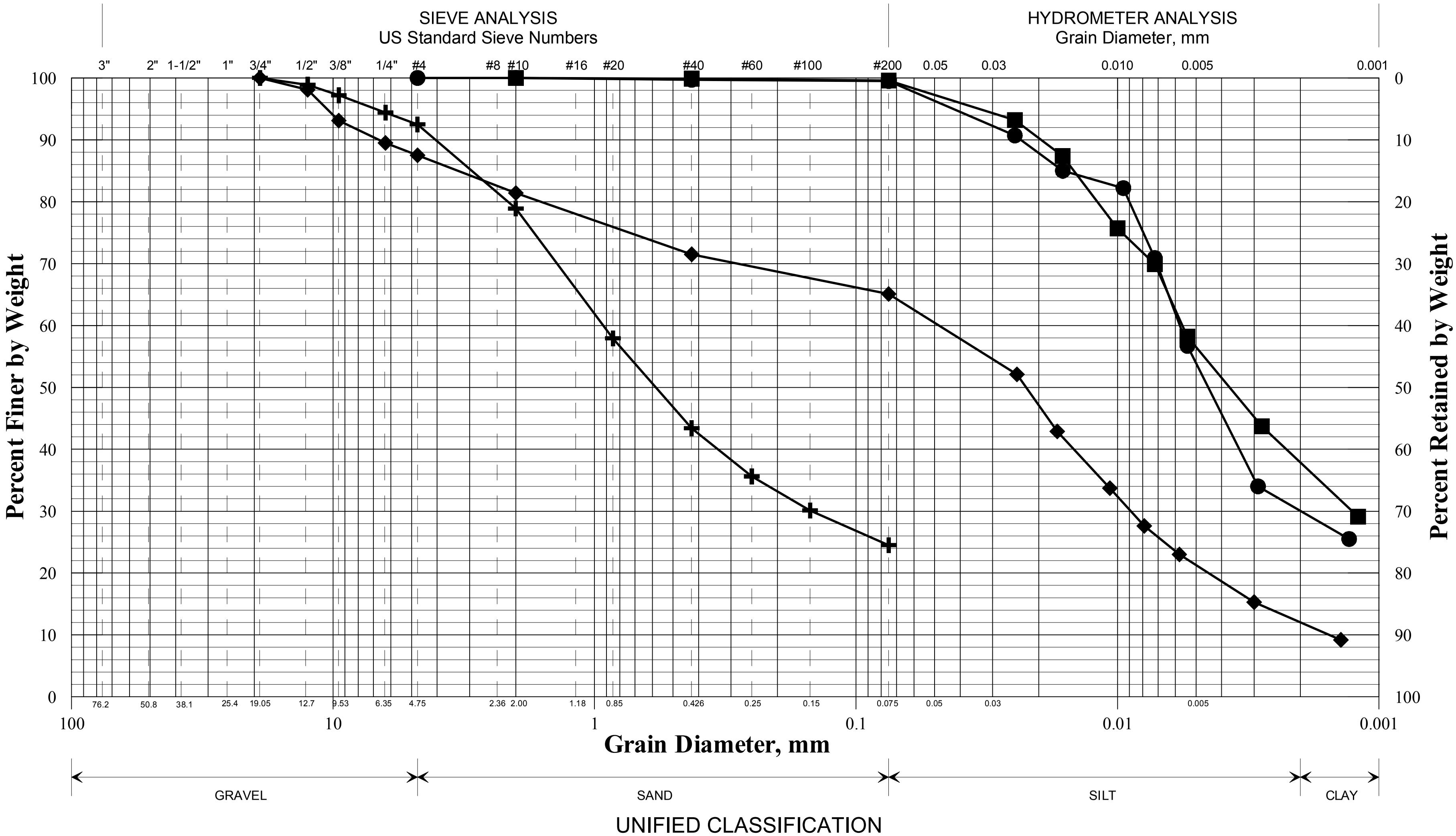
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

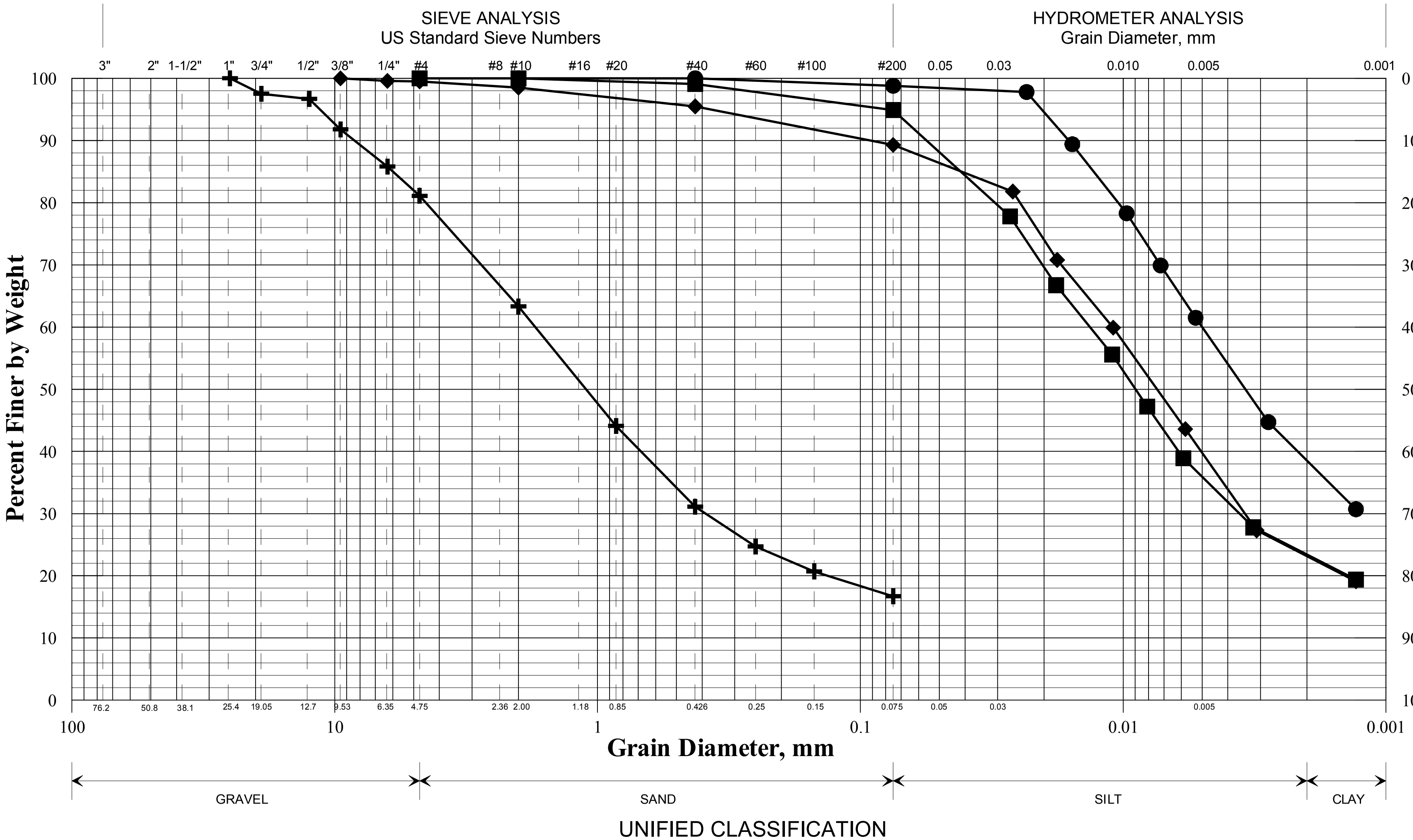


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-CBS-101A/1D	4+72.7	7.5 RT	1.0-3.0	SAND, some silt, trace gravel.	8.0			
◆	BB-CBS-101/2D	4+69.2	7.6 RT	5.0-7.0	SILT, some sand, little gravel, little clay.	26.2			
■	BB-CBS-101/4D	4+69.2	7.6 RT	15.0-17.0	Clayey SILT, trace sand.	29.9	28	22	6
●	BB-CBS-101/1U	4+69.2	7.6 RT	20.0-22.0	SILT, some clay, trace sand.	34.0	26	23	3
▲									
✖									

PIN
015092.00
Town
Carmel
Reported by/Date
WHITE, TERRY A 9/19/2008

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

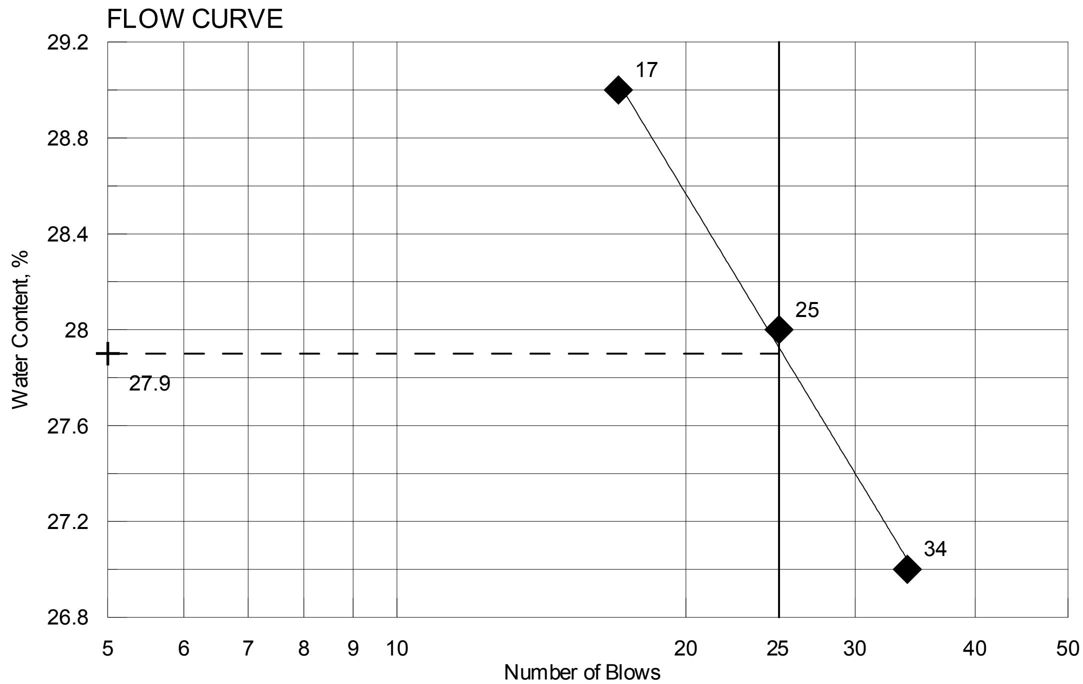


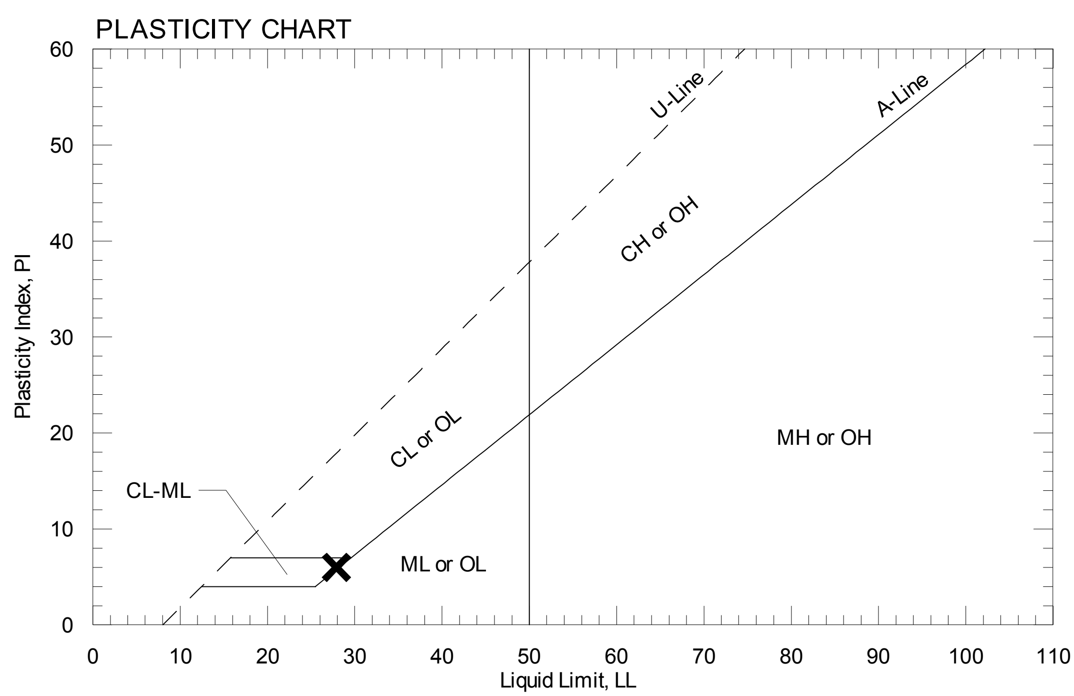
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-CBS-102/1D	5+25.1	6.6 LT	1.0-3.0	SAND, little gravel, little silt.	6.5			
◆	BB-CBS-102/2D	5+25.1	6.6 LT	5.0-7.0	SILT, some clay, little sand, trace gravel.	30.3	32	27	5
■	BB-CBS-102/3D	5+25.1	6.6 LT	10.0-12.0	SILT, some clay, trace sand.	21.8	27	20	7
●	BB-CBS-102/5D	5+25.1	6.6 LT	18.0-20.0	Clayey SILT, trace sand.	36.1	31	25	6
▲									
✖									

PIN
015092.00
Town
Carmel
Reported by/Date
WHITE, TERRY A 9/19/2008

SHEET 2

TOWN	Carmel	Reference No.	210022
PIN	015092.00	Water Content, %	29.9
Sampled	8/21/2008	Plastic Limit	22
Boring No./Sample No.	BB-CBS-101/4D	Liquid Limit	28
Station	4+69.2	Plasticity Index	6
Depth	15.0-17.0	Tested By	BBURR





AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **9/4/2008**

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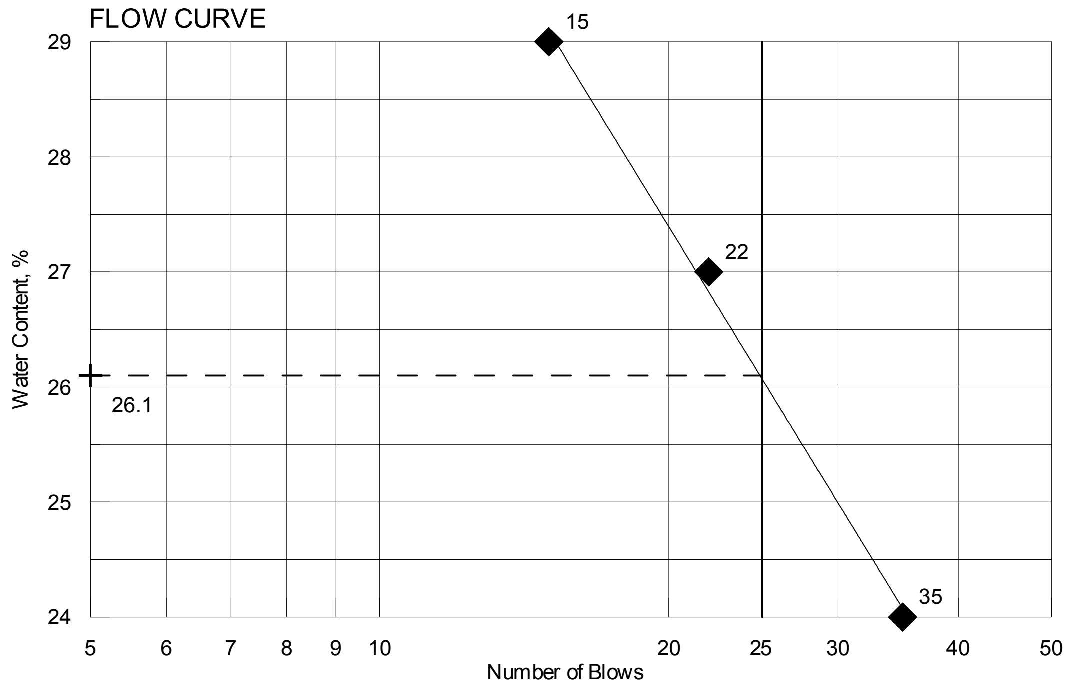
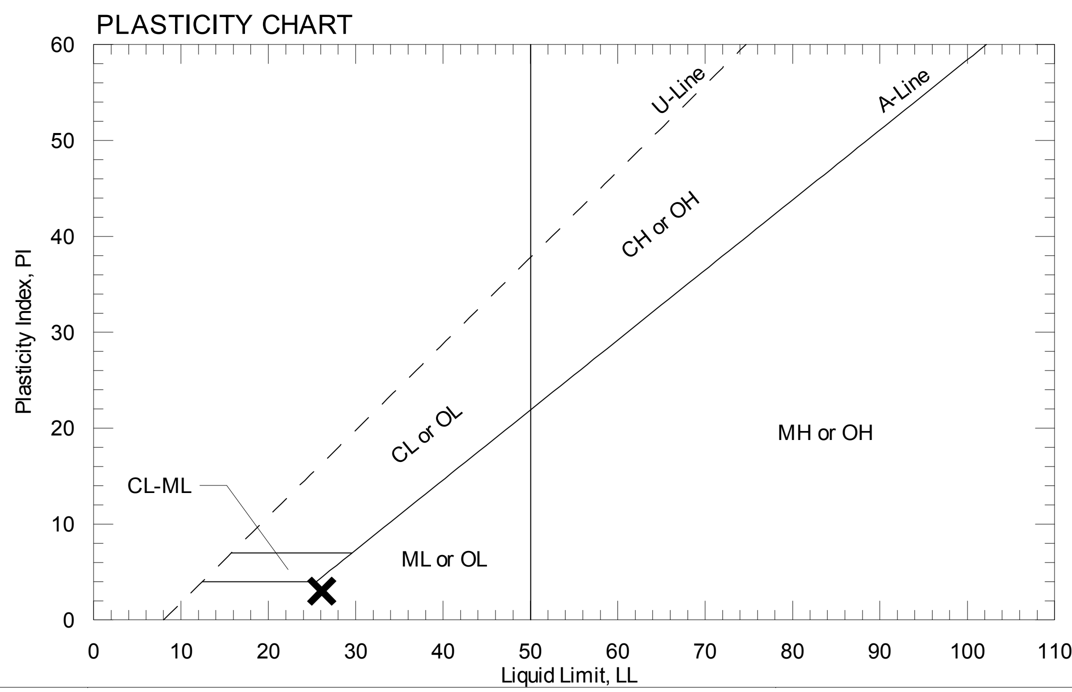
TOWN	Carmel	Reference No.	210023
PIN	015092.00	Water Content, %	34
Sampled	8/21/2008	Plastic Limit	23
Boring No./Sample No.	BB-CBS-101/1U	Liquid Limit	26
Station	4+69.2	Plasticity Index	3
Depth	20.0-22.0	Tested By	BBURR





AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **9/4/2008**

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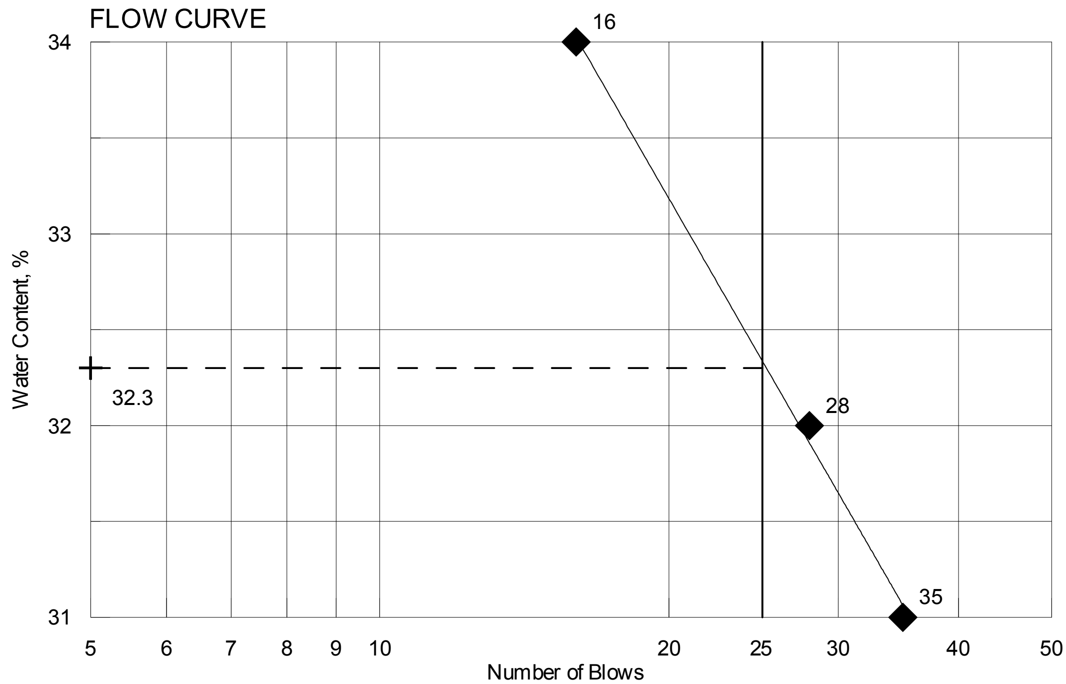
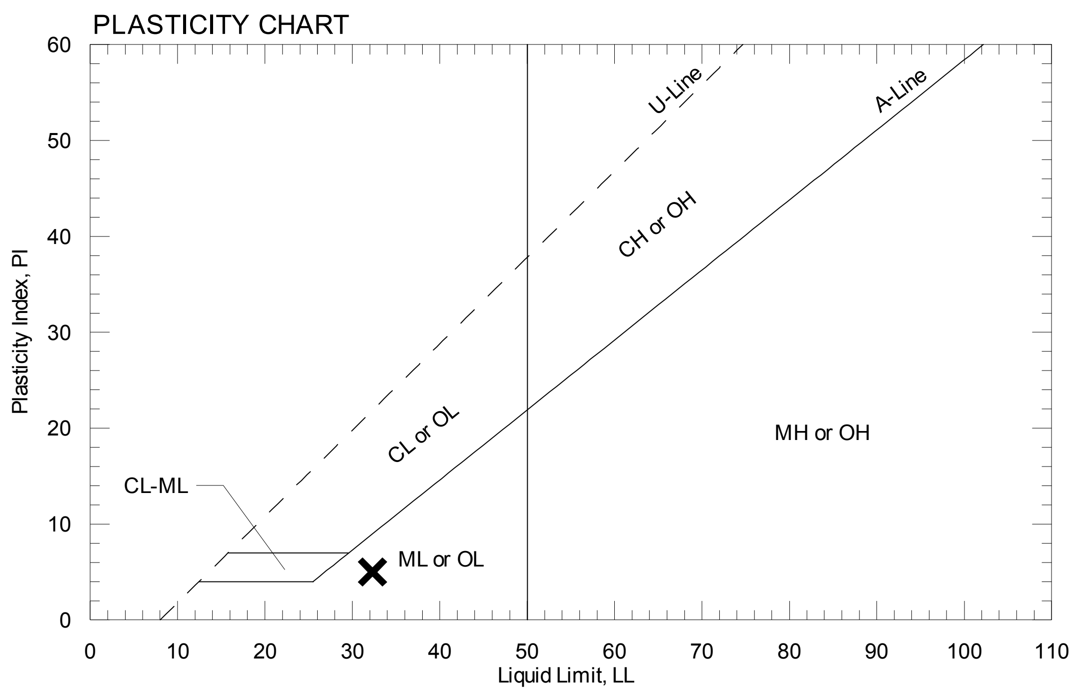
TOWN	Carmel	Reference No.	210027
PIN	015092.00	Water Content, %	30.3
Sampled	8/29/2008	Plastic Limit	27
Boring No./Sample No.	BB-CBS-102/2D	Liquid Limit	32
Station	5+25.1	Plasticity Index	5
Depth	5.0-7.0	Tested By	BBURR



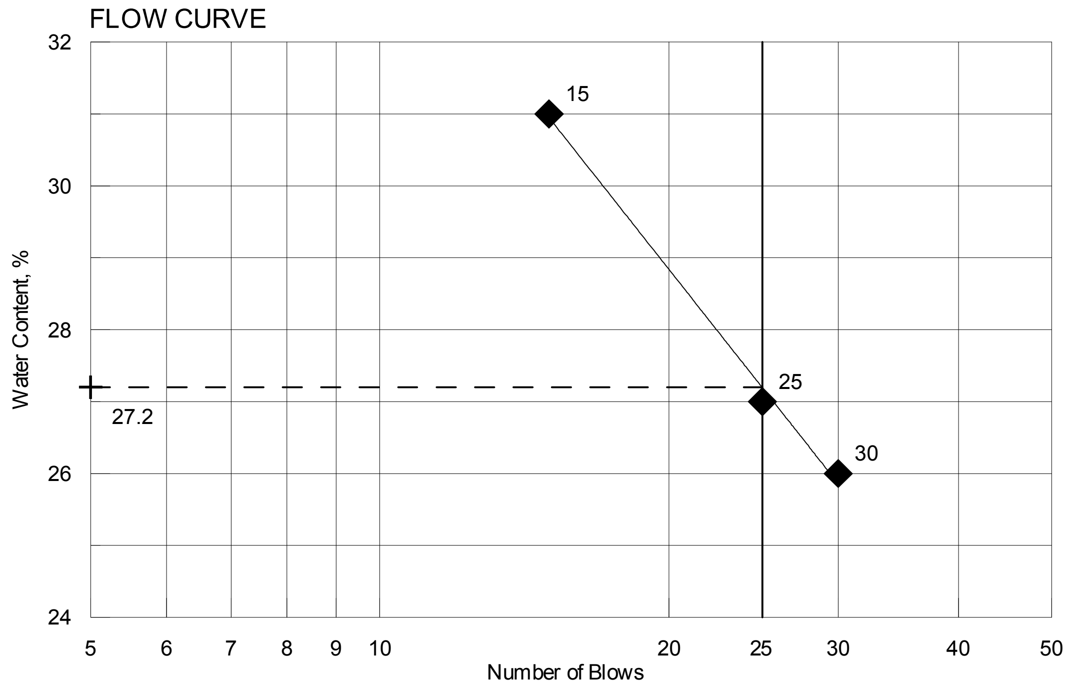


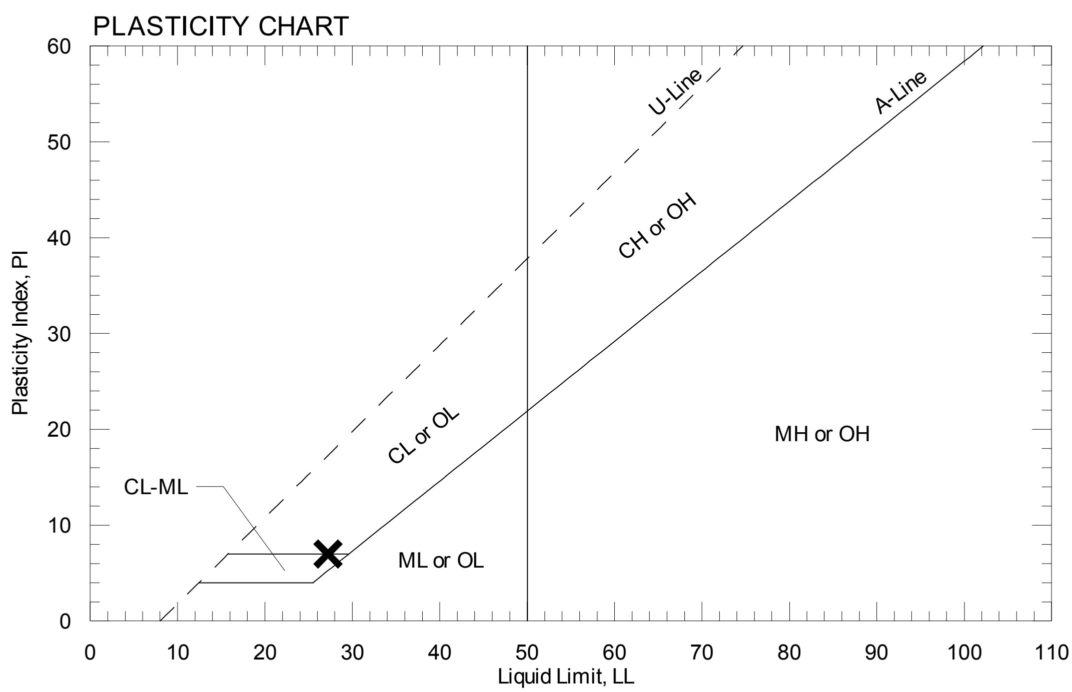
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **9/15/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Carmel	Reference No.	210028
PIN	015092.00	Water Content, %	21.8
Sampled	8/29/2008	Plastic Limit	20
Boring No./Sample No.	BB-CBS-102/3D	Liquid Limit	27
Station	5+25.1	Plasticity Index	7
Depth	10.0-12.0	Tested By	BBURR



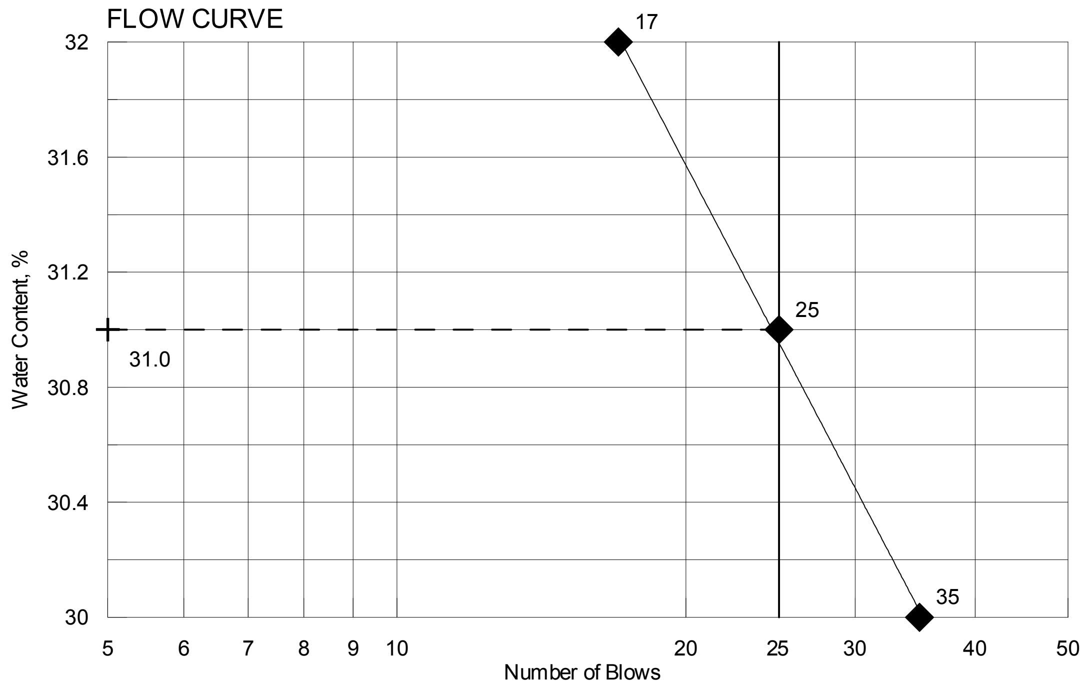


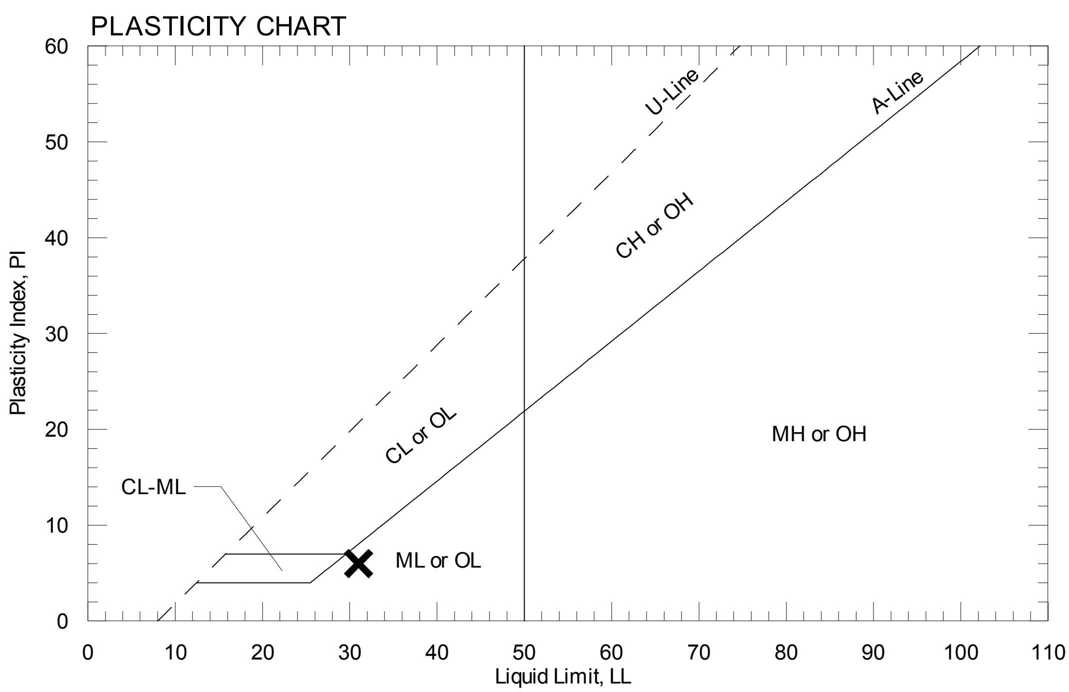
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **9/15/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Carmel	Reference No.	210029
PIN	015092.00	Water Content, %	36.1
Sampled	8/29/2008	Plastic Limit	25
Boring No./Sample No.	BB-CBS-102/5D	Liquid Limit	31
Station	5+25.1	Plasticity Index	6
Depth	18.0-20.0	Tested By	BBURR





A U T H O R I Z A T I O N   A N D   D I S T R I B U T I O N

Reported by: **FOGG, BRIAN** Date Reported: **9/15/2008**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
210023	**BB-CBS-101/1U**	**GEOTECHNICAL (UNDISTURBED)**	**8/21/2008**	**8/22/2008**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **4+69.2** Offset, ft: **7.6 RT** Dbfg, ft: **20.0-22.0**

PIN: **015092.00** Town: **Carmel** Sampler: **GIGUERE, ERVIN M**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**100.0**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**99.5**
[0.0247 mm]	**90.7**
[0.0162 mm]	**85.0**
[0.0095 mm]	**82.2**
[0.0072 mm]	**70.9**
[0.0054 mm]	**56.7**
[0.0029 mm]	**34.0**
[0.0013 mm]	**25.5**

Wash Method

Direct Shear (T 236)

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

Consolidation (T 216)

Trimmings, Water Content, % **33.0**

	Initial	Final		Void Ratio	% Strain
Water Content, %	**33.3**	**22.6**	Pmin		
Dry Density, lbs/ft³	**88.6**	**106.2**	Pp		
Void Ratio	**0.94**	**0.62**	Pmax	0.32 tsf	
Saturation, %	**97.3**	**100.1**	Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
26

Plastic Limit (T 90), %
23

Plasticity Index (T 90), %
3

Specific Gravity, Corrected to 20°C (T 100)
2.76

Loss on Ignition (T 267)
Loss, % H2O, %

Water Content (T 265), %
34.0

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-0.5	0.14	0.01	0.07	0	33.7	Super saturated light gray clay.
0.625-1.0	0.15	0.01	0.15	0.02	31.4	Alternating layers of light to dark gray clay.
1.0-1.5	0.15	0.02	0.17	0.03	32.6	Alternating layers of light to dark gray clay.

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **9/4/2008**

Paper Copy: Lab File; Project File; Geotech File

CONSOLIDATION TEST DATA

Project:
Boring No.: BB-CBS-101
Sample No.: 1U
Test No.: 210023

Location: CARMEL
Tested By: BRIAN FOGG
Test Date: 8/25/08
Sample Type: SHELBY TUBE

Project No.: 015092.00
Checked By:
Depth: 20-22 FT
Elevation: ---

Soil Description: GRAY CLAY
Remarks:

Specific Gravity: 2.76
Initial Void Ratio: 0.94
Final Void Ratio: 0.62

Liquid Limit: 0
Plastic Limit: 0
Plasticity Index: 0

Initial Height: 1.02 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	71	RING	RING	210
Wt. Container + Wet Soil, gm	238.2	415.66	403.31	203.48
Wt. Container + Dry Soil, gm	190.23	377.34	377.34	177.53
Wt. Container, gm	45.06	262.21	262.21	62.5
Wt. Dry Soil, gm	145.17	115.13	115.13	115.03
Water Content, %	33.04	33.29	22.56	22.56
Void Ratio	---	0.94	0.62	---
Degree of Saturation, %	---	97.28	100.13	---
Dry Unit Weight, pcf	---	88.614	106.24	---

CONSOLIDATION TEST DATA

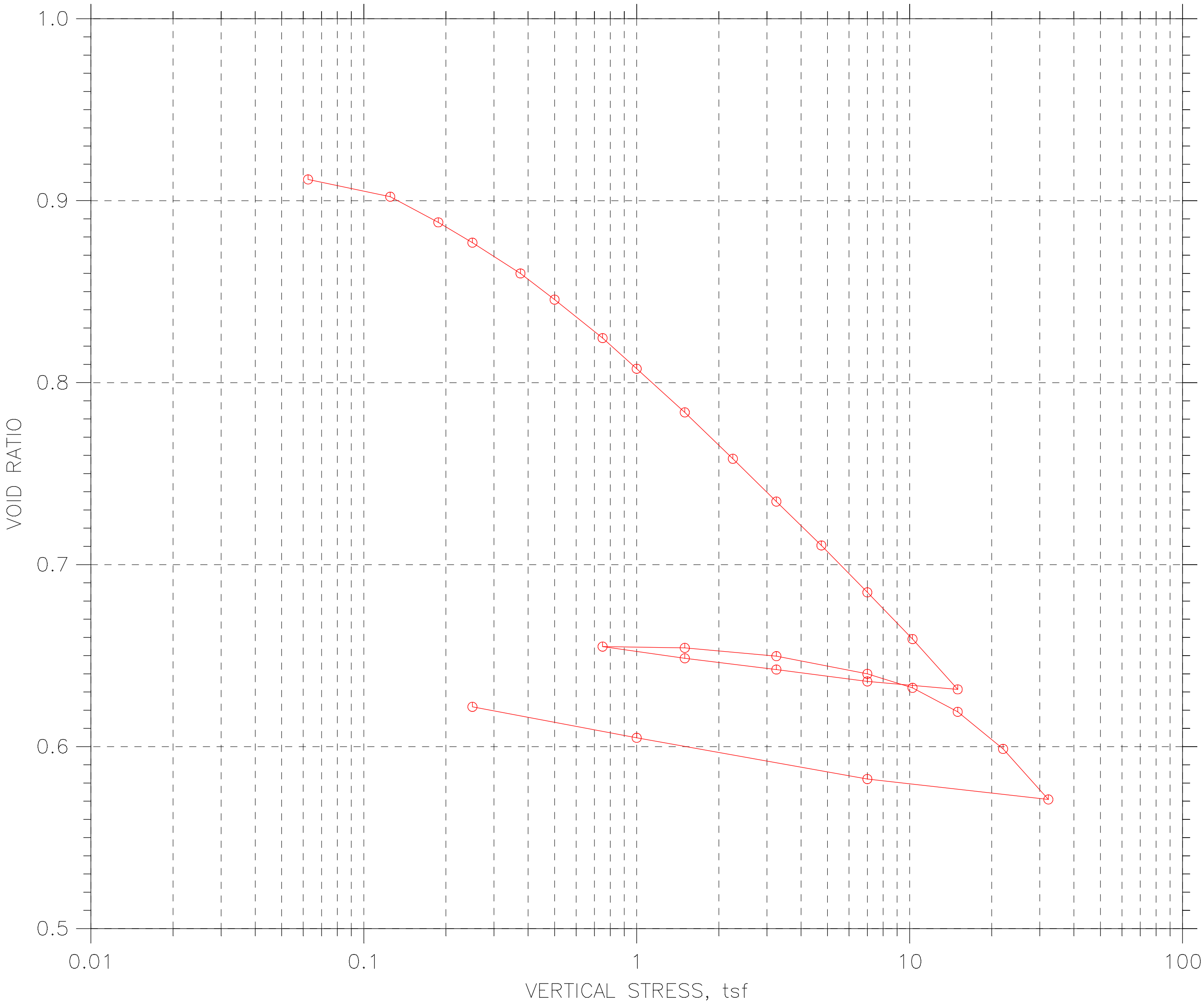
Project:	Location: CARMEL	Project No.: 015092.00
Boring No.: BB-CBS-101	Tested By: BRIAN FOGG	Checked By:
Sample No.: 1U	Test Date: 8/25/08	Depth: 20-22 FT
Test No.: 210023	Sample Type: SHELBY TUBE	Elevation: ---

Soil Description: GRAY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.01719	0.912	1.68	3.7	3.7	1.58e-006	1.58e-006	1.58e-006
2	0.125	0.02218	0.902	2.17	3.5	4.0	1.62e-006	1.42e-006	1.51e-006
3	0.188	0.02956	0.888	2.90	11.2	0.0	5.04e-007	0.00e+000	5.04e-007
4	0.25	0.03544	0.877	3.47	9.3	0.0	6.00e-007	0.00e+000	6.00e-007
5	0.375	0.0443	0.860	4.34	4.7	4.1	1.16e-006	1.34e-006	1.24e-006
6	0.5	0.05186	0.846	5.08	6.9	6.6	7.77e-007	8.14e-007	7.95e-007
7	0.75	0.06294	0.824	6.17	3.6	3.5	1.49e-006	1.50e-006	1.49e-006
8	1	0.07179	0.808	7.03	6.9	4.4	7.53e-007	1.18e-006	9.20e-007
9	1.5	0.08437	0.784	8.27	3.4	3.5	1.51e-006	1.44e-006	1.47e-006
10	2.25	0.09774	0.758	9.58	3.5	2.0	1.40e-006	2.43e-006	1.78e-006
11	3.25	0.1101	0.735	10.79	2.3	2.0	2.11e-006	2.41e-006	2.25e-006
12	4.75	0.1227	0.711	12.03	1.5	1.6	3.04e-006	2.90e-006	2.97e-006
13	7	0.1363	0.685	13.35	1.1	0.8	4.13e-006	6.03e-006	4.90e-006
14	10.3	0.1498	0.659	14.67	1.3	0.7	3.40e-006	6.29e-006	4.42e-006
15	15	0.1642	0.631	16.10	0.9	0.6	4.83e-006	7.27e-006	5.80e-006
16	7	0.162	0.636	15.87	0.0	0.0	0.00e+000	0.00e+000	0.00e+000
17	3.25	0.1585	0.642	15.53	0.1	0.0	4.35e-005	1.16e-004	6.33e-005
18	1.5	0.1553	0.649	15.22	0.2	0.1	2.60e-005	2.97e-005	2.77e-005
19	0.75	0.1519	0.655	14.89	0.7	0.3	5.96e-006	1.60e-005	8.69e-006
20	1.5	0.1523	0.654	14.92	0.0	0.0	1.18e-004	0.00e+000	1.18e-004
21	3.25	0.1547	0.650	15.16	0.2	0.0	2.04e-005	0.00e+000	2.04e-005
22	7	0.1598	0.640	15.66	0.1	0.1	4.05e-005	3.87e-005	3.96e-005
23	10.3	0.1638	0.632	16.05	0.2	0.1	2.57e-005	6.20e-005	3.64e-005
24	15	0.1707	0.619	16.73	0.5	0.1	7.65e-006	2.97e-005	1.22e-005
25	22	0.1814	0.599	17.78	0.7	0.2	5.91e-006	2.16e-005	9.28e-006
26	32.3	0.196	0.571	19.21	0.7	0.2	5.77e-006	2.62e-005	9.45e-006
27	7	0.1901	0.582	18.63	0.0	0.0	0.00e+000	0.00e+000	0.00e+000
28	1	0.1782	0.605	17.47	0.2	0.1	2.01e-005	3.09e-005	2.44e-005
29	0.25	0.1693	0.622	16.59	1.7	2.5	2.47e-006	1.64e-006	1.97e-006

CONSOLIDATION TEST DATA

SUMMARY REPORT



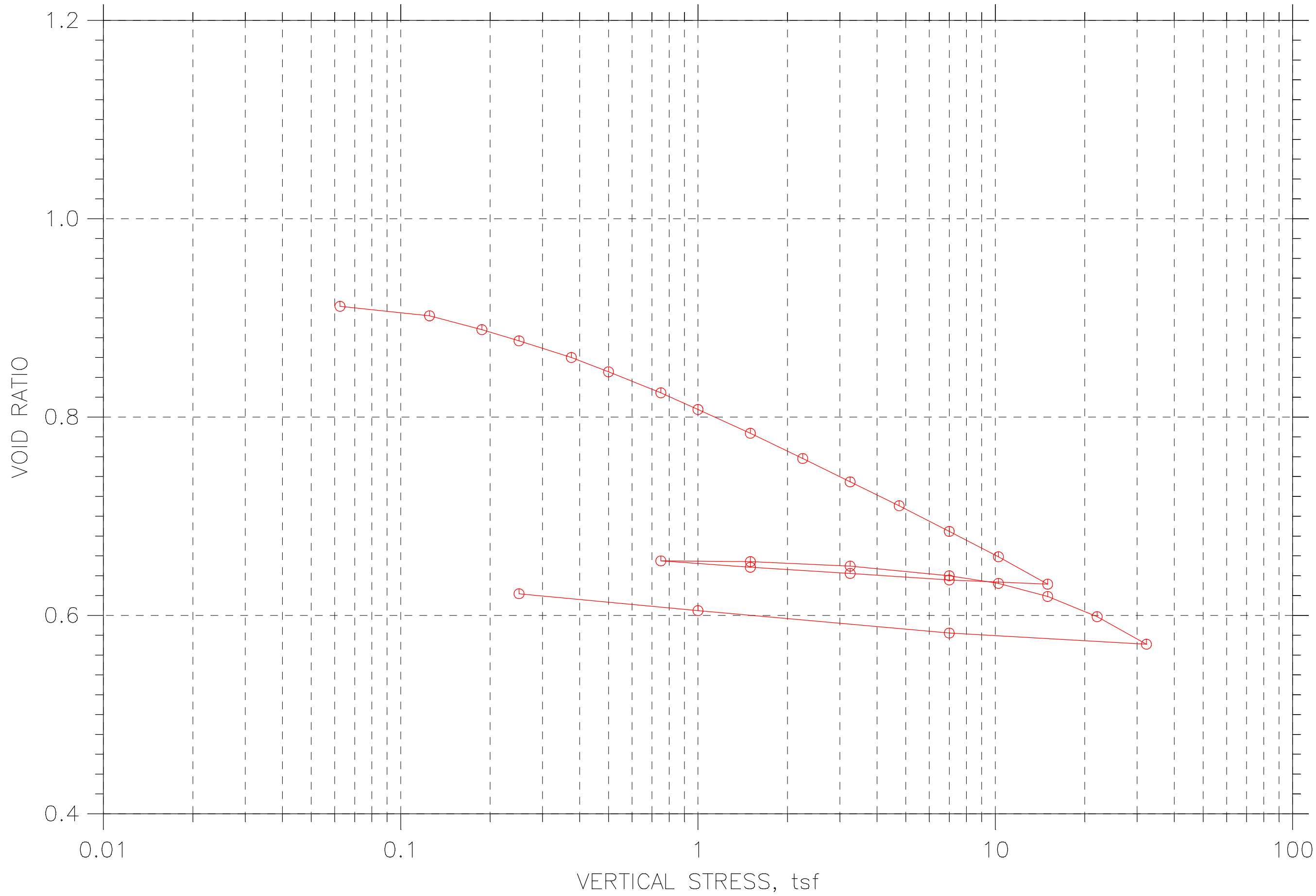
					Before Test	After Test
Overburden Pressure, tsf:				Water Content, %	33.29	22.56
Preconsolidation Pressure, tsf:				Dry Unit Weight, pcf	88.614	106.24
Compression Index:				Saturation, %	97.28	100.13
Diameter: 2.485 in		Height: 1.021 in		Void Ratio	0.94	0.62
LL: 0	PL: 0	PI: 0	GS: 2.76			

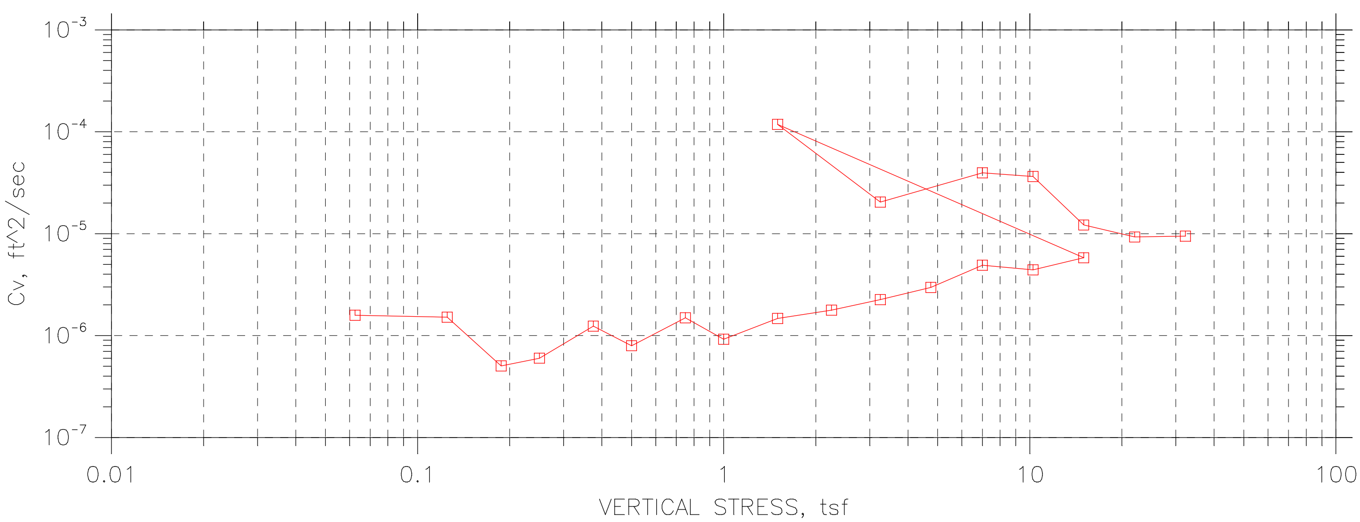
Project:	Location: CARMEL	Project No.: 015092.00
Boring No.: BB-CBS-101	Tested By: BRIAN FOGG	Checked By:
Sample No.: 1U	Test Date: 8/25/08	Depth: 20-22 FT
Test No.: 210023	Sample Type: SHELBY TUBE	Elevation: ---
Description: GRAY CLAY		
Remarks:		

Wed, 29-APR-2009 12:59:00

CONSOLIDATION TEST DATA

SUMMARY REPORT





Project:	Location: CARMEL	Project No.: 015092.00
Boring No.: BB-CBS-101	Tested By: BRIAN FOGG	Checked By:
Sample No.: 1U	Test Date: 8/25/08	Depth: 20-22 FT
Test No.: 210023	Sample Type: SHELBY TUBE	Elevation: ---
Description: GRAY CLAY		
Remarks:		

Wed, 29-APR-2009 12:58:27

Appendix C

Calculations

Bedrock Properties at the Site

RQD from bedrock cores
13% to 30% in BB-CBS-101
63% to 77% in BB-CBS-102

Rock Type: Metasedimentary Greenschist

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

uniaxial compressive strength = Co= 1400 to 21,000 psi - use **10,000 psi** for design AASHTO TABLE 4.4.8.2.B

Pile Properties

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

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

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

Nominal and Factored Structural Compressive Resistance of HP piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Use LRFD Equation 6.9.2.1-1

Normalized column slenderness factor, λ, in equation 6.9.4.1-1 is assumed to be zero since the unbraced length is zero.

$$F_y := 50 \cdot ksi$$

$$\lambda := 0$$

Nominal Axial Structural Resistance

From LRFD 6.9.4.1-1

$$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s \qquad P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Factored Axial Structural Resistance of single H pile

Resistance factor or H-pile in compression, no damage anticipated, LRFD 6.5.4.2

$\phi_c := 0.6$

Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 $P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$$

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Talbe 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example www.fhwa.gov/bridge/lrfd/us_dsp.htm

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Schist $q_{u\_1} := 10000 \cdot psi$

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 < 3.4

$dd = 1$ OK

Ksp

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

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

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

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 977 \\ 960 \\ 959 \\ 958 \end{pmatrix} \cdot ksf$$

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

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$\boxed{R_{p\_1} = \begin{pmatrix} 105 \\ 143 \\ 174 \\ 229 \end{pmatrix} \cdot kip}$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

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

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$\boxed{R_{r\_p1} = \begin{pmatrix} 47 \\ 64 \\ 78 \\ 103 \end{pmatrix} \cdot kip}$$

Drivability Analysis

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

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

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

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.

Table 10.5.5.2.3-1 page 10-38 gives resistance factor for dynamic test,

$\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium variablity. Only 1 to 2 piles will be tested, and the pile group would be nonredundant, i.e. less than five piles. Therefore reduce Φ by 20%.

$\phi_{dyn\_red} := 0.65 \cdot 0.8$ $\phi_{dyn\_red} = 0.52$

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:

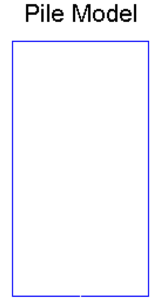
State of Maine Dept. Of Transportation — 23-Feb-2009
Carmel 12 x 53 fuel set reduced — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	19.25	0.12	1.1	4.88	12.81
200.0	27.88	0.40	2.7	5.89	12.07
300.0	35.94	1.78	4.4	6.59	13.00
350.0	39.46	2.08	5.4	6.99	13.79
400.0	42.85	2.49	6.6	7.44	14.71
450.0	45.86	2.90	8.1	7.87	15.57
500.0	48.73	3.63	10.1	8.33	16.48

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.060	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	15.50	in2

Pile Model



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

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 42.85}{45.86 - 42.85}\right)\cdot(450\cdot kip - 400\cdot kip) + 400\cdot kip$$

$$R_{ndr} = 435.7\cdot kip$$

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

$$\boxed{R_{fdr} = 227\cdot kip}$$

Pile Size is 14 x 74

The 14x 73 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 23-Feb-2009
14 x 73 fuel set 9 ft str 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	19.12	0.14	0.9	5.41	15.56
300.0	31.91	0.65	3.7	7.20	14.61
400.0	38.07	1.29	5.2	7.92	15.76
450.0	40.85	3.09	6.1	8.29	16.36
500.0	43.52	5.07	7.2	8.70	17.03
550.0	46.15	5.70	8.4	9.13	17.84

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.060	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	21.40	in2

Limiting driving stress to 45 ksi:

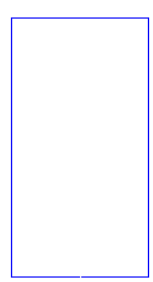
$$R_{ndr} := \left(\frac{45 - 43.52}{46.15 - 43.52}\right)\cdot(550\cdot kip - 500\cdot kip) + 500\cdot kip$$

$$R_{ndr} = 528\cdot kip$$

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

$$\boxed{R_{fdr} = 275\cdot kip}$$

Pile Model



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 — 23-Feb-2009
14 x 89 fuel setting 1 — 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	19.33	0.14	0.8	5.87	17.81
300.0	29.73	0.27	3.5	7.77	16.08
400.0	35.10	0.99	4.8	8.29	16.66
500.0	40.50	2.78	6.3	9.09	17.97
550.0	42.92	3.04	7.2	9.49	18.62
600.0	45.24	3.30	8.3	9.88	19.35
700.0	49.22	4.13	11.5	10.58	20.98

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.060	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	26.10	in2

Limiting driving stress to 45 ksi:

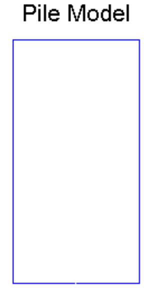
$$R_{ndr} := \left(\frac{45 - 42.92}{45.24 - 42.92}\right)\cdot(600\cdot kip - 550\cdot kip) + 550\cdot kip$$

$$R_{ndr} = 594.8\cdot kip$$

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

$$\boxed{R_{fdr} = 309\cdot kip}$$

Pile Model



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 — 23-Feb-2009
14 x 117 fuel setting 1 — 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	2.64	0.00	76.0	1.88	-0.27
300.0	26.73	0.20	1.9	6.01	26.44
500.0	36.70	0.66	3.6	6.81	26.23
700.0	45.14	1.79	5.5	7.58	28.33
750.0	47.13	1.94	6.0	7.82	29.17
800.0	49.05	2.81	6.6	8.05	30.01
850.0	50.59	3.20	7.4	8.22	30.54

Limiting blow count to 15 bpi:

$R_{ndr} := 700 \cdot kip$

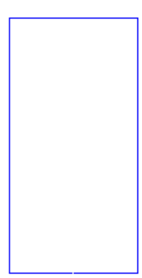
$R_{fdr} := R_{ndr} \cdot \phi_{dyn\_red}$

$\boxed{R_{fdr} = 364 \cdot kip}$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushioı	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.060	in
Skin Damping	0.100	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	34.40	in2

Pile Model



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

Calibration back to ASD - Structural Capacity

Geotechnical design capacity shall not exceed the pile structural allowable design load , based on allowable steel stress for integral piles, use 50 ksi steel, therefore 0.25Fy is the allowable stress.

For 50 ksi steel $F_y := 50 \cdot ksi$ $\sigma_a := \frac{F_y}{4}$ $Q_{all} := \sigma_a \cdot A_s$

$$Q_{all} = \begin{pmatrix} 194 \\ 268 \\ 326 \\ 430 \end{pmatrix} \cdot kip$$

50 ksi steel piles driven to 2.25 times the structural capacity

$$Q_{ult} := Q_{all} \cdot 2.25 \qquad Q_{ult} = \begin{pmatrix} 436 \\ 602 \\ 734 \\ 968 \end{pmatrix} \cdot kip$$

Assume the above equals the nominal geotechnical capacity

Factored resistance = 2.25 times the structural capacity times a resistance factor of 0.65

$$R_{factored} := Q_{ult} \cdot 0.65 \qquad R_{factored} = \begin{pmatrix} 283 \\ 391 \\ 477 \\ 629 \end{pmatrix} \cdot kip$$

Determination of Compression Index & Recompression Index for Clayey Silt Units, OCR and input parameters for Settlement Analyses

BB-CBS-101 Sample 1U

Determine insitu overburden stress

Sample depth $z := 20 \cdot ft$

Groundwater table $dw := 10 \cdot ft$

Initial void ratio $e_o := 0.912$

Effective overburden stress

4 feet of granular fill over silt fill over native, soft clayey SILT
Assume watertable at a depth of 10 feet

$\gamma_{sand} := 125 \cdot pcf$ $\gamma_{clay} := 120 \cdot pcf$

$$\sigma'_{vo} := 4 \cdot ft \cdot \gamma_{sand} + (6 \cdot ft) \cdot \gamma_{clay} + (10 \cdot ft) \cdot \left(\gamma_{clay} - 62.4 \cdot pcf\right)$$

$$\sigma'_{vo} = 1.796 \cdot ksf$$

Maximum past pressure from consolidation curve - A. Casagrande Construction (1936)

$$\sigma'_{vm} := 0.64 \cdot ksf$$

Overconsolidation ratio

$$OCR := \frac{\sigma'_{vm}}{\sigma'_{vo}} \qquad OCR = 0.356$$

This indicates the deposit is under-consolidated , however the lab curve indicates sample disturbance. **Use Shansep Method to backcalculate OCR in lower soft clay silt deposit**

Determine Compression Index (Cc) for lab consolidation curve

$e_1 := 0.735$ $e_2 := 0.631$ $p_1 := 3.25 \cdot 2 \cdot ksf$ $p_2 := 15 \cdot 2 \cdot ksf$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.157$$

Determine Compression Ratio $CC := \frac{C_c}{1 + e_o}$ $CC = 0.082$

Determine Recompression Index (Cr) from lab consolidation curve

$e_{r1} := 0.655 \quad e_{r2} := 0.636 \qquad p_{r1} := 0.75 \cdot 2 \cdot ksf \quad p_{r2} := 7 \cdot 2 \cdot ksf$

$$C_r := \frac{e_{r1} - e_{r2}}{\log\left(\frac{p_{r2}}{p_{r1}}\right)} \qquad C_r = 0.02$$

<u>Shansep Method to Backcalculate OCR - have vane tests in lower soft clayey silt unit only</u>

Range of undrained shear strengths in lower unit: 312, 469, 491, 339,424 psf

$$Su := \frac{312 + 469 + 491 + 339 + 424}{5} \cdot psf \qquad Su = 407 \cdot psf$$

Shansep Method - Reference Ladd (1991) for S and m variables

$S := 0.22$ for maine silt clays

$$m := 0.88 \cdot \left(1 - \frac{C_r}{C_c}\right) \qquad m = 0.77$$

$$OCR_{shan2} := \left(\frac{Su}{0.22 \cdot \sigma'_{vo}}\right)^{1.299} \qquad OCR_{shan2} = 1.039$$

OCR is 1.04 - Say the lower clayey silt is normally deposited

There are no vane shear tests in upper medium stiff to stiff clayey silt layer - assume slightly OC - say 1.5

<u>Recalcuate Cc and Cr for the 2 clayey silt units based on LL correlations</u>

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	

11 foot thick medium stiff silt:

$$LL := \begin{pmatrix} 32 \\ 37 \end{pmatrix} \qquad Cc := .009 \cdot (LL - 10) \qquad Cc = \begin{pmatrix} 0.198 \\ 0.243 \end{pmatrix}$$

Use Terzaghi Correlation

$$Cr := Cc \cdot 0.10 \qquad Cr = \begin{pmatrix} 0.02 \\ 0.024 \end{pmatrix}$$

13 foot thick soft clay-silt:

$$LL := \begin{pmatrix} 28 \\ 26 \\ 31 \end{pmatrix} \qquad Cc := .009 \cdot (LL - 10) \qquad Cc = \begin{pmatrix} 0.162 \\ 0.144 \\ 0.189 \end{pmatrix}$$

Use Terzaghi Correlation

$$Cr := Cc \cdot 0.10 \qquad Cr = \begin{pmatrix} 0.016 \\ 0.014 \\ 0.019 \end{pmatrix}$$

Input Parameters for Settlement Analyses:

11 foot thick medium stiff to stiff silt layer is over consolidated, OCR = 1.5, Cc = 0.20 Cr =0.02

13 foot thick soft silt deposit is normally consolidated, Cc= 0.16, Cr =0.016

FoSSA -- Foundation Stress & Settlement Analysis
Project Date/Time: Wed Apr 29 10:05:25 2009

Carmel Norton Bridge
C:\Program Files\ADAMA\FoSSA(1.0)\15092 Carmel 03.F2S

Carmel Norton Bridge

PROJECT IDENTIFICATION

Title:	Carmel Norton Bridge
Project Number:	-
Client:	
Designer:	Laura Krusinski
Station Number:	

Description:

Company's Information:

Name:	MaineDOT
Street:	
	,
Telephone #:	
Fax #:	
E-Mail:	

Original file path and name: C:\Program Files\ADAMA\FoSSA(1.0)\15092 Carmel 03.F2S
Original date and time of creating this file: Mon Apr 27 14:13:04 2009

GEOMETRY: Analysis of a 2D geometry

INPUT DATA – FOUNDATION LAYERS – 4 layers

	Wet Unit Weight, γ [lb/ft³]	Poisson's Ratio μ	Description of Soil
1	125.00	0.25	Existing Embankment Fills
2	120.00	0.30	OC medium stiff silt - no vanes to backcalc OCR
3	120.00	0.30	NC Clayey Silt - 13 feet thick - soft
4	165.00	0.10	Bedrock

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

Layer #	Undergoing 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	1.50	0.20	0.02	0.91	0.2153	Top
3	Yes	1.00	0.16	0.02	0.91	0.2153	Bottom
4	No	N/A	N/A	N/A	N/A	N/A	N/A

ULTIMATE SETTLEMENT, Sc

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z* [ft.]
1	300.00	0.00	330.00	0.03	329.97
2	310.00	0.00	330.00	0.13	329.87
3	320.00	0.00	334.00	0.06	333.94
4	330.00	0.00	336.00	0.04	335.96
5	340.00	0.00	336.00	0.04	335.96
6	350.00	0.00	333.87	0.05	333.82
7	360.00	0.00	330.00	0.05	329.95
8	370.00	0.00	330.00	0.01	329.99

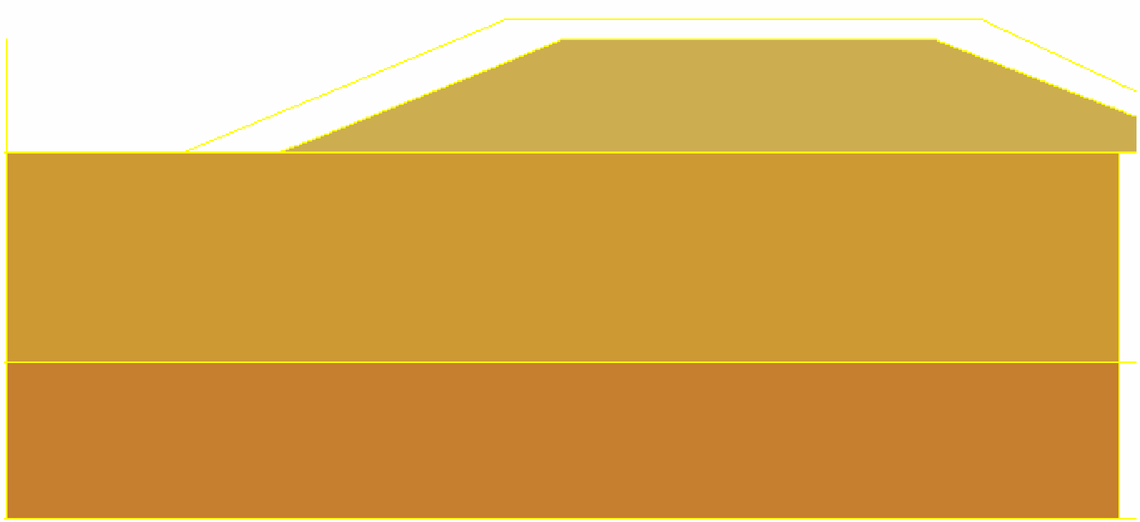
*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

Maximum settlement of 0.13 ft or 1.56 inches near the mid to lower fill extension areas due to the 1.0 foot raise in grade and slope widening.

FoSSA -- Foundation Stress & Settlement Analysis — Carmel Norton Bridge

Present Date/Time: Wed Apr 29 10:06:09 2009

DRAWING OF SPECIFIED GEOMETRY



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

From Design Freezing Index Map:

Carmel, Maine

DFI = 1760 degree-days

Case 1 - Soils are granular fill soils with a water content of 10%

Interpolate between frost depth of 87.5 inches at 1700 DFI and 90.1 inches at 1800 DFI

Depth of Frost Penetration =

$$d := \frac{90.1 - 87.5}{100} \cdot 60 \cdot in + 87.5 \cdot in \qquad d = 7.422 \cdot ft$$

Method 2 - ModBerg Software

Carmel lies on the same Design Freezing Index contour as Orono, BDG Fig. 5-1

Case 1 - coarse-grained fill soils with water content of 10%

--- ModBerg Results ---

Project Location: Orono, Maine

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

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	72.7	10.0	120.0	26	32	1.7	1.5	1,728

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

Total Depth of Frost Penetration = 6.05 ft = 72.7 in.

Recommendation: use 6.0 feet for for design for foundations on granular fill, not founded on bedrock

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04419
Zip Code Latitude = 44.808800
Zip Code Longitude = -068.947300
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.069	PGA - Site Class B
0.2	0.148	Ss - Site Class B
1.0	0.044	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04419
Zip Code Latitude = 44.808800
Zip Code Longitude = -068.947300
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.172	As - Site Class E
0.2	0.370	SDs - Site Class E
1.0	0.156	SD1 - Site Class E

Abutment and Wingwall Passive Earth Pressure

Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

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

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

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

Input parameters

Angle of fill slope to the horizontal

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

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

$$\theta := 90 \cdot \text{deg}$$

Friction angle between fill and wall, δ :

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

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

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

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

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

$$\delta = 21.333 \cdot \text{deg}$$

Passive Earth Pressure - Rankine Theory

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

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

$$\boxed{K_{pslope} = 3.255}$$

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

Passive Earth Pressure - Coulomb Theory

Interface friction is considered in Coulomb.

For a smooth vertical wall with horizontal backfill $\delta = \beta = 0$ and $\theta = 90$ degrees (refer: Bowles, 5th edition, pag 596

$\theta =$ Angle of back face of wall to the horizontal

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

For precast 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)

$$\delta := 19.5 \cdot deg$$

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

$$\boxed{K_{pc} = 6.73}$$