MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MCRR CROSSING BRIDGE
ROUTE 2 AND 100 OVER MAINE CENTRAL RAILROAD
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 Maine Central Railroad (MCRR) Bridge which carries State Route 2 and 100 over Pan Am Railroad (formerly Maine Central and Boston Railroad), in Carmel, Maine. The bridge is a six span structure with a thru girder main span, and a total length of 253 feet. The proposed replacement bridge will be a 58-foot span, simply supported, precast void slab superstructure on full height, cantilever-type abutments and wingwalls. Abutments and wingwalls will be supported by spread footings founded directly on bedrock or seal concrete founded on bedrock. The following design recommendations are discussed in detail in this report:

Cantilever Abutments and Wingwalls - Abutments and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. They shall be designed for all relevant strength and service limit states in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, with 2008 and 2009 interims (herein referred to as LRFD).

The design of project abutments founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. A maximum frictional coefficient of 0.70 at the bedrock-concrete interface should be assumed.

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

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

The proposed abutments are within a distance of 50 feet to the centerline of the railroad track, and therefore should be designed for railway vehicle impact forces or protected by a crashworthy barrier.

Bearing Resistance – The factored bearing pressure at the strength limit state for spread footings on sound bedrock should not exceed the factored bearing resistance of 15 kips per square foot (ksf). Based on presumptive bearing resistance values, a factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

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

Approach Embankment Design Considerations - New approach fills with heights up to 26 and 23 feet are proposed at the approaches to Abutment No. 1 and Abutment No. 2, respectively. It is recommended that the approach embankment subgrade, which consists of loose fills of variable thickness, be grubbed and then compacted with a minimum of 10 passes of a large, smooth drum vibratory roller with a minimum weight of 10,000 lbs. Water should be added or removed, as necessary, in order to obtain sufficient compaction.

Settlement - The grades of existing bridge approaches and side slopes will be not raised, however, the ground below the 3 spans of the current bridge's south approach and the 2 spans of the north bridge approach will be filled in. We anticipate approach embankment settlement on the order of 1.0 inch due to compression of the foundation soils if the loose fill subgrade is not compacted. This settlement is due largely to the fill layer. If the loose fill subgrade is well compacted, approximately 0.5 inch of embankment settlement can be expected. Most of this settlement will occur during and immediately after construction of the embankments. Post-construction settlement will be minimal.

Any settlement of bridge abutments will be due to elastic compression of the bedrock and consolidation settlement of silt-infilled seams in the bedrock and is anticipated to be less than 0.5 inch.

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

Seismic Design Considerations – Seismic analysis is not required for single-span bridges, regardless of seismic zone, however superstructure connections and bridge seat dimensions shall be designed in accordance with LRFD requirements.

Construction Considerations – Temporary lateral earth support systems will be required to shore up the railroad track beds and permit abutment and wingwall construction. Preparation of the bedrock subgrade for abutment and wingwall footings may require excavation of bedrock to create level benches or flatten bedrock surfaces with slopes steeper than 4 horizontal to 1 vertical (4H:1V). All loose bedrock and soil debris should be removed from bearing surfaces and the final bedrock surface washed with high-pressure water and air before concrete is placed for the abutment and wingwall foundations.

Excavation of bedrock may be conducted using conventional equipment, but may require drilling and blasting methods. Blasting should be conducted in accordance with Section

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

The marine silts and glacial till encountered in the borings are considered moisture-sensitive due to the high fines content. The soil is susceptible to disturbance and rutting as a result of exposure to water or construction traffic.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of MCRR Crossing Bridge which carries State Route 2 and 100 over Pan Am Railroad, in Carmel, Maine. This report presents the soils information obtained at the site during the subsurface investigations, foundation recommendations and geotechnical design parameters for bridge replacement.

MCRR Crossing Bridge was built in the 1930 and is a 253-foot, 6-span, steel girder bridge. Three (3) of the six (6) spans have through-girder floor beams. This bridge type is considered "fracture critical" by FHWA and the Maine Department of Transportation (MaineDOT). The approach spans are simply supported steel girders. The middle span is 84 feet and spans the east bound railroad track and the former west bound railroad track. The superstructure is supported on spill thru, concrete gravity abutments on spread footings and five intermediary steel pier bents supported on pedestal footings. The pier bent pedestals consist of spread footings bearing on bedrock or soil. The existing abutments may be founded on either native soils or portions of old split stone abutments. In 1947 the addition of a sidewalk to the bridge required lengthening the abutments with gravity-shaped stub abutments.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate abutment backwall and bridge seat distress in the form of concrete deterioration, cracking and scaling. Year 2007 MaineDOT Bridge Maintenance inspection reports assign the substructures a condition rating of 5 – fair, and indicate a Bridge Sufficiency Rating of 48.3.

The May 2008 MaineDOT Scope Review Team (SRT) Final Report considered the "fracture critical" rating of the three (3) bridge spans that have through-girder floor beams, and recommended total bridge replacement.

Four preliminary foundation alternatives were provided by the geotechnical team member in an internal Geotechnical Design Memorandum, dated December 31, 2008. Subsequent engineering assessments by the MaineDOT Bridge Program identified the preferred bridge structure alternative to be a 56-foot, single-span precast prestressed voided slab superstructure, with foundations consisting of cantilever-type abutments on spread footings founded directly on bedrock or seal concrete founded on bedrock. The superstructure curb-to-curb width will be increased from 24 feet to 34 feet and will be centered on the existing alignment.

2.0 GEOLOGIC SETTING

MCRR Crossing Bridge on State Route 2 and 100 in Carmel, Maine crosses the Pan Am Railroad as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey (MGS) Surficial Geology of Stetson Quadrangle, Maine, Open-file No. 86-39 (1986) indicates that the MCRR Crossing Bridge in Carmel is at a contact of the glacial marine deposits and glacial till.

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

The Bedrock Geologic Map of Maine, MGS, (1985), cite the bedrock at the MCRR Bridge site as the Vassalboro Formation and consists of metasedimentary, calcareous sandstone, interbedded sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling five test borings. All borings were terminated with bedrock cores. Test borings BB-CRR-102 and BB-CRR-103 were drilled at the proposed locations of Abutment No. 1 and Abutment No. 2. Test boring BB-CRR-101 was drilled where a 26-foot high approach embankment to Abutment No. 1 is proposed. These borings were drilled on June 9 and 10, 2008 using the MaineDOT drill rig. Two additional borings, BB-CRR-201 and BB-CRR-202, were drilled to determine approximate bedrock elevations at the south facing wingwalls of Abutment No. 1 and No. 2. Those borings were drilled on August 27, 2009 using the MaineDOT drill rig. The boring locations are shown on Sheet 2 - Boring Location Plan and Sheet 3 - Interpretive Subsurface Profile, found at the end of this report.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance.

The MaineDOT drill rig is newly equipped with a Central Mine Equipment (CME) automatic hammer. The hammer was calibrated by MaineDOT in August of 2007 and February of 2009 and was found to deliver approximately 30 percent, and subsequently in 2009, 40 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 average energy transfer factors of 0.77 or 0.84 to the raw field N-values. These hammer efficiency factors, 0.77 and 0.84, and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the five (5) borings using an NQ-2 core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team Member or a New England Transportation Technical Certification Program (NETTCP) Certified Subsurface Inspector 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 4 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

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

Laboratory testing consisted of three (3) standard grain size analyses, two (2) grain size analyses with hydrometer, five (5) natural water content tests, and one (1) Atterberg Limits test. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4- Boring Logs.

5.0 SUBSURFACE CONDITIONS

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

5.1 Fill

A layer of fill was encountered in all of the borings. The encountered fill layer is approximately 1.6 to 4 feet thick. The fill deposit generally consisted of black, blackish brown, dry to moist, fine to coarse SAND, with some to trace silt, to silty SAND, with some to trace gravel, and a trace of clay, cinders, slag, brick fragments and organics.

Corrected SPT N-values in fill ranged from 4 to 18 blows per foot (bpf) indicating that the fill is very loose to medium dense in consistency.

One grain size analysis resulted in the soil being classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System. The measured water content of the sample tested was approximately 8 percent.

5.2 Glacial Marine Silt

A shallow and discontinuous layer of glacial marine silt was encountered in boring BB-CRR-101. The encountered thickness was approximately 1.2 feet thick at the boring location. The glacial silt was weathered and generally consisted of yellowish brown, moist, SILT, some clay, trace sand and fine gravel, with layered structure.

One corrected SPT N-value in silt unit was 23 bpf, indicating a soil of very stiff consistency. The unit is considered heavily preconsolidated.

Laboratory testing of one sample of the marine silt deposit indicates the USCS soil classification is CL-ML. The AASHTO classification for the sample tested is A-4. One Atterberg Limits test with natural water content was conducted. The measured water content of the tested sample was approximately 22 percent. The sample was nonplastic.

5.3 Weathered Glacial Till

A relatively shallow layer of weathered glacial till was encountered in all except one boring. The encountered thickness was approximately 3.1 to 4.4 feet thick at the boring locations. The weathered glacial till unit has a high portion of fine grained soil and pockets of weathered bedrock. The weathered glacial till generally consisted of brown to yellowish brown, wet, silt or sand or silty sand, some silt, with varying lesser percentages of clay, gravel and weathered bedrock fragments. The unit is nonplastic, weathered and compact.

Corrected SPT N-values in weathered glacial till unit ranged from 17 to >50 bpf, indicating a soil of medium to very dense consistency.

Laboratory testing of three (3) samples of the glacial till deposit indicates the USCS soil classifications are SC-SM and SM. The AASHTO classifications for the samples tested are A-4 and A-1-b. The measured water contents of the tested samples were approximately 8 to 15 percent.

5.4 Bedrock

Bedrock at the site was encountered and cored at depths ranging from approximately 5.0 feet below ground surface (bgs) and approximate Elevation 150.50 feet in boring BB-CRR-202 to a depth of approximately 9.3 feet bgs and approximate Elevation 145.2 feet in boring BB-CRR-101. In borings BB-CRR-201 and BB-CRR-202 a 0.3 to 1.0 foot layer of weathered bedrock was encountered above more competent bedrock.

The bedrock at the site is identified as grey and green-grey, fine grained, calcareous, metamorphic, greenschist, moderately hard to hard, moderately weathered to very slightly weathered, with irregular foliation, close bedding, surfaces tight, stained with occasional open seams with silt infilling. The RQD of the bedrock was determined to range from 33 to 94 percent, correlating to a rock quality of poor to excellent.

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

Proposed Substructure/ Feature	Boring	Station	Offset	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
South approach embankment	BB-CRR-101	13+46.1	0.81 Lt	9.3	145.2
Abutment No. 1	BB-CRR-102	13+96.1	5.7 Rt.	6.6	147.9
Abutment No. 1 south wingwall	BB-CRR-201	13+95	30.0 Rt.	6.3	148.2
Abutment No. 2	BB-CRR-103	14+51.5	5.4 Lt.	5.1	149.4
Abutment No. 2 south wingwall	BB-CRR-202	14+70	28.0 Rt.	5.0	150.5

Table 1. Summary of Approximate Bedrock Elevations

5.5 Groundwater

The groundwater levels observed in three borings ranged from approximately 3 to 4 feet bgs. Groundwater levels will fluctuate with seasonal changes, runoff, and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

Prior to the development of the Preliminary Design Report (PDR) for MCRR Crossing Bridge, several foundation alternatives were provided to the designer in an internal geotechnical design memorandum dated December 31, 2009. Four (4) foundation alternatives were identified for the replacement substructures in the Design Memorandum:

- Full-height, cantilever-type, reinforced concrete abutments on spread footings constructed directly on bedrock or seal concrete on bedrock.
- Pile-supported integral or stub abutments on spread footings, supported laterally by 1.75H:1V protected slopes.

- Pile-supported integral abutments supported laterally by approach fill volumes retained by Mechanically Stabilized Earth (MSE) walls or Prefabricated Concrete Modular Gravity (PCMG) walls. The piles are driven to bedrock in open sleeves after construction of the walls. The risk associated with the pile alternative is not achieving a fixed condition at the pile tips due to the proximity of bedrock to the ground surface, and pile lateral capacity will be reduced to that provided by stone fill placed by free fall method in the sleeves.
- Conventional stub abutments on spread footings constructed on MSE wall-wrapped approach embankments.

Our initial assessment indicated the most effective foundation types for this site to be (1) cantilever-type abutments on spread footings founded directly on bedrock, (2) pile-supported integral or stub abutments supported laterally by 1.75H:1V slopes and (3) stub abutments on spread footings constructed on MSE wall-wrapped approach embankments.

Subsequently, cantilever-type abutments on spread footing founded directly on bedrock or on seal concrete on bedrock was selected by the Designer and is the recommended foundation type in the PDR. Design recommendations for this selected foundation alternative are discussed in detail in Section 7.0 - Geotechnical Design Recommendations.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 General - Spread Footings on Bedrock

Bedrock was encountered at depths approximately 5 to 7 feet below the proposed Abutment No. 1 and Abutment No. 2 locations and the south facing abutment wingwalls. It is therefore considered feasible that spread footings or seals, if required, could be practically and economically constructed to bear on bedrock within shallow excavations requiring temporary soil support systems.

The borings indicate that suitable bedrock with a minimum RQD of approximately 30 percent will be encountered at the bedrock surface, however, the bedrock surface shall be cleared of all loose bedrock and loose, decomposed bedrock. Based on borings conducted at the site and top of bedrock elevation encountered in those borings, the bottom of footing elevations are estimated to be approximately Elev. 147.9 feet at Abutment No. 1 and approximately Elev. 149.4 feet at Abutment No. 2.

7.2 Abutment and Wingwall Design

Abutments and extension wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength and service limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure.

Failure by sliding shall be investigated. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

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

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement, and overall stability.

Cantilever-type abutments and wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever-type abutments and wingwalls. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

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

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

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

Abutment No. 1 and Abutment No. 2 are within a distance of 50 feet to the centerline of the railroad track. Per LRFD Article 3.6.5.2 the abutments should be designed for railway vehicle impact forces or protected by a crashworthy barrier as described in LRFD Article 3.6.5.1.

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

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

Slopes above the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 Bearing Resistance

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 15 ksf. This assumes a bearing resistance factor, $\varphi_{b,}$ for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. See Appendix C – Calculations, for supporting documentation.

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

7.4 Approach Embankment Design Considerations

It is recommended that all stumps, roots, organics, vegetation or other objectionable material be removed from the approach embankment plan area within 100 feet of the abutment locations. To assess the suitability of the in-situ fill unit as embankment subgrade, stability analyses to determine factors of safety against global failure of the new approach embankments were conducted. New approach fills with maximum heights on the order of 26 and 23 feet are proposed at the approaches to Abutments No. 1 and No. 2, respectively. The software used to conduct the stability analyses was GeoStudio Slope/W 6.20 which applied the Bishop method in the analyses. A minimum factor of safety of 1.3 is required in accordance with FHWA Soils and Foundations Manual, 2006.

Results of the slope stability analyses indicate that compaction of the 2 to 4-foot thick fill unit will provide a minimum factor of safety of 1.5 against slope instability. Supporting calculations are provided in Appendix C – Calculations.

It is recommended that the loose fill subgrade in the bridge approach embankment plan areas be grubbed and then compacted with a minimum of 10 passes of a large, smooth drum vibratory roller with a minimum weight of 10,000 lbs. Water should be added or removed, as necessary, in order to obtain sufficient compaction.

The encountered fill layer in the borings was approximately 2 to 4 feet thick, but the thickness of the unit will be variable. Grubbing and removal of the unsuitable material may result in the exposure of naturally deposited soils consisting of medium stiff to very stiff, glacial marine silt and subunits of glacial till with high fines content. The marine silt and glacial till soils at the subgrade will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. The Contractor should protect the subgrade from exposure to water and traffic and remove and replace with compacted gravel borrow if disturbance and rutting occur.

7.5 Settlement

Replacing the existing 253 foot span bridge with single, 60-foot span bridge will require the construction of new approach fill embankments up to 100 feet long at both bridge approaches. Earth fill embankment with heights on the order of 25 feet will be constructed adjacent to the railroad tracks. Placing 25 feet of earth fill over approximately 4 feet of fill, 1.2 feet of stiff glacial marine silt and 4 feet of fine-grained glacial till soils will cause moderate consolidation and densification of the underlying soils and subsequent settlement of the embankments. We anticipate approach embankment settlement on the order of 1.0 inch due to compression of the foundation soils if the subgrade fill soils are not compacted. This settlement is due largely to the in-situ fill subgrade. If the loose fill subgrade is compacted, approximately 0.5 inch of embankment settlement can be expected. Most of this settlement will occur during and immediately after construction of the embankments. Post-construction settlement will be minimal.

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

7.6 Frost Protection

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

Any foundations placed on granular fill should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Carmel has a design freezing index of approximately 1750 F-degree days. An assumed water content of 15% was used for granular soils above the water table. These components correlate to a frost depth of 6.8 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Carmel was assigned a design freezing index of approximately 1588 F-degree days. An assumed water content of 15% was used for granular soils above the water table. These components correlate to a frost depth of 6.6 feet. We recommend that foundations constructed within granular fill soils be founded a minimum of 6.5 feet below finished exterior grade for frost protection.

7.7 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. MCRR Crossing 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 designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak ground acceleration coefficient (PGA) = 0.069g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.148g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.044g
- Site Class B (rock with a average shear wave velocity for the upper 100 ft of the soil profile <5000 ft./sec.)
- Seismic Zone 1, based on a S_{D1} < 0.15g

7.8 Construction Considerations

Construction activities will include earth support systems construction to support the railroad track beds during construction of abutments and wingwalls. Construction activities will also include common earth and rock excavation.

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

It is recommended that the approach embankment plan area within 100 feet of the abutment locations be grubbed and then compacted with a minimum of 10 passes of a large vibratory-type smooth wheel roller or a large pneumatic tired roller. Water should be added or removed, as necessary, in order to obtain sufficient compaction.

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

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

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

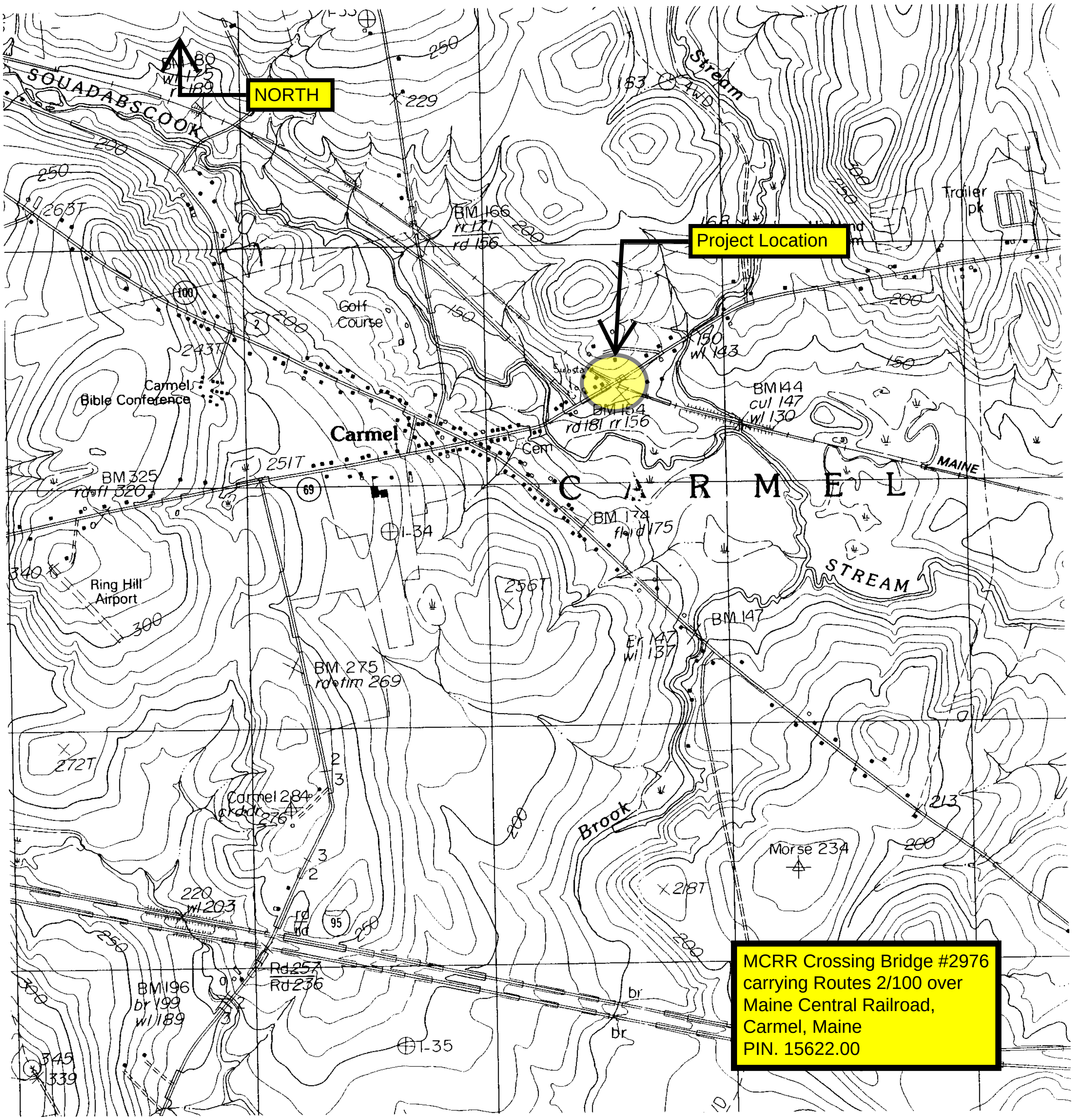
It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

7.0 CLOSURE

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

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

Sheets



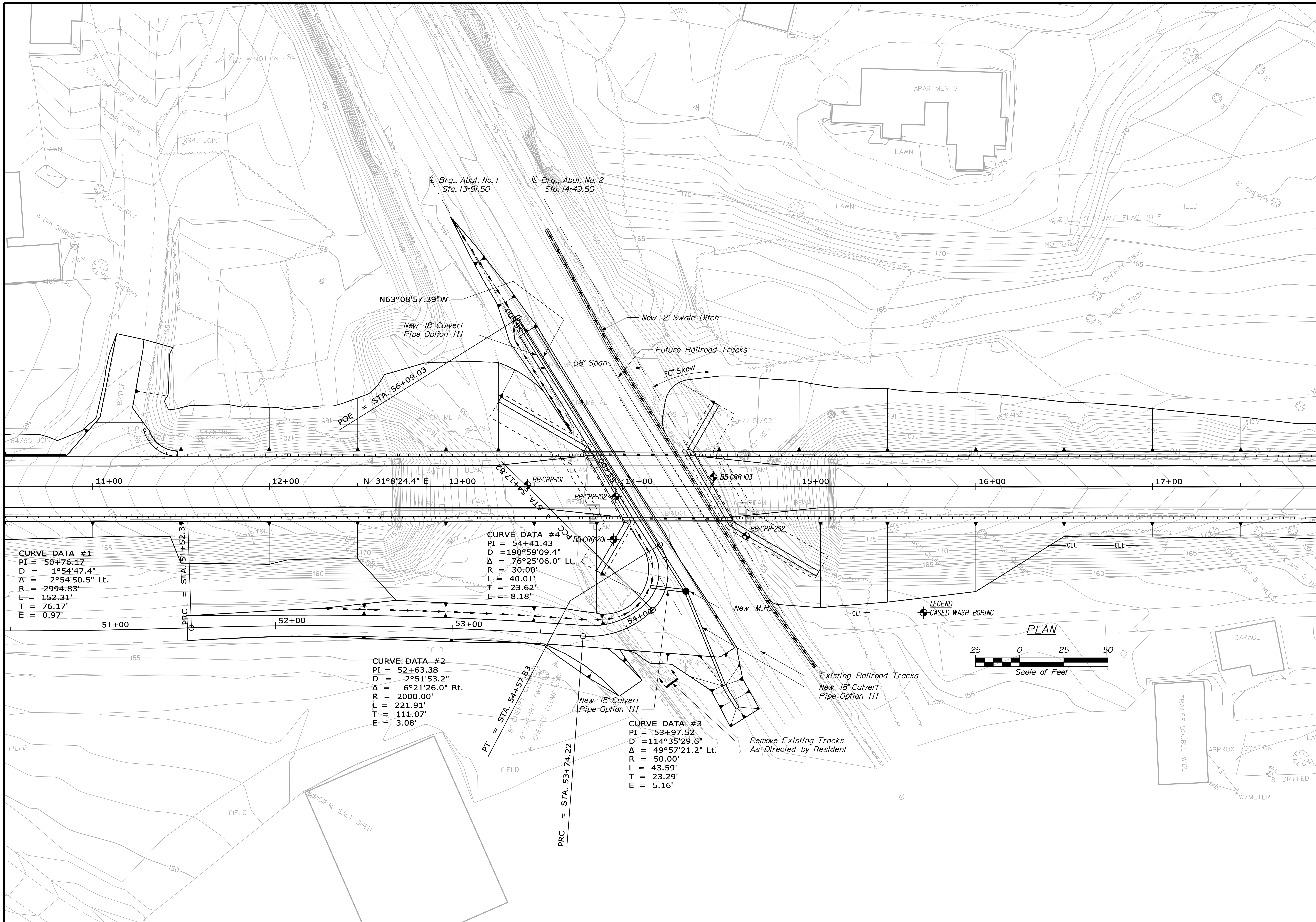
CARMEL QUADRANGLE
MAINE – PENOBSCOT CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000

KILOMETERS: 1 .5 0 1 2
METERS: 1000 0 1000 2000
MILES: 1 .5 0 1
FEET: 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10 000

CONTOUR INTERVAL 10 FEET

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:11/24/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

015622.00

BRIDGE PLANS

PIN
15622.00

BRIDGE NO. 2976

	BY	DATE
PROJ. MANAGER	D. Anderson	
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	OCT 2009
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MCRR CROSSING BRIDGE
MAINE CENTRAL RAILROAD
CARMEL PENOBSCOT COUNTY

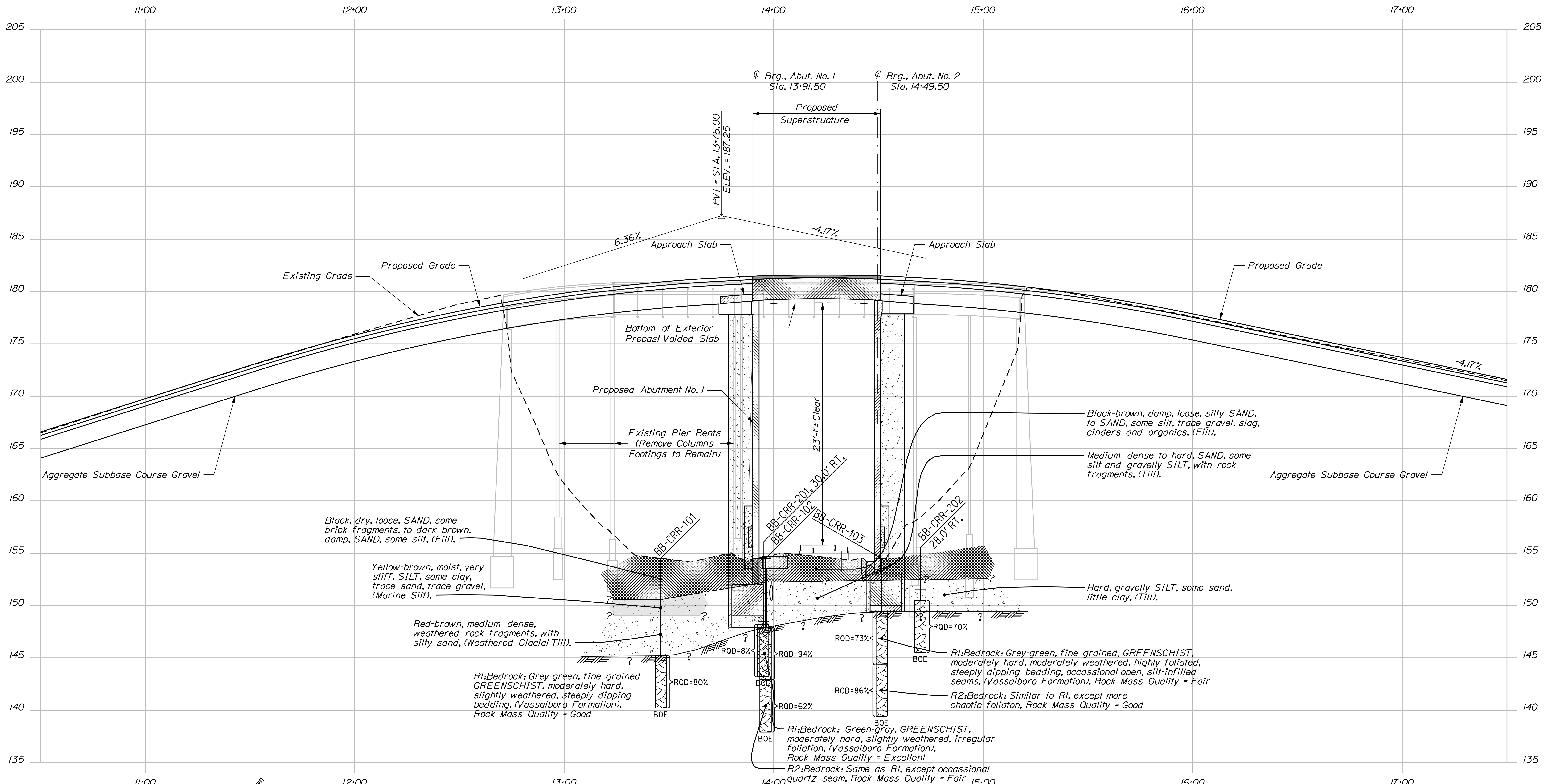
BORING LOCATION PLAN

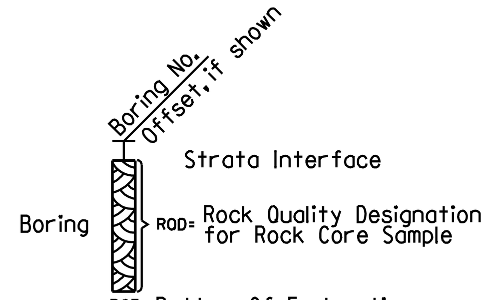
SHEET NUMBER

2

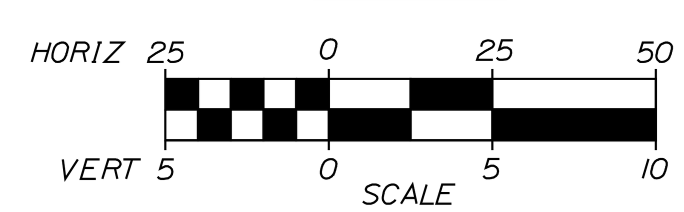
OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:11/24/2009





PROFILE



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

Details of borings BB-CRR-201 and BB-CRR-202 not shown for clarity. For more specific information refer to the Boring Logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

015622.00

PIN 15622.00

BRIDGE NO. 2976

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	OCT 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MCRR CROSSING BRIDGE
MAINE CENTRAL RAILROAD
CARMEL PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:11/24/2009

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

Project: MCRR Crossing Bridge #2976 carring Routes 2/100 over Maine
Location: Carmel, Maine

Boring No.: BB-CRR-101
PIN: 15622.00

Driller:	MaineDOT	Elevation (ft.)	154.5	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:	6/10/08-6/10/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+46.1, 0.81 Lt.	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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
1D	24/24	0.00 - 2.00	2/2/3/3	5	6	SSA	
2D/AB	24/24	4.00 - 6.00	2/5/13/10	18	23	5	150.50
						30	149.30
						55	
						a147 bWA	
R1	60/60	9.30 - 14.30	RQD = 80%			NQ-2	145.20
							140.20

Visual Description and Remarks:

Black-orange, dry, loose, coarse SAND trace of brick fragments (upper 12-inches), grading to dark brown, damp, medium SAND, some silt. (Fill).

4.00

(2D/A) 4.0-5.2' bgs.
Yellow-brown, moist, very stiff, SILT, some clay, trace fine sand, trace fine gravel, layered, nonplastic. (Overconsolidated, weathered Presumpscot Formation).

5.20

(2D/B) 5.2-6.0' bgs.
Red-brown to yellow-brown, medium dense, weathered angular rock fragments, (fine to medium gravel) in a matrix of moist, very stiff, silty medium to coarse sand, stained, weathered. (Weathered Glacial Till).
PP_1 = 2500 psf, PP_2 = 2500 psf
PP_3 = 2500 psf
a147 blows for 0.8'.
bWashed Ahead to 9.3' bgs.

9.30

Top of Bedrock at Elev. 145.2'.
R1: Bedrock: Grey-green, fine-grained, calcareous metasedimentary GREENSCHIST, moderately hard, slight weathering, rust staining and some wearing, steeply dipping, very close laminate bedding, (Vassalboro Formation). Rock Mass Quality: Good.
R1:Core Times (min:sec)
9.3-10.3' (5:10)
10.3-11.3' (6:45)
11.3-12.3' (5:00)
12.3-13.3' (3:45)
13.3-14.3' (3:00) 100% Recovery

14.30

Bottom of Exploration at 14.30 feet below ground surface.

Laboratory Testing Results/AASHTO and Unified Class: G#210014 A-4, CL-ML WC=22.2% Non-Plastic

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-CRR-101

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

Project: MCRR Crossing Bridge #2976 carring Routes 2/100 over Maine
Location: Carmel, Maine

Boring No.: BB-CRR-102
PIN: 15622.00

Driller:	MaineDOT	Elevation (ft.)	154.5	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:	6/9/08; 08:00-10:45	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+96.1, 5.7 Rt.	Casing ID/OD:	HW	Water Level*:	3.0' bgs.
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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
1D	24/24	0.00 - 2.00	2/4/4/8	8	10	SSA	
2D	24/24	2.00 - 4.00	3/4/9/14	13	17		152.50
						4	
3D/AB	24/22	4.00 - 6.00	6/30/40/27	70	90	42	150.50 / 149.90
						119	
R1	60/60	6.60 - 11.60	RQD = 94%			a128 NQ-2	147.90
R2	60/60	11.60 - 16.60	RQD = 62%				142.90
							137.90

Visual Description and Remarks:

Black, damp, loose, silty, fine SAND, little medium to coarse sand, trace fine gravel, trace cinders, slag, organics, (Fill).

2.00

Light brown, mottled, medium dense, SAND, some silt, some weathered slate fragments and fine gravel, little clay, blocky (Weathered fine-grained Glacial Till).
PP_1 = 3500 psf

4.00

(3D/A) 4.0-4.6' bgs.
Yellowish brown, wet, hard, fine angular gravelly SILT, some clay, some staining, little coarse angular sand, (Weathered fine-grained Glacial Till).

4.60

(3D/B) 4.6-6.0' bgs.
Yellowish-red and brown, dry, very dense, weathered rock (slate) fragments, some silt, little clay pockets/seam, rust staining, (Weathered Glacial Till and Bedrock).
a128 blows for 0.5'.
Washed ahead/Roller Coned Ahead to 6.6' bgs.

6.60

Top of Bedrock at Elev. 147.9'.
R1: Bedrock: Grey-green, fine-grained, metasedimentary, calcareous GREENSCHIST, moderately hard, slight weathering, irregular foliation, oxidized stains on breaks, no infilling. (Vassalboro Formation). Rock Mass Quality: Excellent.
R1:Core Times (min:sec)
6.6-7.6' (5:00)
7.6-8.6' (3:23)
8.6-9.6' (2:50)
9.6-10.6' (3:22)
10.6-11.6' (2:09) 100% Recovery

11.60

R2: Bedrock: Greenish grey, fine grained, metasedimentary GREENSCHIST, moderately hard, slightly weathered, foliation irregular, predominaty steep, close bedding, second joint at low angles, surfaces tight to slightly open, stained, slight weathering, occasional quartz seams. (Vassalboro Formation).
Rock Mass Quality: Fair.
R2:Core Times (min:sec)
11.6-12.6' (4:40)
12.6-13.6' (4:46)
13.6-14.6' (4:13)
14.6-15.6' (3:48)
15.6-16.6' (4:17) 100% Recovery

16.60

Bottom of Exploration at 16.60 feet below ground surface.

Laboratory Testing Results/AASHTO and Unified Class: G#210013 A-4, SC-SM WC=15.4%

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-CRR-102

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

Project: MCRR Crossing Bridge #2976 carring Routes 2/100 over Maine
Location: Carmel, Maine

Boring No.: BB-CRR-103
PIN: 15622.00

Driller:	MaineDOT	Elevation (ft.)	154.5	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:	6/9/08; 12:00-16:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+51.5, 5.4 Lt.	Casing ID/OD:	HW	Water Level*:	4.0' bgs.
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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
						SSA	
1D	24/19	1.00 - 3.00	2/1/2/1	3	4		152.50
2D	24/14	4.00 - 6.00	4/9/12(2")	---		47	149.40
R1	60/60	5.10 - 10.10	RQD = 73%			NQ-2	
R2	60/57	10.10 - 15.10	RQD = 86%				144.40
							139.40

Visual Description and Remarks:

Black-brown, damp, very loose, fine to medium SAND, some silt, little fine to coarse gravel, trace slag and organics, (Fill).

2.00

Yellowish brown, wet, hard, angular gravelly SILT, some fine to coarse sand, little clay, increasing weathered bedrock fragments with depth, blocky, oxidized. (Weathered, fine-grained Glacial Till).

5.10

Top of Bedrock at Elev.149.4'.
R1: Bedrock: Grey green, fine grained metasedimentary, GREENSCHIST, moderately hard, moderately weathered, one 2" silt seam, steeply dipping bedding, highly foliated, bedding surfaces tight, but some with staining or silt. At 4'6" 0.2' brown silt seam, (Vassalboro Formation). Lost water at silt seam.
Rock Mass Quality: Fair.
R1:Core Times (min:sec)
5.1-6.1' (4:49)
6.1-7.1' (4:48)
7.1-8.1' (4:52)
8.1-9.1' (3:38)
9.1-10.1' (2:50) 100% Recovery

10.10

R2:Bedrock: Similar to R1, but more chaotic foliation, blue green, fine grained, GREENSCHIST, hard, very slightly weathered, highly foliated at chaotic angles, all moderately tight with stained surfaces except open seam with silt infilling at 4.3', frequent white veins. Losing water during coring. (Vassalboro Formation)
Rock Mass Quality: Good.
R2:Core Times (min:sec)
10.1-11.1' (4:26)
11.1-12.1' (4:45)
12.1-13.1' (3:24)
13.1-14.1' (7:28)
14.1-15.1' (4:48) 97% Recovery

15.10

Bottom of Exploration at 15.10 feet below ground surface.

Remarks:

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

Page 1 of 1 — Boring No.: BB-CRR-103

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

Project: MCRR Crossing Bridge #2976 carring Routes 2/100 over Maine
Location: Carmel, Maine

Boring No.: BB-CRR-201
PIN: 15622.00

Driller:	MaineDOT	Elevation (ft.)	154.5	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/Wright	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/27/09; 07:30-10:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+95, 30.0 Rt.	Casing ID/OD:	HW	Water Level*:	3.3' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
1D/A	24/20	0.00 - 2.00	4/6/7/4	13	18	SSA	152.90
2D	15.6/13	5.00 - 6.30	10/23/50(3.6")	---			148.50
R1	60/54	6.30 - 11.30	RQD = 33%			a100 NQ-2	148.20
							143.20

Visual Description and Remarks:

(1D) 0.0-1.6' bgs.
Black, moist, medium dense, fine to medium SAND, some organic silt, trace roots, brick fragments, and slag. (Fill)

1.60

(1D/A) 1.6-2.0' bgs.
Brown-olive, mottled, moist, medium dense, SAND, little gravel (rock fragments), little silt, blocky. (Glacial Till.)

Brown-olive, mottled, moist, SAND some gravel, little silt, with staining and oxidation, blocky (Weathered Glacial Till).

6.00

a100 blows for 0.3'.
Weathered BEDROCK.

6.30

Top of Intact Bedrock at Elev. 148.2'.
R1: Bedrock: Grey, fine-grained, calcareous metasedimentary, GREENSCHIST, moderately hard, moderately weathered,k highly fractured, highly foliated, steep to vertical angles, tight, stained, some silt infilled. Some quartz veins in the upper 1 foot. Rock Mass Quality = poor.
Vassalboro Formation.
R1: Core Times (min:sec)
6.3-7.3' (4:30)
7.3-8.3' (6:00)
8.3-9.3' (5:30)
9.3-10.3' (5:00)
10.3-11.3' (4:50) 90% Recovery

11.30

Bottom of Exploration at 11.30 feet below ground surface.

Laboratory Testing Results/AASHTO and Unified Class: G#246342 A-1-b, SM WC=11.3%; G#246343 A-1-b, SM WC=7.7%

Remarks:
400-500 lbs. down pressure on Core Barrel.

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-CRR-201

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

Project: MCRR Crossing Bridge #2976 carring Routes 2/100 over Maine
Location: Carmel, Maine

Boring No.: BB-CRR-202
PIN: 15622.00

Driller:	MaineDOT	Elevation (ft.)	155.5	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/Wright	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/27/09; 07:30-10:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+70, 28.0 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
1D	24/16	0.00 - 2.00	2/3/2/3	5	7	SSA	
						a100	151.50
R1	60/57	5.00 - 10.00	RQD = 73%			NQ-2	150.50
							145.50

Visual Description and Remarks:

Dark brown, dry to damp, loose, fine to coarse SAND, some rounded gravel, little silt, trace organics and roots. (Fill.)

4.00

a100 blows for 0.5'.
Weathered BEDROCK.

5.00

Top of Intact Bedrock at Elev. 150.5'.
R1:Bedrock: Grey, fine-grained GREENSCHIST, hard, fresh to moderately weathered, highly foliated, steep to vertical, very close, tight, slightly stained, second joint orthogonal to bedding. Highly fractured zone with weathered seams in 0-12". Vassalboro Formation. Rock Mass Qualtity: Fair.
R1:Core Times (min:sec)
50.-6.0' (3:20)
6.0-7.0' (3:00)
7.0-8.0' (3:10)
8.0-9.0' (3:00)
9.0-10.0' (2:45) 95% Recovery

10.00

Bottom of Exploration at 10.00 feet below ground surface.

Laboratory Testing Results/AASHTO and Unified Class: G#246344 A-1-b, SM WC=7.6%

Remarks:
400-500 lbs. down pressure on Core Barrel.

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-CRR-202

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
015622.00
BRIDGE PLANS
PIN 15622.00
BRIDGE NO. 2976

	BY	DATE
PROJ. MANAGER	D. Anderson	
DESIGN-DETAILED	L. KRUSINSKI	OCT 2009
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MCRR CROSSING BRIDGE
MAINE CENTRAL RAILROAD
CARMEL PENOBSCOT COUNTY
BORING LOGS

SHEET NUMBER 4 OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
-spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
-tightness (tight, open or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: MCRR Crossing Bridge #2976 carrying Routes 2/100 over Maine Central RR
Location: Carmel, Maine

Boring No.: BB-CRR-101
PIN: 15622.00

Driller:	MaineDOT	**Elevation (ft.)**	154.5	**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:	6/10/08-6/10/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+46.1, 0.81 Lt.	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/24	0.00 - 2.00	2/2/3/3	5	6	SSA			Black-orange, dry, loose, coarse SAND trace of brick fragments (upper 12-inches), grading to dark brown, damp, medium SAND, some silt. (Fill).	
								150.50		4.00	
5	2D/AB	24/24	4.00 - 6.00	2/5/13/10	18	23	5			(2D/A) 4.0-5.2' bgs. Yellow-brown, moist, very stiff, SILT, some clay, trace fine sand, trace fine gravel, layered, nonplastic. (Overconsolidated, weathered Presumpscot Formation).	G#210014 A-4, CL-ML WC=22.2% Non-Plastic
							30	149.30		5.20	
							55			(2D/B) 5.2-6.0' bgs. Red-brown to yellow-brown, medium dense, weathered angular rock fragments, (fine to medium gravel) in a matrix of moist, very stiff, silty medium to coarse sand, stained, weathered. (Weathered Glacial Till). PP_1 = 2500 psf, PP_2 = 2500 psf PP_3 = 2500 psf [a]147 blows for 0.8'. [b]Washed Ahead to 9.3' bgs.	
							[a]147 [b]WA				
	R1	60/60	9.30 - 14.30	RQD = 80%			NQ-2	145.20		9.30	
10										Top of Bedrock at Elev. 145.2'. R1: Bedrock: Grey-green, fine-grained, calcareous metasedimentary GREENSCHIST, moderately hard, slight weathering, rust staining and some wearing, steeply dipping, very close laminate bedding, (Vassalboro Formation). Rock Mass Quality: Good. R1:Core Times (min:sec) 9.3-10.3' (5:10) 10.3-11.3' (6:45) 11.3-12.3' (5:00) 12.3-13.3' (3:45) 13.3-14.3' (3:00) 100% Recovery	
								140.20			
15										14.30 **Bottom of Exploration at 14.30 feet below ground surface.**	
20											
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: MCRR Crossing Bridge #2976 carrying Routes 2/100 over Maine Central RR
Location: Carmel, Maine

Boring No.: BB-CRR-102
PIN: 15622.00

Driller:	MaineDOT	**Elevation (ft.)**	154.5	**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:	6/9/08; 08:00-10:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+96.1, 5.7 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.0' bgs.
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	1D	24/24	0.00 - 2.00	2/4/4/8	8	10	SSA			Black, damp, loose, silty, fine SAND, little medium to coarse sand, trace fine gravel, trace cinders, slag, organics, (Fill).	
								152.50		2.00	
	2D	24/24	2.00 - 4.00	3/4/9/14	13	17				Light brown, mottled, medium dense, SAND, some silt, some weathered slate fragments and fine gravel, little clay, blocky (Weathered fine-grained Glacial Till). PP_1 = 3500 psf	G#210013 A-4, SC-SM WC=15.4%
							4	150.50		4.00	
	3D/AB	24/22	4.00 - 6.00	6/30/40/27	70	90	42	149.90		(3D/A) 4.0-4.6' bgs. Yellowish brown, wet, hard, fine angular gravelly SILT, some clay, some staining, little coarse angular sand, (Weathered fine-grained Glacial Till).	
5							119			4.60	
	R1	60/60	6.60 - 11.60	RQD = 94%			a128 NQ-2	147.90		(3D/B) 4.6-6.0' bgs. Yellowish-red and brown, dry, very dense, weathered rock (slate) fragments, some silt, little clay pockets/seam, rust staining, (Weathered Glacial Till and Bedrock). a128 blows for 0.5'. Washed ahead/Roller Coned Ahead to 6.6' bgs.	
										6.60	
10										Top of Bedrock at Elev. 147.9'. R1: Bedrock: Grey-green, fine-grained, metasedimentary, calcareous GREENSCHIST, moderately hard, slight weathering, irregular foliation, oxidized stains on breaks, no infilling. (Vassalboro Formation). Rock Mass Quality: Excellent. R1:Core Times (min:sec) 6.6-7.6' (5:00) 7.6-8.6' (3:23) 8.6-9.6' (2:50) 9.6-10.6' (3:22) 10.6-11.6' (2:09) 100% Recovery	
	R2	60/60	11.60 - 16.60	RQD = 62%				142.90		11.60	
15								137.90		R2: Bedrock: Greenish grey, fine grained, metasedimentary GREENSCHIST, moderately hard, slightly weathered, foliation irregular, predominaty steep, close bedding, second joint at low angles, surfaces tight to slightly open, stained, slight weathering, occasional quartz seams. (Vassalboro Formation). Rock Mass Quality: Fair. R2:Core Times (min:sec) 11.6-12.6' (4:40) 12.6-13.6' (4:46) 13.6-14.6' (4:13) 14.6-15.6' (3:48) 15.6-16.6' (4:17) 100% Recovery	
										16.60	
20										**Bottom of Exploration at 16.60 feet below ground surface.**	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: MCRR Crossing Bridge #2976 carrying Routes 2/100 over Maine Central RR
Location: Carmel, Maine

Boring No.: BB-CRR-103
PIN: 15622.00

Driller:	MaineDOT	**Elevation (ft.)**	154.5	**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:	6/9/08; 12:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+51.5, 5.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA				
	1D	24/19	1.00 - 3.00	2/1/2/1	3	4				Black-brown, damp, very loose, fine to medium SAND, some silt, little fine to coarse gravel, trace slag and organics, (Fill).	
								152.50		2.00	
	2D	24/14	4.00 - 6.00	4/9/12(2")	---		47			Yellowish brown, wet, hard, angular gravelly SILT, some fine to coarse sand, little clay, increasing weathered bedrock fragments with depth, blocky, oxidized. (Weathered, fine-grained Glacial Till).	
5	R1	60/60	5.10 - 10.10	RQD = 73%			NQ-2	149.40		5.10	
										Top of Bedrock at Elev.149.4'. R1: Bedrock: Grey green, fine grained metasedimentary, GREENSCHIST, moderately hard, moderately weathered, one 2" silt seam, steeply dipping bedding, highly foliated, bedding surfaces tight, but some with staining or silt. At 4'6" 0.2' brown silt seam, (Vassalboro Formation). Lost water at silt seam. Rock Mass Quality: Fair. R1:Core Times (min:sec) 5.1-6.1' (4:49) 6.1-7.1' (4:48) 7.1-8.1' (4:52) 8.1-9.1' (3:38) 9.1-10.1' (2:50) 100% Recovery	
10	R2	60/57	10.10 - 15.10	RQD = 86%				144.40		10.10	
										R2:Bedrock: Similar to R1, but more chaotic foliation, blue green, fine grained, GREENSCHIST, hard, very slightly weathered, highly foliated at chaotic angles, all moderately tight with stained surfaces except open seam with silt infilling at 4.3', frequent white veins. Losing water during coring. (Vassalboro Formation) Rock Mass Quality: Good. R2:Core Times (min:sec) 10.1-11.1' (4:26) 11.1-12.1' (4:45) 12.1-13.1' (3:24) 13.1-14.1' (7:28) 14.1-15.1' (4:48) 97% Recovery	
15								139.40		15.10	
										Bottom of Exploration at 15.10 feet below ground surface.	
20											
25											

Remarks:

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* 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: MCRR Crossing Bridge #2976 carrying Routes 2/100 over Maine Central RR
Location: Carmel, Maine

Boring No.: BB-CRR-201
PIN: 15622.00

Driller:	MaineDOT	**Elevation (ft.)**	154.5	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/Wright	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/27/09; 07:30-10:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+95, 30.0 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.3' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	1D/A	24/20	0.00 - 2.00	4/6/7/4	13	18	SSA			(1D) 0.0-1.6' bgs. Black, moist, medium dense, fine to medium SAND, some organic silt, trace roots, brick fragments, and slag. (Fill)	
								152.90		1.60	
										(1D/A) 1.6-2.0' bgs. Brown-olive, mottled, moist, medium dense, SAND, little gravel (rock fragments), little silt, blocky. (Glacial Till.)	G#246342 A-1-b, SM WC=11.3%
5	2D	15.6/13	5.00 - 6.30	10/23/50(3.6")	---					Brown-olive, mottled, moist, SAND some gravel, little silt, with staining and oxidation, blocky (Weathered Glacial Till).	G#246343 A-1-b, SM WC=7.7%
								148.50		6.00	
	R1	60/54	6.30 - 11.30	RQD = 33%			a100 NQ-2	148.20		a100 blows for 0.3'. Weathered BEDROCK.	
										6.30	
										Top of Intact Bedrock at Elev. 148.2'. R1: Bedrock: Grey, fine-grained, calcareous metasedimentary, GREENSCHIST, moderately hard, moderately weathered,k highly fractured, highly foliated, steep to vertical angles, tight, stained, some silt infilled. Some quartz veins in the upper 1 foot. Rock Mass Quality = poor. Vassalboro Formation. R1: Core Times (min:sec) 6.3-7.3' (4:30) 7.3-8.3' (6:00) 8.3-9.3' (5:30) 9.3-10.3' (5:00) 10.3-11.3' (4:50) 90% Recovery	
10								143.20		11.30	
										Bottom of Exploration at 11.30 feet below ground surface.	
15											
20											
25											

Remarks:

400-500 lbs. down pressure on Core Barrel.

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: MCRR Crossing Bridge #2976 carrying Routes 2/100 over Maine Central RR
Location: Carmel, Maine

Boring No.: BB-CRR-202
PIN: 15622.00

Driller:	MaineDOT	**Elevation (ft.)**	155.5	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/Wright	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/27/09; 07:30-10:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+70, 28.0 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	2/3/2/3	5	7	SSA			Dark brown, dry to damp, loose, fine to coarse SAND, some rounded gravel, little silt, trace organics and roots. (Fill.)	G#246344 A-1-b, SM WC=7.6%
							a100	151.50		4.00 a100 blows for 0.5'. Weathered BEDROCK.	
5	R1	60/57	5.00 - 10.00	RQD = 73%			NQ-2	150.50		5.00 Top of Intact Bedrock at Elev. 150.5'. R1:Bedrock: Grey, fine-grained GREENSCHIST, hard, fresh to moderately weathered, highly foliated, steep to vertical, very close, tight, slightly stained, second joint orthogonal to bedding. Highly fractured zone with weathered seams in 0-12". Vassalboro Formation. Rock Mass Qualtity: Fair. R1:Core Times (min:sec) 50.-6.0' (3:20) 6.0-7.0' (3:00) 7.0-8.0' (3:10) 8.0-9.0' (3:00) 9.0-10.0' (2:45) 95% Recovery	
10								145.50		10.00 **Bottom of Exploration at 10.00 feet below ground surface.**	
15											
20											
25											

Remarks:

400-500 lbs. down pressure on Core Barrel.

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: 15622.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-CRR-101, 2D/A	13+46.1	0.81 Lt.	4.0-5.2	210014	1	22.2	-N	P-	CL-ML	A-4	IV
BB-CRR-102, 2D	13+96.1	5.7 Rt.	2.0-4.0	210013	1	15.4			SC-SM	A-4	III
BB-CRR-201, 1D/A	13+95	30.0 Rt.	1.6-2.0	246342	1	11.3			SM	A-1-b	II
BB-CRR-201, 2D	13+95	30.0 Rt.	5.0-6.3	246343	1	7.7			SM	A-1-b	II
BB-CRR-202, 1D	14+70	28.0 Rt.	0.0-2.0	246344	1	7.6			SM	A-1-b	II

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

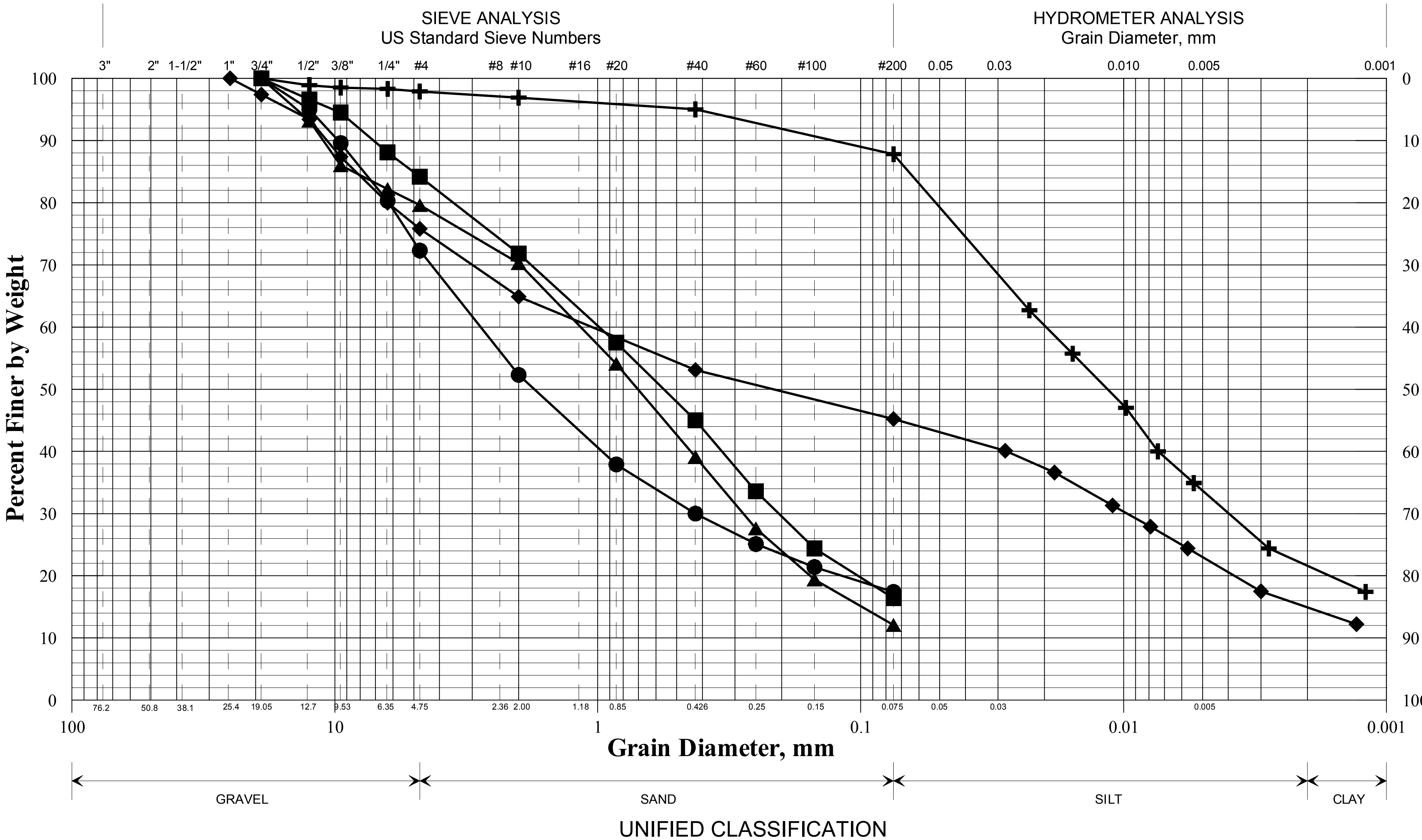
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-CRR-101/2D/A	13+46.1	0.81 LT	4.0-5.2	SILT, some clay, trace sand, trace gravel.	22.2			NP
◆	BB-CRR-102/2D	13+96.1	5.7 RT	2.0-4.0	SAND, some silt, some gravel, little clay.	15.4			
■	BB-CRR-201/1DA	13+95	30.0 RT	1.6-2.0	SAND, little silt, little gravel.	11.3			
●	BB-CRR-201/2D	13+95	30.0 RT	5.0-6.3	SAND, some gravel, little silt.	7.7			
▲	BB-CRR-202/1D	14+70	28.0 RT	0.0-2.0	SAND, some gravel, little silt.	7.6			
✖									

PIN	
015622.00	
Town	
Carmel	
Reported by/Date	
WHITE, TERRY A	10/19/2009

SHEET 1

Appendix C

Calculations

Bearing Resistance- Abutment 1 and 2 Spread Footing Foundations

Method 1

Method: LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Description of Bearing Material:

Abutment 1: Boring BB-CRR-102, upper 5-ft core, Metamorphic GREENSCHIST, moderately hard, slightly weathered, irregular foliation, no infilling. RQD=94%. Lower 5-ft core similar but more jointing and some slightly open seams - RQD=64%

Abutment 2: Boring BB-CRR-103, upper 5-ft core is metamorphic, GREENSCHIST, moderately hard, moderately weathered, one 2" silt seam, highly foliated. Lost water in the silt seam. RQD is 73%. Lower 5-ft core is similar, but with more chaotic foliation, hard, very slightly weathered. Lost some water during coring.

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

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

Method 2

AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2. Footings on Broken or Jointed Rock

Table 4.4.8.1.2.A - for footings supported on jointed rock.

a. estimated RMR, Rock Mass Rating, Fair. RQD Range is 62 to 94% (Fair to Excellent)

b. Rock Category per 4.4.8.1.2B E = Schist

c. Unconfined compressive strength, Co 10,000 psi (average from range 1,400 - 21,000 psi)

d. Nms, per Table 4.4.8.1.2A Table states to use Nms=.081 for Fair Rock, however considering the silt seam and water loss, use the category with "joints spaced 1-20 inches with some gouge". Nms=0.024

e. Nominal Bearing Resistance Nms x Co

Nominal Bearing Resistance

$Q_{nom} := 0.024 \cdot 10000 \cdot psi$ $Q_{nom} = 34.56 \cdot ksf$

Factored Bearing Resistance

$\phi := 0.45$

$Q_{factored} := Q_{nom} \cdot \phi$

$\boxed{Q_{factored} = 15.552 \cdot ksf}$

Recommend a factored bearing resistance 15 ksf for the Strength Limit State Analysis.

Assume an unfactored Service Load Combination of a maximum of 20 ksf to perform a settlement analysis (follows).

Settlement Analysis of Footings on Rock, LRFD 10.6.2.4.4

Per LRFD, 10.6.2.4.4, elastic settlements may generally be assumed to be less than 0.5 inches. However, the magnitude of consolidation settlement in rock masses containing soft seams should be estimated by applying procedures specified in Article 10.6.2.4.3.

Open silt infilled seams observed in rock cores of BB-CRR-103, at 4'6" into core. Seam 2.4" thick - use 3".

Silt properties - assume OCR => 1.0 e_o=1.00 and C_c = 0.30 and Cr = 0.03. Assume preconsolidated since the silt seam is near the surface of the bedrock

$$e_o := 1.0$$

$$C_r := 0.030$$

Depth of seam is 54 inches below footing with applied load of 16 ksf (Unfactored Service Load Combination.

Per LRFD Figure 10.6.2.4.1-1, Boussinesq Vertical Stress Contours

Assume Footing Width, B = 15 feet

Depth of interest is approximately 0.3B

Stress is approximately 0.9q_o

$$q_o := 20 \cdot ksf$$

$$\Delta\sigma_v := 0.9 \cdot q_o$$

$$\Delta\sigma_v = 18 \cdot ksf$$

Existing overburden stress

Profile is approx. 1-ft of granular fill soils and 4 ft of fine-grained till, w/ water table at 4 ft bgs, and 4.5 feet of bedrock

$$\gamma_{fill} := 120 \cdot pcf$$

$$\gamma_{rock} := 150 \cdot pcf$$

$$\gamma_{till} := 125 \cdot pcf$$

$$\sigma_v := \left(\gamma_{fill} \cdot 1 \cdot ft\right) + \gamma_{till} \cdot 3 \cdot ft + \left(\gamma_{till} - 62.4 \cdot pcf\right) \cdot 1 \cdot ft + \left(\gamma_{rock} - 62.4 \cdot pcf\right) \cdot 4.5 \cdot ft$$

$$\sigma_v = 0.952 \cdot ksf$$

Calculate Settlement

$$\Delta H := 3 \cdot in \cdot \left(\frac{C_r}{1 + e_o}\right) \log\left(\frac{\sigma_v + \Delta\sigma_v}{\sigma_v}\right)$$

$$\Delta H = 0.058 \cdot in$$

Settlment of up to 0.1 inches possible due to consolidation settlement in a soft seam in the bedrock

By: Laura Krusinski
Date: 11/2009
Check By: MJM 11-19-09

Slope Stability Analyses

Slope Stability Analysis Location	Factor of Safety without Compaction of Surficial Fill Unit	Factor of Safety with Compaction of Surficial Fill Unit
Abutment 1 - 26 feet of new fill	1.5	1.6
Abutment 2 – 23 feet of new fill	1.4	1.5

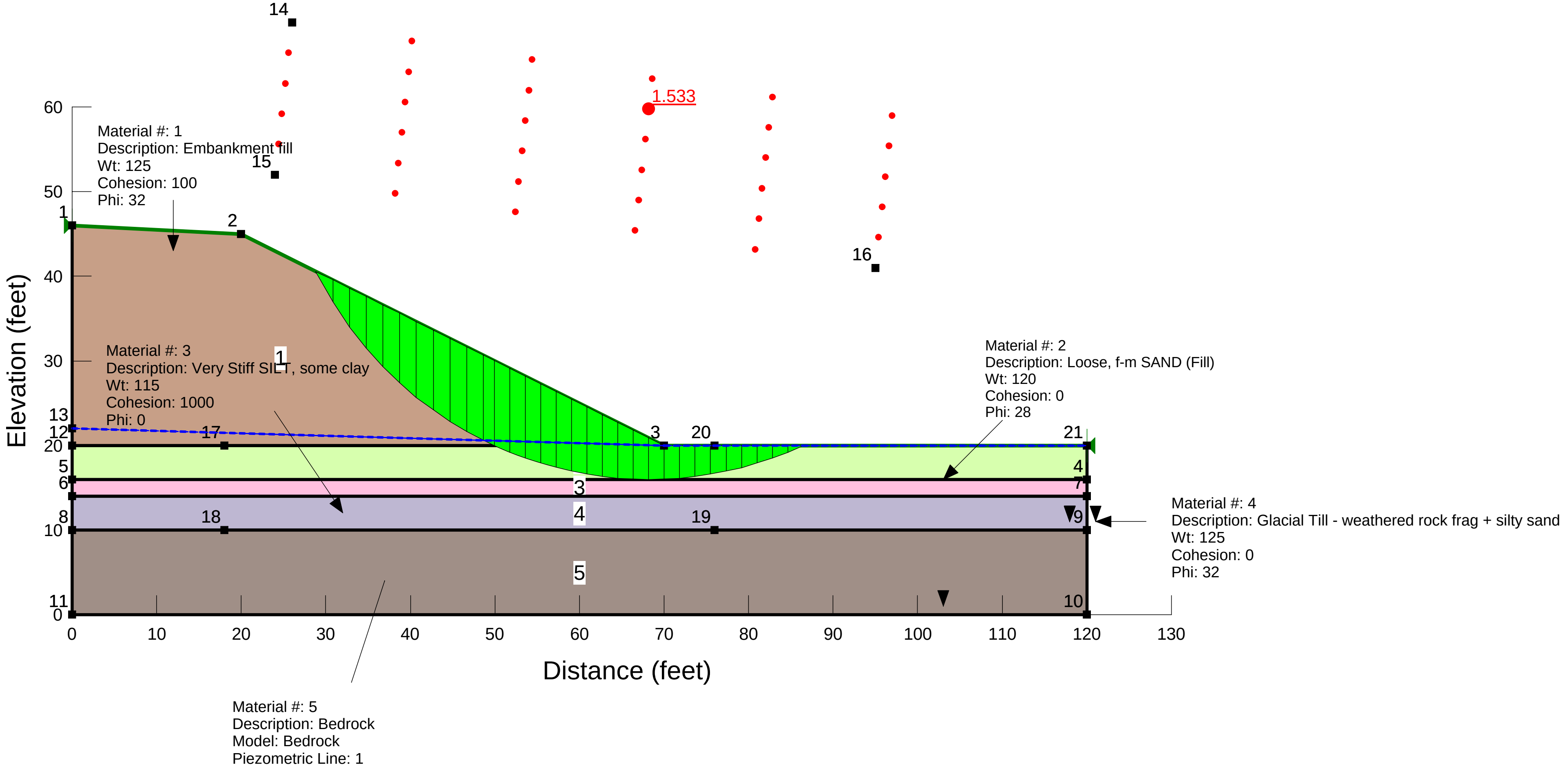
Slope Stability Factors of Safety

Sheet 1 – Abutment 1 X-section slope at Sta 13+50 – native fill soil unit uncompacted
Sheet 2 - Abutment 1 X-section slope at Sta 13+50 – with compaction of surficial fill layer
Sheet 3 – Abutment 2 X-section slope at Sta 14+75 – native fill unit uncompacted
Sheet 4 - Abutment 2 X-section slope at Sta 14+75 – with compaction of fill unit

Name: Carmel Abutment 1 X-section 13+50 cohesion in fill.gsz

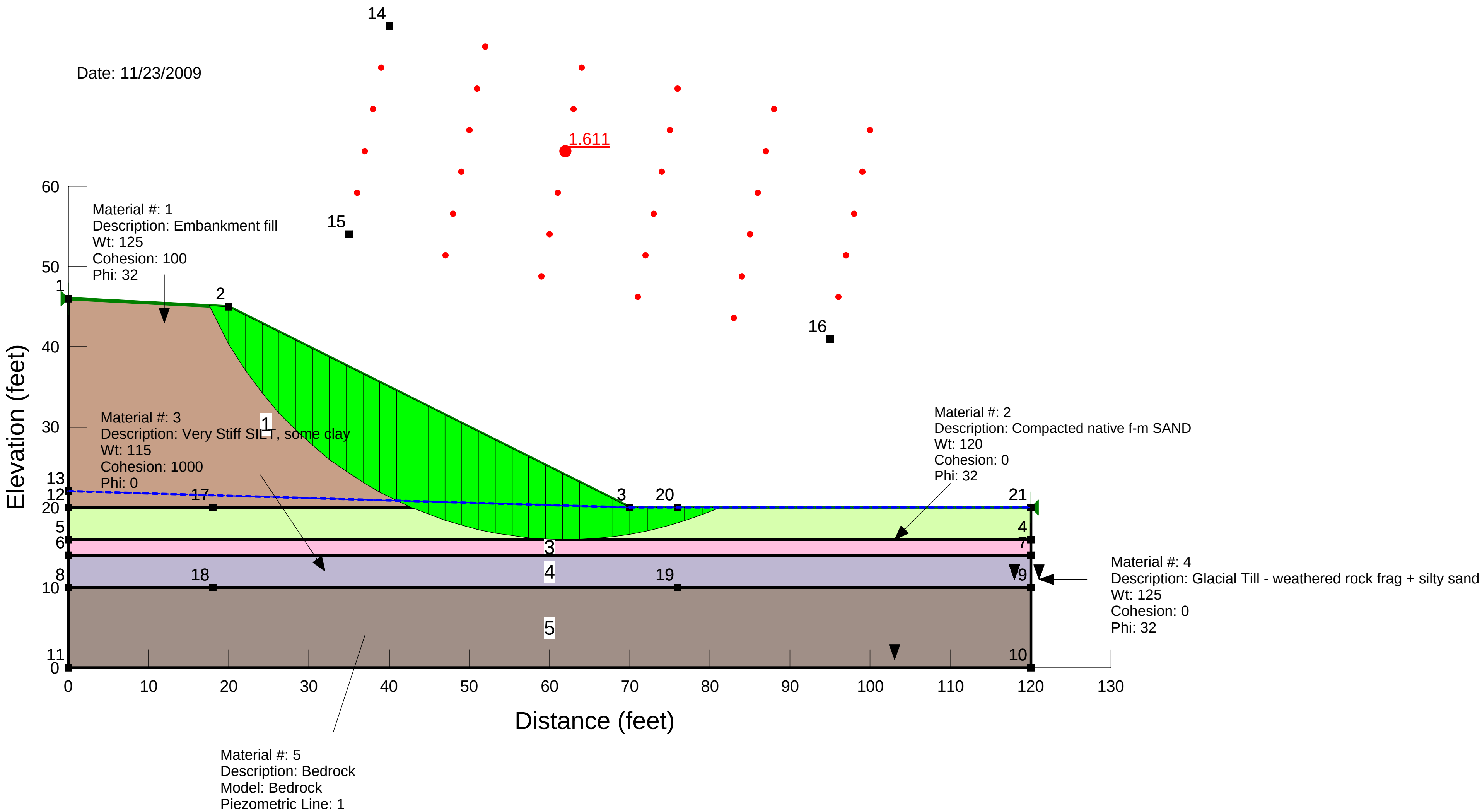
Date: 11/23/2009

Name: Carmel Abutment 1 X-section 13+50 cohesion in fill.gsz
Title: Carmel Slope Stability, (cohesion added to embank fill)
Date: 11/23/2009
Comments: Left to right slope failure at Station 13+50



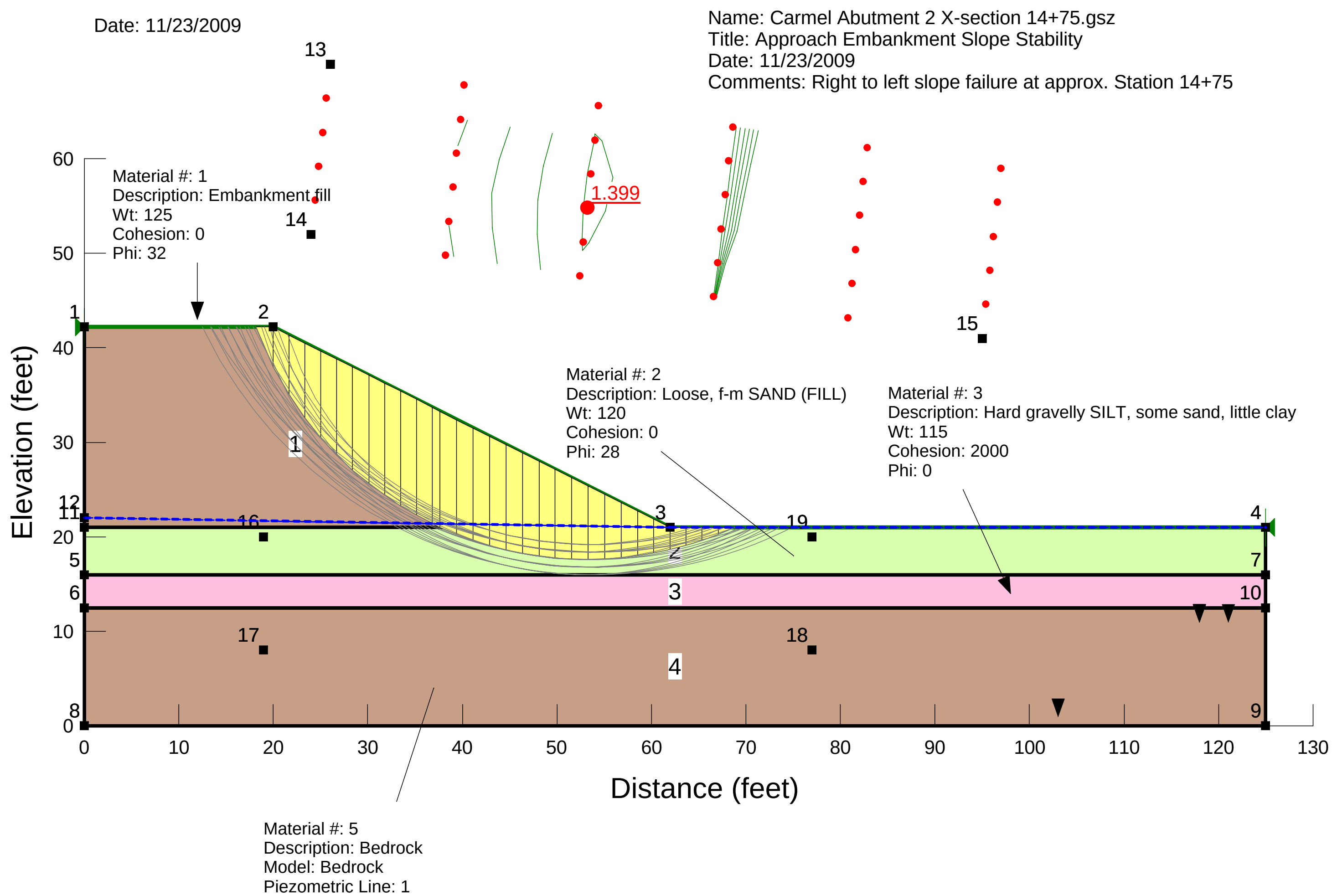
Name: Carmel Abutment 1 X-section 13+50 compacted fill.gsz
Title: Carmel Slope Stability, (RR fill compacted, cohesion added to embank fill)
Date: 11/23/2009
Comments: Left to right slope failure at Station 13+50

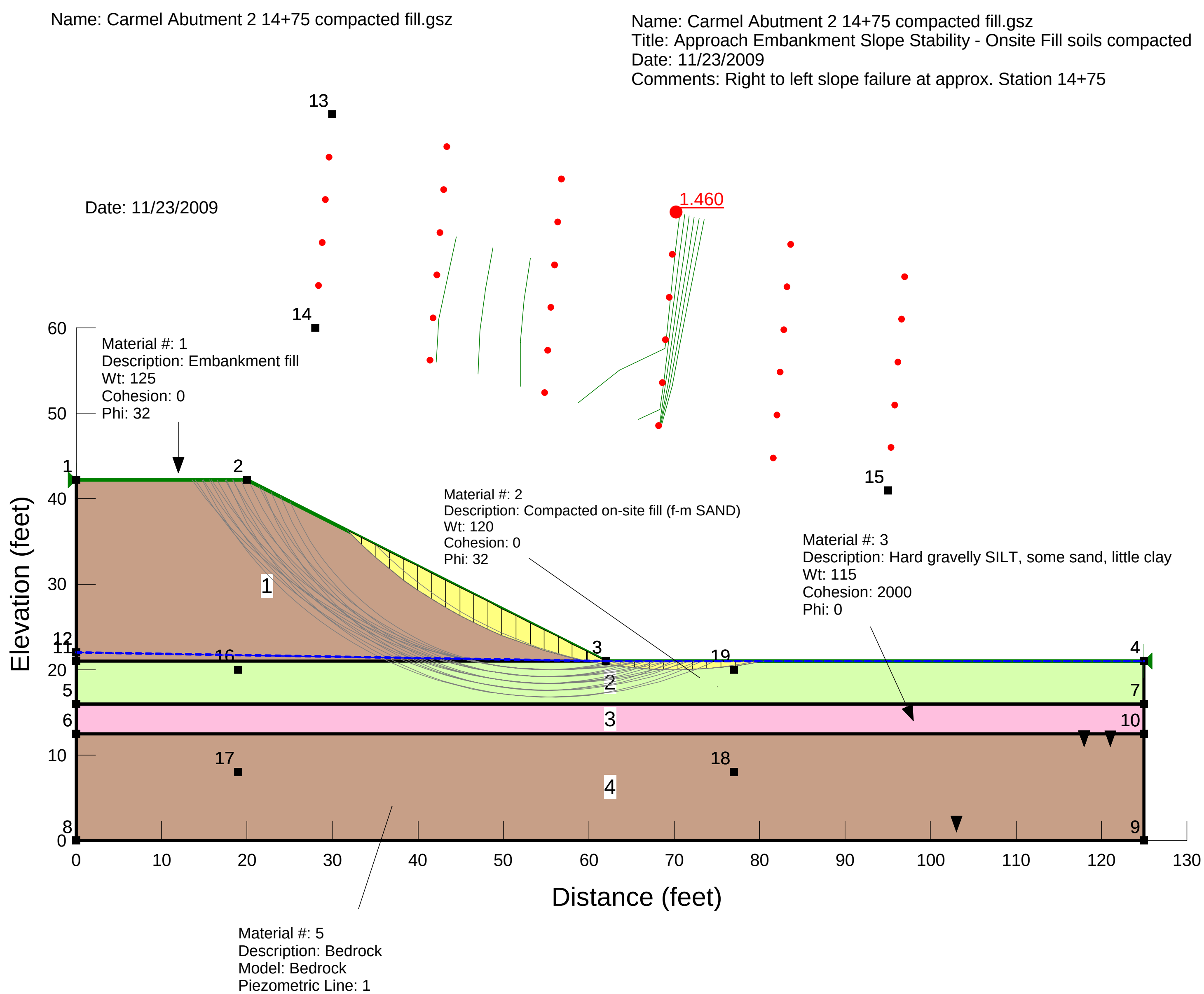
Name: Carmel Abutment 1 X-section 13+50 compacted fill.gsz



Name: Carmel Abutment 2 X-section 14+75.gsz

Date: 11/23/2009





Name: Carmel Abutment 2 14+75 compacted fill.gsz
Name: Carmel Abutment 2 14+75 compacted fill.gsz
Title: Approach Embankment Slope Stability - Onsite Fill soils compacted
Date: 11/23/2009
Comments: Right to left slope failure at approx. Station 14+75
Date: 11/23/2009
1.460
Material #: 1
Description: Embankment fill
Wt: 125
Cohesion: 0
Phi: 32
Material #: 2
Description: Compacted on-site fill (f-m SAND)
Wt: 120
Cohesion: 0
Phi: 32
Material #: 3
Description: Hard gravelly SILT, some sand, little clay
Wt: 115
Cohesion: 2000
Phi: 0
Material #: 5
Description: Bedrock
Model: Bedrock
Piezometric Line: 1
Elevation (feet)
Distance (feet)
0
10
20
30
40
50
60
70
80
90
100
110
120
130

Calculation of Elastic and Consolidation Settlement due to 26 ft of embankment fill for proposed Abutment No. 1 approach - Soil profile based on strata encountered at BB-CRR-101

Soil properties & groundwater conditons; unit weight per LRFD 3.5.1-1

$\gamma_t := 120 \cdot pcf$ $\quad \gamma_w := 62.4 \cdot pcf$ $\quad \gamma' := \gamma_t - \gamma_w$ $\quad \gamma' = 57.6 \cdot pcf$ $\quad D_w := 10 \cdot ft$

groundwater not observed

N values already corrected for hammer efficiency

$$N := \begin{pmatrix} 6 \\ 23 \end{pmatrix}$$

Drained friction angles per LRFD 10.4.6.2.4-1

N160	ϕ
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Soil Profile at BB-CRR-101

First Layer- loose to medium dense fill soil with slag, cinder, brick fragments
0-4 feet, sand and sand some silt. H=4 feet
Second Stratum - glacial marine silt
4 - 5.2 feet, very stiff silt. H=1.2 ft
Third Stratum - glacial till
4.1 feet thick, rock fragments and very stiff, silty sand.

Settlement Computation for Cohesionless Soils

Reference : FHWA Soils and Foundation Workshop Manual NHI-06-088, 2006

Existing Vertical Overburden Stress and Change in vertical stress due to 26 feet of embankment fill.

Break soil profile into six layers:

Layer 1 - 4 feet of fill, 120 pcf, 32 degrees
Layer 2 - 1.2 feet of silt, 115 pcf, 17 degrees, c=2000 psf if very stiff, 1000 psf if stiff
Layer 3 - 4 feet of glacial till, 125 pcf 32 degrees

See Sheet 5 for STRESS output for change in vertical stress.

The change in stresses below are at the center of each layer:

$$\Delta\sigma z := \begin{pmatrix} 1624.87 \\ 1623.51 \\ 1619.54 \end{pmatrix} \cdot psf$$

Layer 1

SPT (bpf) $\quad Navg := \frac{(6 + 10 + 18 + 4 + 7)}{4} \quad Navg = 11$

If SPT at 0-2 feet $\quad \sigma_2 := 1 \cdot ft \cdot 120 \cdot pcf \quad \sigma_2 = 120 \cdot psf \quad$ at 1 ft bgs

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_2 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_2}\right)$$

Should not exceed 2.0

$$CN_2 = 1.943$$

$$Ncor1 := CN_2 \cdot Navg$$

$$Ncor1 = 22$$

FHWA NHI-06-088, Figure 7-7, Curve for "Clean well graded fine to coarse SAND"

Bearing Capacity Index $\quad C_2 := 72$

Layer $\quad H_2 := 4 \cdot ft$

Effective overburden stress at midpoint of layer $\quad \sigma'_2 := 2 \cdot ft \cdot 120 \cdot pcf \quad \sigma'_2 = 240 \cdot psf$

Do not use a σv less than 200 psf

Settlement

$$\Delta H_2 := \overrightarrow{\left[H_2 \cdot \frac{1}{C_2} \cdot \log\left[\frac{(\sigma'_2) + \Delta\sigma z_0}{\sigma'_2}\right]\right]}$$

$$\Delta H_2 = 0.594 \cdot in$$

Layer 2

Field SPT (bpf) $N_1 = 23$ at 4-6 ft bgs

Overburden pressure at SPT elevation $\sigma_3 := 4.0 \cdot ft \cdot 120 \cdot pcf + 1 \cdot ft \cdot 115 \cdot pcf$

$\sigma_3 = 595 \cdot psf$

N - value correction for overburdent per LRFD 10.4.6.2.4

$CN_3 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_3}\right)$ Should not exceed 2.0

$CN_3 = 1.407$

$Ncor1 := CN_3 \cdot N_1$

$Ncor1 = 32$

NHI-08-088, Figure 7-7, Curve for INORGANIC SILT

Bearing Capacity Index $C_3 := 60$

Layer $H_3 := 1.2 \cdot ft$

Effective overburden stress at midpoint of layer $\sigma'_3 := 4 \cdot ft \cdot 120 \cdot pcf + 0.6 \cdot ft \cdot 115 \cdot pcf$

$\sigma'_3 = 549 \cdot psf$

Settlement

$$\Delta H_3 := \overrightarrow{\left[H_3 \cdot \frac{1}{C_3} \cdot \log\left[\frac{(\sigma'_3) + \Delta\sigma z_1}{\sigma'_3}\right]\right]}$$

$\Delta H_3 = 0.143 \cdot in$

Layer 3

Estimated field SPT (bpf) from interval above $N_1 = 23$

Overburden pressure at SPT elevation
(SPT from 4' -6 use 5 ft) $\sigma_4 := \sigma_3$

$$\sigma_4 = 595 \cdot psf$$

N - value correction for overburden per LRFD 10.4.6.2.4

$$CN_4 := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_4}\right)$$

Should not exceed 2.0

$$CN_4 = 1.407$$

$$Ncor1 := CN_4 \cdot N_1$$

$$Ncor1 = 32$$

NHI-06-088, Figure 7-7, Curve for Well graded silty SAND & GRAVEL

Bearing Capacity Index $C_4 := 110$

Layer $H_4 := 4 \cdot ft$

Effective overburden stress at midpoint of layer

$$\sigma'_4 := \sigma'_3 + 0.6 \cdot ft \cdot 115 \cdot pcf + 2 \cdot ft \cdot 125 \cdot pcf$$

$$\sigma'_4 = 868 \cdot psf$$

Settlement

$$\Delta H_4 := \overrightarrow{\left[H_4 \cdot \frac{1}{C_4} \cdot \log\left[\frac{(\sigma'_4) + \Delta\sigma z_2}{\sigma'_4}\right]\right]}$$

$$\Delta H_4 = 0.2 \cdot in$$

Total Elastic Settlement

$$\Delta H_T := \Delta H_2 + \Delta H_3 + \Delta H_4$$

$$\boxed{\Delta H_T = 0.937 \cdot in}$$

Calcution of change in vertical streess due to 26 feet of new fill

$\text{Load} := 26.0 \cdot \text{ft} \cdot 125 \cdot \text{pcf}$ $\qquad$ $\text{Load} = 3250 \cdot \text{psf}$

Embank. slope a = 50.00(ft)
Embank. width b = 70.00(ft)
p load/unit area = 3250.00(psf)

INCREMENT OF
STRESSES FOR Z-DIRECTION
X = 70.00(ft)

Z (ft)	Vertical Stress (psf)	
0.00	1625.00	
0.20	1625.00	
0.40	1625.00	
0.60	1625.00	
0.80	1624.99	
1.00	1624.98	
1.20	1624.97	
1.40	1624.96	
1.60	1624.94	
1.80	1624.91	
2.00	1624.87	at z=2.0 ft Δσ = 1624.87 psf
2.20	1624.83	
2.40	1624.78	
2.60	1624.72	
2.80	1624.66	
3.00	1624.58	
3.20	1624.49	
3.40	1624.39	
3.60	1624.28	
3.80	1624.15	
4.00	1624.01	
4.20	1623.86	
4.40	1623.69	
4.60	1623.51	at z=4.60 ft, Δσ=1623.51 psf
4.80	1623.31	
5.00	1623.10	
5.20	1622.87	
5.40	1622.62	
5.60	1622.35	
5.80	1622.07	
6.00	1621.77	
6.20	1621.45	
6.40	1621.11	
6.60	1620.75	
6.80	1620.37	
7.00	1619.97	
7.20	1619.54	at z=7.2 ft, Δσ=1619.54 psf
7.40	1619.10	
7.60	1618.64	
7.80	1618.15	
8.00	1617.65	
8.20	1617.12	
8.40	1616.57	
8.60	1616.00	
8.80	1615.40	
9.00	1614.78	
9.20	1614.14	
9.40	1613.48	
9.60	1612.79	
9.80	1612.08	

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 = 1750 degree-days

Case I - Soils at elevation of possible footings of WC=15% and coarse grained fill or glacial till

Interpolate between frost depth of 79.95 inches at 1700 DFI and 82.3 inches at 1800 DFI

Depth of Frost Penetration =

$$d := \frac{82.3 - 79.95}{100} \cdot 60 \cdot in + 79.95 \cdot in \qquad d = 6.78 \cdot ft$$

Method 2 - ModBerg Software

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

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

--- 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	79.5	15.0	125.0	31	40	2.9	1.8	2,700

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

Total Depth of Frost Penetration = 6.62 ft = 79.5 in.

Recommend 6.5 feet for the design frost embedment of foundations 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 B

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.069	As,	Site Class B
0.2	0.148	SDs,	Site Class B
1.0	0.044	SD1,	Site Class B

MCRR Crossing Bridge, Carmel, Maine 15622.00
Date and Time: 10/6/2009 3:32:21 PM

Abutment and Wingwall Active 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 $c_1 := 0 \cdot \text{psf}$

Active Earth Pressure - Rankine Theory

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

- For cantilever walls with horizontal backslope

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

- For a sloped backfill

β = Angle of fill slope to the horizontal

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

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

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

Coulomb Theory

In general, for cases where the back face of the wall interferes with the development of a full sliding surface in the backfill, as assumed by Rankine Theory, use Coulomb.

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

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

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

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

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

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

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface.
- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface, Pa is oriented at an angle of ϕ/3 to 2/3*ϕ to the normal of a vertical line extending up from the heel of the wall