SCHONEWALD
ENGINEERING
ASSOCIATES, INC.

GEOTECHNICAL DESIGN REPORT
REPLACEMENT OF WEYMOUTH BRIDGE (BRIDGE #2934) OVER SANDY RIVER
MADRID TOWNSHIP, MAINE
MaineDOT WIN 22615.00

PREPARED FOR:

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EXECUTIVE SUMMARY

This Geotechnical Design Report presents subsurface information and provides final geotechnical design and construction recommendations for the replacement of Weymouth Bridge over the Sandy River located on Route 4 in Madrid Township, Maine (MaineDOT WIN 22615.00). The replacement bridge will be a 108-foot long, single span structure with substructure elements founded on hard bedrock; the abutments will be at a 35-degree skew with the roadway. The West Abutment (Abutment 1) will be a fixed-integral stub abutment that is supported on H-piles that are grouted into rock sockets; and the east abutment (Abutment 2) will be a semi-integral full-height abutment bearing on the prepared bedrock surface.

Subsurface conditions encountered in the test borings completed at the Weymouth Bridge site consisted of about 2 to 3 feet of Granular Fill over an approximately 5- to 10-foot thickness of reworked glacial till (Till Fill). The Till Fill was underlain by approximately 2 to 16 feet of glacial till. Bedrock underlying the glacial till consisted of Hornfels, a polymineralic massive metamorphic rock (Rock Type D). The protolith rock (pelite or mudstone) underwent alteration likely as a result of a large granitic intrusion that is mapped in close proximity to the project site. Test results indicate the uniaxial compressive strength of the hornfels ranged from 2,522 to 2,700 ksf. Based on these test results and the rock core obtained from the test borings, the Rock Mass Rating (RMR), corrected for joint orientation with respect to foundations, was determined to be 59, which is Fair to Good.

A pile-supported integral abutment bridge is not feasible at this location due to inadequate depth of overburden overlying bedrock on the east end of the proposed structure. However, the deep bedrock pocket beneath the proposed west abutment is not conducive for the construction of a full-height abutment bearing on the bedrock surface at Abutment 1. Concerns that driven piles would be overstressed during driving through the glacial till and/or would not achieve the required depth due to obstructions, thereby resulting in the need to pre-excavate the pile locations does not favor a traditional driven pile-supported west abutment. To accommodate the variable bedrock depth and address the constructability challenges related to the "boney" glacial till, the structural design calls for a mixed substructure. This consists of a drilled-pile-supported integral stub abutment on the west that is provided with riprap slope/scour protection and a full-height abutment bearing directly on a prepared bedrock surface on the east.

The structural design of the replacement bridge calls for the West Abutment (Abutment 1) to be a fixed-integral stub abutment that is supported on H-piles that are grouted into rock sockets. Bedrock is anticipated to be at approximately elevations 905 to 906 beneath the west abutment. Due to anticipated obstructions (e.g., cobbles and boulders) in the glacial till overlying the bedrock surface that could overstress driven piles, the piles should be installed in temporary casing that is driven or spun into the bedrock surface and subsequently backfilled with select backfill material as the temporary casing is withdrawn. Lateral resistance can be achieved by embedding the piles by grouting the piles into drilled rock sockets. The rock socket length should be adequate to provide fixity and to satisfy free length requirements based on the structural design. Once the minimum rock socket length is determined to achieve the structural design requirements, the design documents should call for drilling a rock socket that is at least 2 feet deeper. The rock socket length should be measured in the field from the low side of the rock, the depth at which the temporary casing is seated into bedrock around its entire circumference.

The diameter of the bedrock sockets should be at least 4 inches greater than the diagonal dimension of the specified H-pile. The completed rock sockets should be cleaned of all debris to the extent practicable. Recognizing that some debris will remain, the H-piles should be suspended above the bottom of the socket while tremie grouting. The grout strength should be at least as strong as the nominal compressive strength of the rock mass (570 ksf) or 4,000 psi.

As the temporary casing is withdrawn from the overburden, it should be backfilled with Underdrain Backfill Material (703.22) – Type C. The borrow should be tested for corrosion parameters by AASHTO test methods. The level of the backfill material should always remain above the bottom of the temporary casing as the casing is withdrawn to ensure a continuous column of specified material.

Design of the Abutment 1 pile cap should be based upon use of MaineDOT Granular Borrow backfill and consider full passive earth pressures acting on the back face of the cap. The design should use an earth pressure load factor equal to 1.50.

The footing for the full height abutment (Abutment 2) should bear directly on prepared bedrock or a mud leveling pad (seal) poured on the prepared bedrock surface. No part of the footing should bear on soil. The contractor should remove any overburden soil and weathered rock that can be removed using ordinary excavation equipment to expose competent bedrock. The prepared bearing surface should not slope steeper than 4H:1V; stepping the bearing surface to flatten the overall slope is acceptable. The final bedrock surface should be cleaned prior to pouring a seal or the footings. Dowels should be used to pin the footing or seal to the prepared bedrock surface.

The strength limit state bearing resistance was evaluated based on the Rock Mass Rating (RMR) of the bedrock underlying the site and using methods outlined by Wyllie and Hoek-Brown. A factored bearing resistance (q_r) of 250 ksf is recommended for the strength limit state design of footings bearing on rock. Regardless of the calculated bearing pressure, the minimum footing dimension should not be less than two feet. For evaluation of sliding on the base of the footings if dowels are not used to pin the footing or seal, the bearing material should conservatively be treated as a soil with the friction angle (δ) between the cast-in-place concrete footing and the soil taken as 30 degrees. A resistance factor ($\varphi\tau$) for sliding, based upon the footing bearing on sandy soil, is 0.80, in accordance with Table 10.5.5.2.2-1 of the 2017 LRFD Manual. Sliding resistance will be increased significantly if dowels are used to pin the footing or seal to the prepared bedrock surface.

The footing stems (walls) should be designed based upon use of MaineDOT Granular Borrow backfill and the geotechnical recommendations provided in this report. To provide adequate drainage, MaineDOT Granular Borrow for Underwater Backfill, which allows significantly less fines, should be used within 10 feet behind the stem wall, measured from 18 inches off the back of the footing. Granular Borrow for Underwater Backfill should extend up to or slightly above the predicted Q_{100} flood elevation.

Settlement and frost action are not concerns for abutments supported on bedrock. The stub abutment (Abutment 1) should be provided with scour protection by armoring with riprap. Because the Abutment 2 footing and/or seal will bear directly on prepared bedrock, scour protection is not a concern. However, doweling the footing or seal to bedrock should provide protection against extreme scour events. The Weymouth Bridge site is classified as a Seismic Zone 1. Seismic analysis is not required for bridges in Seismic Zone 1, however, the design of superstructure connections should consider seismic forces.

INTRODUCTION

Schonewald Engineering Associates, Inc. (SchonewaldEA) has prepared this Geotechnical Design Report for Stantec Consulting Services, Inc. (Stantec) to present subsurface information and provide final geotechnical design and construction recommendations for the replacement of Weymouth Bridge over the Sandy River located on Route 4 in Madrid Township, Maine (MaineDOT WIN 22615.00).

SchonewaldEA's work on this project has been completed under two separate contracts. Preliminary work that included the subsurface investigation and laboratory testing program, as well as development of preliminary design recommendations was completed in accordance with a project assignment under SchonewaldEA's GCA with MaineDOT. The work for MaineDOT culminated with the preparation of a Preliminary Design Memorandum for the project that was dated September 2, 2015. Final design work commenced in Fall 2016. SchonewaldEA's final design phase of work has been completed in accordance with a Subconsultant Agreement with Stantec that is dated October 25, 2016.

A quality assurance review of the technical aspects of SchonewaldEA's work was completed by Stephen J. Rabasca, P.E. of SoilMetrics, LLC located in Cape Elizabeth, Maine. This report is subject to the limitations contained in the Closure section of the report.

BACKGROUND AND PROJECT DESCRIPTION

In July 2014, SchonewaldEA was retained by MaineDOT to provide geotechnical field and design services to support the preliminary engineering for the reconstruction of an approximately 4.6-mile long segment of Route 4 in Madrid Township-Phillips. The segment of Route 4 includes two bridges over the Sandy River; Weymouth Bridge (Bridge #2934) in the Madrid Township portion of the segment and Wing Bridge (Bridge #2955) in the Phillips portion of the segment. The Weymouth Bridge replacement in Madrid Township (MaineDOT WIN 22615.00) is the subject of this geotechnical design report.

Sheet 1 – Location Map depicts the location of the Weymouth Bridge and is included with the Figures. Where Route 4 crosses the Sandy River, the river flows south to north and is at a skew of approximately 35 degrees to the roadway. Based on field observations and review of the as-built plans for the Weymouth Bridge, the existing bridge is an approximately 45-foot-long single-span structure founded on shallow spread footings with mass concrete abutment walls that was constructed around 1933. Rock was observed in the river channel at all but the northwest corner of the existing Weymouth Bridge. The 1930s project plans call out foundations on ledge, but the bridge profile included on the same sheet indicates the east abutment is founded on ledge and the west abutment is founded on coarse gravel and boulders.

SchonewaldEA understands the following about the replacement structure:

- Single, 108-foot long span with a 35-degree skew;
- Profile raise of approximately 2.5 to 3 feet;
- No horizontal alignment shift;
- Width of 30 feet curb to curb;
- Abutments located approximately 25 to 30 feet behind the existing abutments;
- West Abutment (Abutment 1) to be a fixed-integral stub abutment that is supported on H-piles that are grouted into rock sockets;
- East abutment (Abutment 2) to be a semi-integral full-height abutment bearing on bedrock;
- Temporary earth support will be required; and
- Construction-phase detour bridge will be located upstream (southerly) of the Route 4 alignment.

GEOLOGICAL SETTING

Surficial geology along the Sandy River through Madrid Township, including the project site, is mapped as glacial till and shallow bedrock (thin drift) (Surficial Geologic Map of Maine). The project site is mapped near a contact between the Smalls Falls Formation (a meta carbonaceous pelite (mud stone) of Silurian age) and the Madrid Formation (a meta interbedded sandstone and limestone of Devonian-Silurian age); and is in close proximity to a muscovite granodiorite intrusive body (Bedrock Geologic Map of Maine) that post-dates both the Smalls Falls and Madrid Formations.

SUBSURFACE INVESTIGATION

SchonewaldEA retained Maine Test Borings of Hermon, Maine to drill four test borings at the Weymouth Bridge site to evaluate subsurface conditions. The test borings were designated BB-MSR-101 through -104. The test borings were drilled on August 25 and 26, 2014 and were observed and logged by SchonewaldEA. The approximate locations of the test borings are depicted on Sheet 2 –Boring Location Plan & Interpretive Subsurface Profile that is included with the Figures.

The test borings were drilled through the approach fill behind each corner of the existing bridge abutments to ascertain the depth to and the engineering characteristics of the bedrock underlying the proposed abutments. The test borings were advanced using standard cased wash boring techniques. Each boring was extended through overburden to refusal and 10 feet of NQ2 (N-size, double-barrel core barrel) rock core was obtained. Standard Penetration Tests (SPTs) were completed and split-spoon soil samples obtained near the ground surface and at five-foot intervals to the bottom of the overburden. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration is recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. It is customary for the raw field N-values to be corrected based on the relative energy of the actual hammer system utilized to complete the SPTs. The corrected N-values are referred to as N_{60} values, which are used in correlations and analyses to evaluate the engineering characteristics of the overburden soils. SPTs for this project were conducted using a rope and cathead, which is considered the “standard” hammer system resulting in no correction needed to obtain N_{60}. In other words, N_{60} is equal to the raw field N-value.

Logs of the subsurface explorations are included as Appendix A. Note that the logs contained in Appendix A are formatted for an 8-1/2-inch by 14-inch page size so that each log will fit on a single page. Photographs of the rock core collected from the Weymouth Bridge test borings are included as Appendix B.

LABORATORY TESTING

Representative specimens of the rock core obtained in the test borings were submitted to the Thielsch Engineering geotechnical laboratory in Cranston, Rhode Island for unconfined (uniaxial) compressive strength testing. The unconfined compression tests on rock were conducted in accordance with ASTM D-7012 Method D (Elastic Moduli in Uniaxial Compression) and included specimen preparation per ASTM D-4543, measuring strength, developing a stress-strain curve, reporting elastic moduli (Young’s Modulus and Poisson’s Ratio), and providing before and after photographs of the test specimens. The laboratory testing program is summarized in the following table.

Boring No.	Sample No.	Specimen Depth	Sample Representative of: Tests Performed:
BB-MSR-101	R1	12.0 to 12.8 ft. BGS	bedrock; uniaxial compressive strength test
BB-MSR-104	R1	32.0 to 32.6 ft. BGS	bedrock; uniaxial compressive strength test

Subsurface conditions, including the results of the rock testing, are discussed in the following section. Laboratory test results are summarized on the test boring logs included in Appendix A and the laboratory test report is included as Appendix C.

GEOPHYSICAL SURVEY

MaineDOT retained Hager-Richter Geoscience, Inc. (HRG) to complete a geophysical survey of the approaches to the Weymouth Bridge to obtain additional information regarding bedrock depths. HRG conducted seismic refraction profiling along two lines, one in each lane of Route 4 that were centered on the existing bridge and extended approximately 115 feet behind each abutment. HRG also conducted a ground penetrating radar (GPR) survey of an approximately 425-foot by 25-foot area centered on the bridge. HRG completed their field work on September 9, 2014. A copy of HRG's report is included as Appendix D.

The locations of the seismic refraction transects are depicted on Figure 2 in HRG's report. HRG's interpretation of the topography of the top of the bedrock surface in the study area is depicted on Figure 4 in their report. Figure 4 is a color contour plot of the bedrock surface topography model generated from the seismic refraction survey and the logs for nearby borings provided to HRG by SchonewaldEA. The contours shown on Figure 4 represent interpolations based on the seismic refraction data and available boring information relative to NAVD88. Based on comparing the seismically determined elevations with refusal information from nearby borings, and on the results from other similar seismic refraction surveys, HRG estimates the accuracy (standard deviation) of the depths of competent bedrock determined by the seismic refraction survey to be about ± 10 percent of the depth of bedrock or ± 2 feet, whichever is greater.

SchonewaldEA notes that the test boring logs that are included in the HRG report were preliminary in nature and provided to HRG to aid them in the field; the test borings logs provided in the HRG report have been superseded by the test boring logs included in Appendix A of this geotechnical design report.

SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings completed at the Weymouth Bridge site consisted of about 2 to 3 feet of Granular Fill over an approximately 5- to 10-foot thickness of reworked glacial till (Till Fill). The Till Fill was underlain by approximately 2 to 16 feet of glacial till. Bedrock underlying the glacial till consisted of Hornfels of fair to good quality. An Interpretive Subsurface Profile is included on Sheet 2 in the Figures.

Granular Fill: All of the test borings encountered Granular Fill below pavement. The Granular Fill was typically medium dense, gravelly fine to coarse sand, with varying amounts of silt. The bottom of the Granular Fill was observed to range from 2.5 to 2.9 feet Below the Ground Surface (BGS).

Till Fill: All of the test borings encountered reworked glacial till below the Granular Fill. The Till Fill was typically loose, fine to medium sand, with varying amounts of gravel and silt. The bottom of the Till Fill was observed to range from 7.5 feet BGS behind the east abutment to about 12 feet BGS behind the west abutment. It is likely that glacial till excavated for the construction of the highway was reused as fill behind the bridge abutments, the loose in-situ relative density being the result of standard construction practices and available compaction equipment at the time the existing bridge was constructed around 1933.

Glacial Till: Glacial Till was encountered in all of the test borings below the fill materials. The Glacial Till was typically dense to very dense, gravel, with varying amounts of sand and silt. Drilling behavior suggested that large gravel and cobbles were present throughout the glacial till stratum. The Glacial Till varied from about 2 to 4 feet thick behind the east abutment to approximately 15 to 17 feet thick behind the west abutment.

Detailed descriptions of the soils encountered in the test borings are provided on the logs included in Appendix A.

Bedrock: Bedrock was encountered at depths ranging from 9.5 and 11.3 feet BGS (elevations 923.6 and 921.6 feet, respectively) behind the east abutment. Bedrock was encountered at depths ranging from 27.2 and 27.6 feet BGS (elevations 905.8 and 905.3 feet, respectively) behind the west abutment. The bedrock core obtained in the test borings consisted of hard, typically fresh, aphanitic to fine grained, light grey HORNFELS, a polymineralic massive metamorphic rock (Rock Type D). The protolith rock (pelite or mudstone) underwent alteration likely as a result of a large granitic intrusion that is mapped in close proximity to the project site.

The Rock Quality Designations (RQDs) of the rock cores obtained in the test borings located behind the east abutment ranged from 47 to 93 percent. The RQDs of the rock cores obtained in the test borings located behind the west abutment ranged from 77 to 100 percent. Two specimens of the rock core were submitted for uniaxial compressive strength tests. The test results indicate the uniaxial compressive strength of the HORNFELS ranged from 2,522 to 2,700 ksf. Based on these test results and the rock core obtained from the test borings, the Rock Mass Rating (RMR), corrected for joint orientation with respect to foundations, was determined to be 59, which is Fair to Good.

Detailed descriptions of the rock encountered in the test borings are provided on the logs included in Appendix A.

The interpreted bedrock surface elevation contours presented by HRG based on their geophysical survey are consistent with the test boring data at those points and generally show rock is lowest/ deepest behind the existing west abutment. Reference is made to Figure 4 – Bedrock Elevation in the HRG report that is provided as Appendix D.

IMPLICATIONS OF SUBSURFACE CONDITIONS

Dense Glacial Till Deposit: The glacial till was observed to contain large gravel, cobbles, and boulders that was difficult to penetrate with the casing and the action of a roller cone.

Variable Bedrock Depth: Bedrock was encountered at depths ranging from 9.5 and 11.3 feet BGS (elevations 923.6 and 921.6 feet, respectively) behind the east abutment and at depths ranging from 27.2 and 27.6 feet BGS (elevations 905.8 and 905.3 feet, respectively) behind the west abutment. The bedrock surface is significantly deeper under the west end of the bridge.

A pile-supported integral abutment bridge is not feasible at this location due to inadequate depth of overburden overlying bedrock on the east end of the proposed structure (MaineDOT's Bridge Design Guide dated August 2003 together with March 2014 updates (MaineDOT BDG)). However, the deep bedrock pocket beneath the proposed west abutment is not conducive for the construction of a full-height abutment bearing on the bedrock surface at Abutment 1. Concerns that driven piles would be overstressed during driving through the glacial till and/or would not achieve the required depth due to obstructions, thereby resulting in the need to pre-excavate the pile locations does not favor a traditional driven pile-supported west abutment. To accommodate the variable bedrock depth and address the constructability challenges related to the “boney” glacial till, the structural design calls for a mixed substructure. This consists of a drilled-pile-supported integral stub abutment on the west that is provided with riprap slope/scour protection and a full-height abutment bearing directly on a prepared bedrock surface on the east.

GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

SchonewaldEA provides the following geotechnical recommendations for the design and construction of the Weymouth Bridge replacement. These recommendations are based on the MaineDOT BDG, as well as the geotechnical provisions set forth in the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 (2010 LRFD manual) and the 8th Edition, 2017 (2017 LRFD manual). In summary, the replacement bridge will be a 108-foot long, single span structure with substructure elements founded on hard bedrock; the abutments will be at a 35-degree skew with the roadway. The West Abutment (Abutment 1) will be a fixed-integral stub abutment that is supported on H-piles that are grouted into rock sockets; and the east abutment (Abutment 2) will be a semi-integral full-height abutment bearing on the prepared bedrock surface.

WEST ABUTMENT (ABUTMENT 1) – DRILLED-PILE-SUPPORTED STUB ABUTMENT

The structural design of the replacement bridge calls for the West Abutment (Abutment 1) to be a fixed-integral stub abutment that is supported on H-piles that are grouted into rock sockets. Bedrock is anticipated to be at approximately elevations 905 to 906 beneath the west abutment. Due to anticipated obstructions (e.g., cobbles and boulders) in the glacial till overlying the bedrock surface that could overstress driven piles, the piles should be installed in temporary casing that is driven or spun into the bedrock surface and subsequently backfilled with select backfill material as the temporary casing is withdrawn. Lateral resistance can be achieved by embedding the piles by grouting the piles into drilled rock sockets.

The nominal resistance of piles bearing on hard rock, like the hornfels underlying the Weymouth Bridge site, will be controlled by structural limit state (structural capacity), not geotechnical capacity. Piles should be designed to be end bearing on rock. Skin friction achieved on the rock socket side wall may be neglected in this application since geotechnical capacity does not control and capacity in compression is adequate without accounting for skin friction. The maximum factored applied axial load should not exceed the resistance values determined in accordance with Section 6.9.4.1 of the LRFD Manual using the resistance factors set forth in Section 6.5.4.2.

Lateral resistance can be achieved by embedding the piles by grouting them into drilled rock sockets to create a fixed-tip pile that can neither rotate or translate. Pile size should be selected from the table of typical H-pile sizes that is included in Section 5.4.2.1 of the MaineDOT BDG if possible, keeping in mind that a smaller section will require smaller diameter rock sockets. Horizontal deflection of the piles should be evaluated using software such as LPILE.

To model the lateral soil resistance of the overburden soil and bedrock encountered underlying the west abutment of Weymouth Bridge, SchonewaldEA recommends the following LPILE geotechnical input:

LAYER	DESCRIPTION	p-y CURVE MODEL	PARAMETERS (assign top and bottom)
1	12-ft thick layer GRANULAR FILL/ TILL FILL Elevation 933 to 921 ft	REESE SAND	Effective unit weight = 110 pcf Friction angle = 29 deg Use default p-y moduli K
2	15.5-ft thick layer GLACIAL TILL Elevation 921 to 905.5 ft	REESE SAND	Effective unit weight = 68 pcf Friction angle = 36 deg Use default p-y moduli K
3	HARD ROCK (HORNFELS) Elevation 905.5 to bottom of LPILE model	VUGGY LIMESTONE	Effective unit weight = 169 pcf Uniaxial compressive strength = 17,500 psi

The above recommended geotechnical input was based on test borings BB-MSR-103 and -104, as well as results of uniaxial compressive strength testing of rock core specimens. The Weymouth Bridge site is not underlain by soils that require modeling by way of the development of soil-specific p-y curves.

If the structural design requires more rigorous modeling (e.g., the use of a finite element soil-structure model (FEM)), SchonewaldEA recommends the same p-y curves used in the LPILE analysis be used to back-calculate spring constants for use in the FEM. It is SchonewaldEA's experience that FEM predictions are not particularly sensitive to geotechnical input, namely spring constants, but suggests that a quick parametric study will speak to whether more rigorous geotechnical input is warranted.

The rock socket length should be adequate to provide fixity and to satisfy free length requirements based on the structural design. Once the minimum rock socket length is determined to achieve the structural design requirements (e.g., fixity and free length), the design documents should call for drilling a rock socket that is at least 2 feet deeper. The rock socket length should be measured in the field from the low side of the rock, the depth at which the temporary casing is seated into bedrock around its entire circumference.

The diameter of the bedrock sockets should be at least 4 inches greater than the diagonal dimension of the specified H-pile. The rock sockets should be drilled using rotary down hole hammers, rotary percussive methods, or solid rock coring techniques. The completed rock sockets should be cleaned of all debris to the extent practicable. Recognizing that some debris will remain, the H-piles should be suspended above the bottom of the socket while tremie grouting. The grout strength should be at least as strong as the nominal compressive strength of the rock mass (570 ksf) or 4,000 psi.

As the temporary casing is withdrawn from the overburden, it should be backfilled with Underdrain Backfill Material (703.22) – Type C. The borrow should be tested for corrosion parameters by AASHTO test methods (AASHTO T288 – soil resistivity; T289 – pH, T290 – soluble sulfates in soil; and T291 – soluble chlorides in soil). The level of the backfill material should always remain above the bottom of the temporary casing as the casing is withdrawn to ensure a continuous column of the specified material.

EAST ABUTMENT (ABUTMENT 2) – FULL-HEIGHT ABUTMENT WITH FOOTING ON ROCK

The structural design of the replacement bridge calls for the East Abutment (Abutment 2) to be a semi-integral full-height abutment bearing on bedrock. The footing for the full-height abutment should bear directly on prepared bedrock or a mud leveling pad (seal) poured on the prepared bedrock surface. No part of the footings should bear on soil. The contractor should remove any overburden soil and weathered rock that can be removed using ordinary excavation equipment to expose competent bedrock. The prepared bearing surface should not slope steeper than 4H:1V; stepping the bearing surface to flatten the overall slope is acceptable. In accordance with Section 5.3.1.2 of MaineDOT's BDG, the final bedrock surface should be cleaned prior to pouring a seal or the footings. The prepared bedrock surface should be inspected and accepted by the Resident prior to any concrete placement. Dowels should be used to pin the footing or seal to the prepared bedrock surface.

For service limit state only, a presumptive bearing resistance equal to 160 ksf may be used for the design of footings bearing on prepared bedrock. This is anticipated to result in total (immediate) settlements less than one inch. For the service limit state structural design, use the lesser of this recommended bearing resistance or the nominal resistance of concrete ($0.3f'c$).

For the evaluation of strength limit state bearing resistance, SchonewaldEA evaluated the Rock Mass Rating (RMR) of the bedrock underlying the site using methodology developed by Hoek and Brown and outlined in the 2010 LRFD Manual and based on rock core descriptions and laboratory unconfined compressive strength test data. Methods outlined by Wyllie and Hoek-Brown were then used to develop recommendations for the strength limit state bearing resistance. A nominal bearing resistance (q_n) on rock equal to 570 ksf is reasonable. Using a bearing resistance factor (φ_b) of 0.45 for spread footings on rock in accordance with Table 10.5.5.2.2-1 in the 2017 LRFD Manual, a factored bearing resistance (q_r) of 250 ksf is recommended for the strength limit state design of footings bearing on rock. Regardless of the calculated bearing pressure, the minimum footing dimension should not be less than two feet. Rock Mass Rating and Rock Mass Bearing Capacity calculations are provided as Appendix E.

For evaluation of sliding on the base of the footings if dowels are not used to pin the footing or seal, the bearing material should conservatively be treated as a soil with the friction angle (δ) between the cast-in-place concrete footing and the soil taken as 30 degrees. This is in accordance with Table C3.11.5.3-1 in the 2017 LRFD Manual. A resistance factor ($\varphi\tau$) for sliding, based upon the footing bearing on sandy soil is 0.80, in accordance with Table 10.5.5.2.2-1 of the 2017 LRFD Manual. Sliding resistance will be increased significantly if dowels are used to pin the footing or seal to the prepared bedrock surface.

INTEGRAL ABUTMENT 1 CAP DESIGN

Design of the Abutment 1 pile cap should be designed based upon use of MaineDOT Granular Borrow backfill and should consider full passive earth pressures acting on the back face of the cap in accordance with the MaineDOT BDG Section 5.4.2.11. The design should evaluate passive earth pressures using Coulomb methods if adequate rotation is achieved to fully mobilize passive pressures, otherwise Rankine methods for evaluating passive earth pressures should be used. An earth pressure load factor equal to 1.50 should be used for structural design of the Abutment 1 cap. Passive earth pressures are discussed in more detail in the subsequent section.

The pile cap should be provided with drainage/strip drains and weep holes to limit hydrostatic pressures.

SEMI-INTEGRAL ABUTMENT 2 STEM WALL DESIGN

The Abutment 2 footing stem (wall) should be designed based upon use of MaineDOT Granular Borrow backfill, having the following design parameters:

- Internal friction angle (φ) equal to 32 degrees;
- Total unit weight (γ_t) equal to 0.125 kips per cubic foot (kcf);
- Rankine active earth pressure coefficient (K_a) equal to 0.31 assuming the backfill behind the cap/ end diaphragm is level;

- Coulomb passive earth pressure coefficient ($K_{p\text{-Coulomb}}$) equal to 8.38 assuming the backfill behind the cap/ end diaphragm is level and stem wall rotation is sufficient to mobilize full passive pressure; and
- If stem wall rotation in not sufficient to mobilize full passive pressure, as discussed below, a Rankine passive earth pressure coefficient ($K_{p\text{-Rankine}}$) equal to 3.25 assuming the backfill behind the end diaphragm/ cap is level.

The Abutment 2 footing stem (wall) should be designed for active earth pressure over the rigid abutment height and using a uniform pressure distribution to account for the height of soil behind the superstructure/ end diaphragm. In designing for active pressure, use of the Rankine active earth pressure coefficient noted above is recommended.

The superstructure backwall should be designed for full passive pressure using the Coulomb passive pressure coefficient noted above if the calculated wall rotation is sufficient to mobilize full passive pressure. To mobilize full passive pressure, the ratio of lateral movement to the height of the backwall (y/H) must exceed 0.005. If the calculated ratio is less than 0.005, then the Rankine passive pressure coefficient noted above should be used. A maximum load factor equal to 1.50 may be used to calculate factored passive earth pressures.

To provide adequate drainage, MaineDOT Granular Borrow for Underwater Backfill, which allows significantly less fines, should be used within 10 feet behind the stem wall, measured from 18 inches off the back of the footing. Granular Borrow for Underwater Backfill should extend up to or slightly above the predicted Q_{100} flood elevation. Additionally, footing drains, drainage/strip drains, and weep holes should be provided to limit hydrostatic pressures. Groundwater should be set at a level three feet up the stem in accordance with the MaineDOT BDG regardless of the groundwater controls provided.

SETTLEMENT

For abutments supported on bedrock, settlement is not a concern.

FROST ACTION

Likewise, frost action is not a concern for abutments supported on bedrock.

SCOUR PROTECTION

The stub abutment should be provided with scour protection by armoring with riprap. Specifically, scour protection should consist of a 3-foot thickness of riprap conforming to MaineDOT Standard Specification Section 703.26 Plain and Hand Laid Riprap. Slopes shall be no steeper than 2H:1V to the extent practicable. The riprap on the slopes shall be underlain by a non-woven, Class 1 Erosion Control Geotextile meeting the requirements of MaineDOT Standard Specification 722.03 that is underlain by a 1-foot thick layer of bedding material consisting of Granular Borrow Material for Underwater Backfill (703.19). The toe of the riprap sections shall be keyed into the existing soils 1 foot below the streambed elevation.

Scour protection is not a concern for the full-height abutment supported on a footing bearing on a prepared bedrock surface, though doweling the footing or seal to bedrock should provide protection against extreme scour events.

SEISMIC DESIGN CONSIDERATIONS

The following seismic parameters for the Weymouth Bridge site were developed using the general procedure for determining seismic hazard that is set forth in Articles 3.10.1.2 and 3.10.4 of the 2017 LRFD, and assigning a Site Class of "A" based on related Table 3.10.3.1-1.

Peak Ground Acceleration Coefficient	PGA	0.08g	Fig 3.10.2.1-1
Short-Period Spectral Acceleration Coefficient	S_s	0.17g	Fig 3.10.2.1-2
Long-Period Spectral Acceleration Coefficient	S_1	0.05g	Fig 3.10.2.1-3
Site Factor with Site Class A	F_v	0.8	Table 3.10.3.2-3
Site Factor with Site Class A	F_{pga}	0.8	Table 3.10.3.2-1
Seismic Zone Determination ($S_{D1} = F_v$*S_1)	S_{D1}	0.04	Eq. 3.10.4.2-6
Acceleration Coefficient ($A_s = F_{pga}$*PGA)	A_s	0.06	Eq. 3.10.4.2-2

With S_{D1} less than 0.15, the Weymouth Bridge site is classified as Seismic Zone 1 in accordance with Table 3.10.6-1 of the 2017 LRFD Manual. Per Article 4.7.4 of the 2017 LRFD Manual, seismic analysis is not required for bridges in Seismic Zone 1. However, the design of superstructure connections should consider seismic forces.

CONSTRUCTION CONSIDERATIONS

Construction will require soil excavation. Earth support systems shall be implemented if laying back slopes is not feasible. Regardless of the method of excavation, all excavations and earth support systems/cofferdams shall meet all applicable OSHA regulations. The Contractor's design of earth support systems shall account for shallow bedrock conditions and the possibility of encountering obstructions in the fill materials and underlying glacial till. The design of temporary earth support systems should consider internal bracing and/or rock sockets.

The specified rock sockets should be at least 2 feet deeper than the length required to achieve the structural design requirements. The rock socket length should be measured in the field from the low side of the rock, the depth at which the temporary casing is seated into bedrock around its entire circumference.

The diameter of the bedrock sockets should be at least 4 inches greater than the diagonal dimension of the specified H-pile. The rock sockets should be drilled using rotary down hole hammers, rotary percussive methods, or solid rock coring techniques. The completed rock sockets should be cleaned of all debris to the extent practicable. Recognizing that some debris will remain, the H-piles should be suspended above the bottom of the socket while tremie grouting. The grout strength should be at least as strong as the nominal compressive strength of the rock mass (570 ksf) or 4,000 psi.

As the temporary casing is withdrawn from the overburden, it should be backfilled with Underdrain Backfill Material (703.22) – Type C. The borrow should be tested for corrosion parameters by AASHTO test methods (AASHTO T288 – soil resistivity; T289 – pH, T290 – soluble sulfates in soil; and T291 – soluble chlorides in soil). The level of the backfill material should always remain above the bottom of the temporary casing as the casing is withdrawn to ensure a continuous column of the specified material.

The stub abutment should be provided with scour protection by armoring with riprap. Specifically, scour protection should consist of a 3-foot thickness of riprap conforming to MaineDOT Standard Specification Section 703.26 Plain and Hand Laid Riprap. Slopes shall be no steeper than 2H:1V to the extent practicable. The riprap on the slopes shall be underlain by a non-woven, Class 1 Erosion Control Geotextile meeting the requirements of MaineDOT Standard Specification 722.03 that is underlain by a 1-foot thick layer of bedding material consisting of Granular Borrow Material for Underwater Backfill

(703.19). The toe of the riprap sections shall be keyed into the existing soils 1 foot below the streambed elevation.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. The Contractor shall control such water, as well as groundwater infiltration from the overburden and surface water run-on using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water to allow construction in the dry.

If soil-filled pockets are encountered at the bearing elevations, the soil shall be removed. The contractor shall also remove any weathered bedrock that can be removed using ordinary excavation equipment to expose competent bedrock at the required elevations. The bedrock bearing surface shall be clean and free of debris, soil or loose rock.

If blasting is necessary and allowed per environmental permits, it shall be preapproved by the resident and shall be conducted in accordance with section 105.2.7 of the MaineDOT standard specifications. The contractor shall conduct pre-and post-blast surveys, as well as blast vibration monitoring, at any nearby structures in accordance with industry standards at the time of the blast.

The design of the foundations for the temporary detour bridge should also account for shallow bedrock and the possibility of encountering obstructions in the overburden soils.

CLOSURE

This report has been prepared for the use of Stantec Consulting Services, Inc. for specific application to the replacement of Weymouth Bridge over the Sandy River that is located on Route 4 in Madrid Township, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

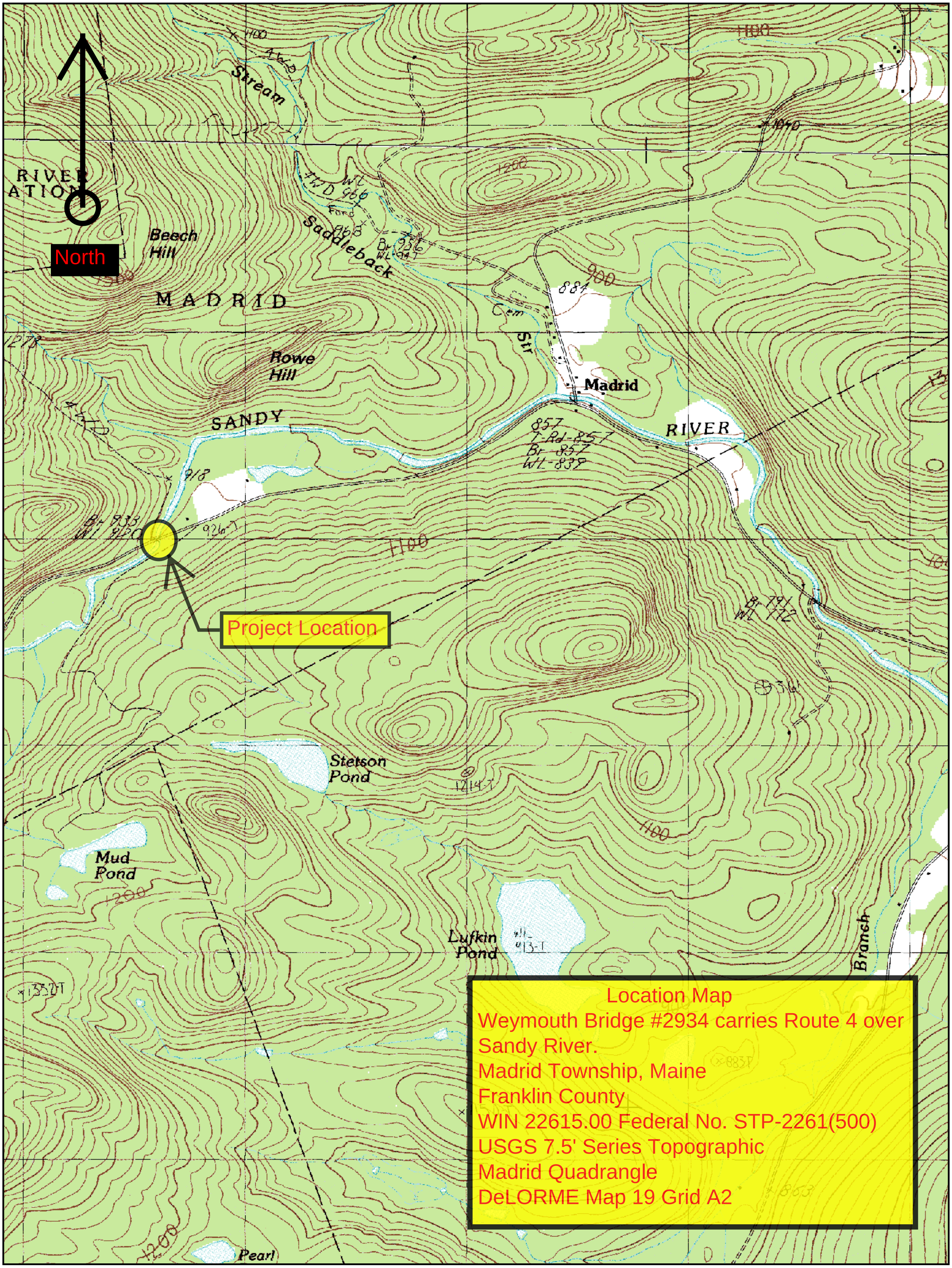
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon a limited subsurface investigation at discrete exploratory 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.

It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

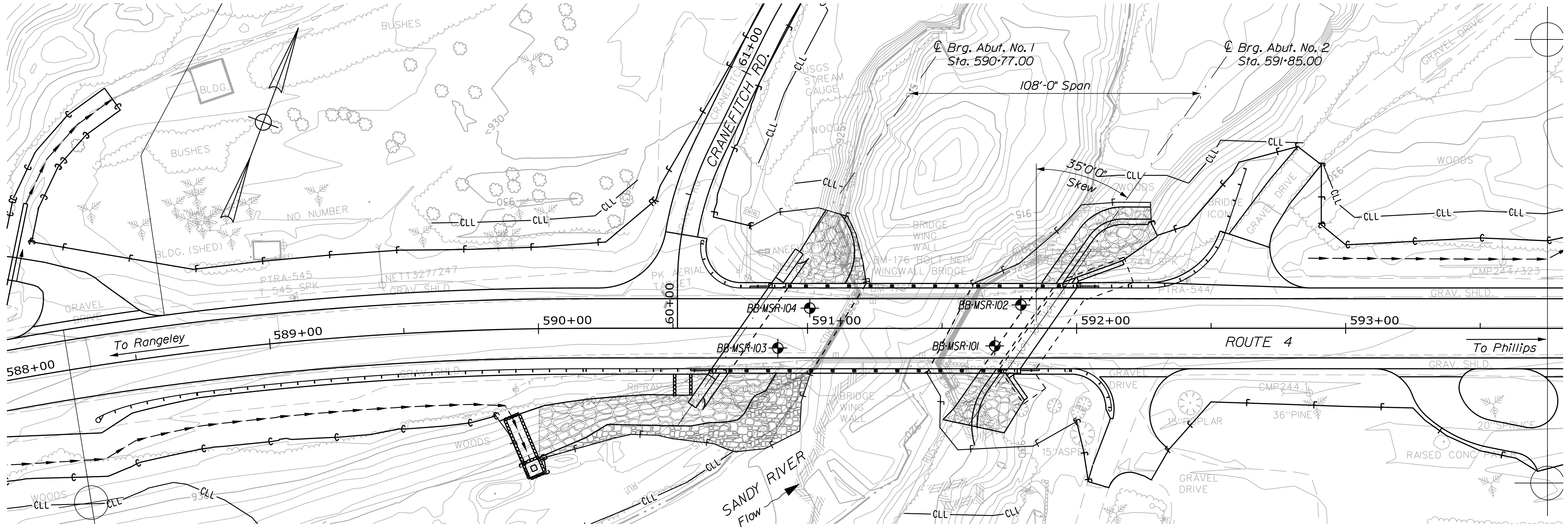
SCHONEWALD ENGINEERING ASSOCIATES, INC.

FIGURES

SHEETS 1 AND 2



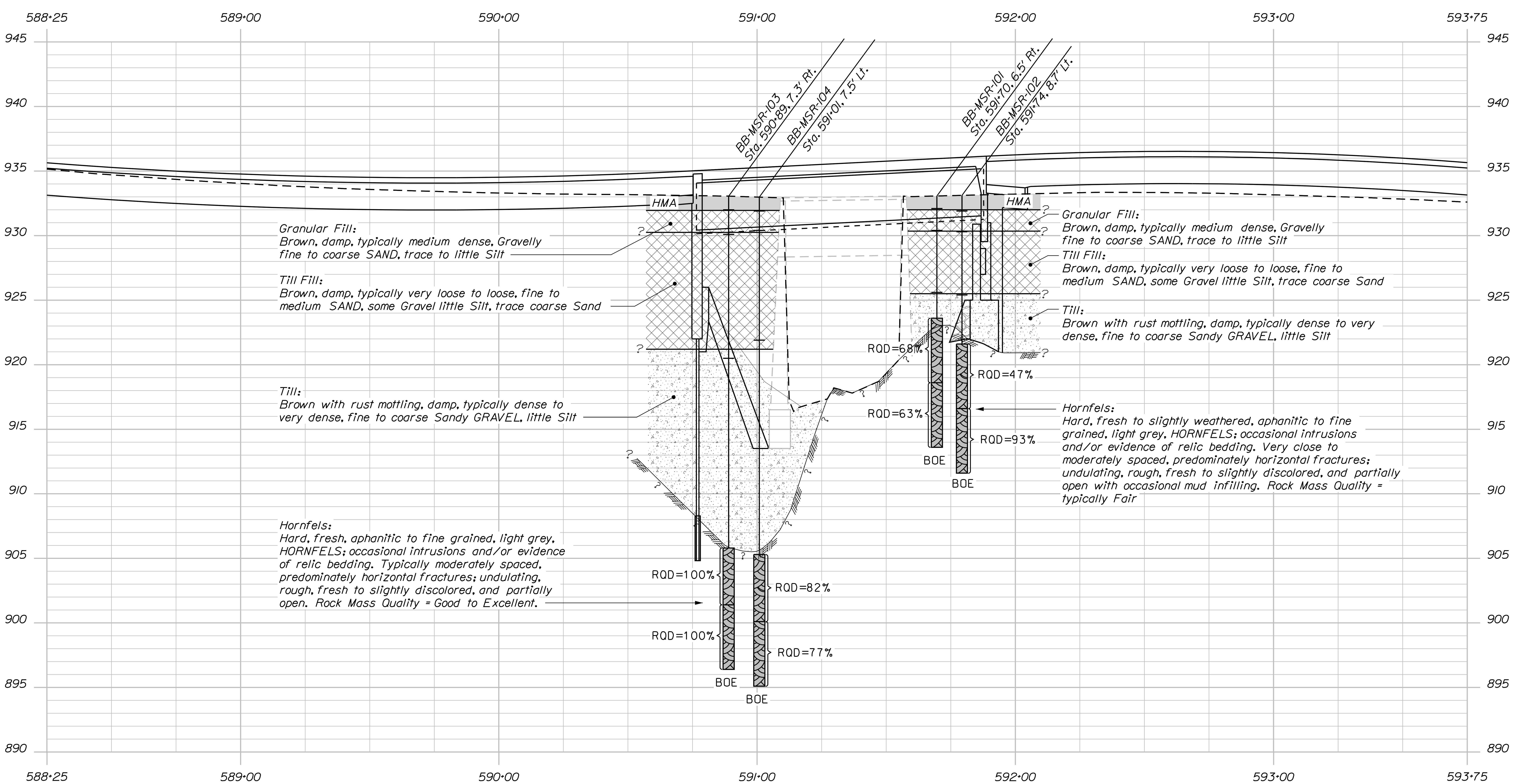
Map Scale 1:24000



BORING LOCATION PLAN

25 0 25 50
Scale of Feet

LEGEND
CASED WASH BORING



NOTES:

1. 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 and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

2. The approximate top of bedrock depicted on this Interperative Subsurface Profile was inferred from subsurface information in widely spaced explorations, as well as the result of a geophysical survey. A copy of the geophysical survey report is included as an appendix to the project geotechnical report.

LEGEND

Weathered Bedrock, if applicable
Boring No. Offset, if shown
Pavement Thickness, if applicable
Approximate Top of Bedrock
RQD= Rock Quality Designation of Bedrock Core Sample
BOE= Bottom of Exploration

INTERPRETIVE SUBSURFACE PROFILE

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2261(500)

WIN 22615.00

BRIDGE PLANS

BRIDGE NO. 2934

	M. WIGHT	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	IVS	DB	MAY 2018
CHECKED-REVIEWED	IVS	IVS	MAY 2018
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

WEYMOUTH BRIDGE
SANDY RIVER
MADRID FRANKLIN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 2

Filename: ...\MSTA419_BLP-ISP_MAD_01.dgn Division: BRIDGE Username: dataylor Date:12/14/2018

SCHONEWALD ENGINEERING ASSOCIATES, INC.

APPENDIX A

SUBSURFACE EXPLORATION LOGS

SCHONEWALD ENGINEERING ASSOCIATES, INC.	PROJECT: Weymouth Bridge Route 4 over Sandy River LOCATION: Madrid Township, Maine	Boring No.: BB-MSR-101 WIN: 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	933.1	**Auger ID/OD:**	solid stem auger to 9.5 ft.
Operator:	Enos/Dube	**Datum:**	NAVD88	**Sampler:**	standard split-spoon
Logged By:	Schonewald	**Rig Type:**	Mobile Drill B-51	**Hammer Wt./Fall:**	rope and cathead; 140#/30"
Date Start/Finish:	8/25/14; 1000 / 8/25/14; 1220	**Drilling Method:**	cased wash boring	**Core Barrel:**	NQ2
Boring Location:	Sta 591+70, 6.5 RT	**Casing ID/OD:**	NW (3") - spun	**Water Level*:**	

IN-SITU SAMPLING AND TESTING:	ADDITIONAL DEFINITIONS:	BOREHOLE ADVANCEMENT METHOD:
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	SSA= solid stem auger / RC=roller cone
MD = Unsuccessful Split Spoon Sample attempt	R = Rock Core Sample	**LABORATORY TEST RESULTS:**
U = Thin Wall Tube Sample	RQD = Rock Quality Designation (%)	LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
MU = Unsuccessful Thin Wall Tube Sample attempt	WOH = weight of 140lb. hammer	WC = water content, percent
V = Insitu Vane Shear Test	WOR = weight of rods	-#200 = percent fines from grain size analysis
MV = Unsuccessful Insitu Vane Shear Test attempt	-- = not recorded	UCT qp = peak compressive strength of rock

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
0						SSA			12.5 inches HMA	
							932.1		1.0	
	1D	24/13	1.0 - 3.0	13-14-11-9	25				1D: Brown, damp, m. dense, Gravelly fine to coarse SAND, trace to little Silt (GRANULAR FILL);	
							930.4		2.7	
									Changing at 2.7 ft. to brown, damp, Gravelly fine to medium SAND, little Silt, trace coarse Sand (TILL FILL).	
5	2D	24/13	4.0 - 6.0	6-9-4-3	13				2D: Brown, damp, m. dense, Gravelly fine to medium SAND, little Silt, trace coarse Sand (TILL FILL).	
							925.6		7.5	
									8.8 ft.: Augering becomes difficult, likely due to cobbles.	
	3D	3/3	9.2 - 9.5	20/3" (bounce)			923.6		3D: Brown changing to grey, damp changing to dry, fine Sandy GRAVEL, little Silt, trace medium to coarse SAND. (TILL changing to WEATHERED ROCK).	
10	R1	60/60	9.5 - 14.5	RQD = 68%					9.5	
									R1: Hard, fresh to slightly weathered, aphanitic to fine grained, light grey, HORNFELS; little evidence of relic bedding. Close to moderately spaced, predominately horizontal fractures; undulating, rough, fresh to slightly discolored, and partially open with occasional mud infilling. Core times: 1:00, 0:50, 1:05, 0:55, 1:00 min:sec/ft.	
			12.0 - 12.8	SAMPLE						UCT qp = 18,751 psi
15	R2	60/57	14.5 - 19.5	RQD = 63%					R2: Same as R1, except very close fractures from 14.5 to 14.8 feet and 19.0 to 19.5 feet. Core times: 1:00, 0: 55, 0:55, 0:55, 0:55 min:sec/ft.	
							913.6		19.5	
20									**Bottom of Exploration at 19.5 feet below ground surface.**	
25										
30										
35										

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.

Schonewald Engineering Associates, Inc.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid Township, Maine	**Boring No.:** BB-MSR-102 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	932.9	**Auger ID/OD:**	solid stem auger to 9.0 ft.
Operator:	Enos/Dube	**Datum:**	NAVD88	**Sampler:**	standard split-spoon
Logged By:	Schonewald	**Rig Type:**	Mobile Drill B-51	**Hammer Wt./Fall:**	rope and cathead; 140#/30"
Date Start/Finish:	8/25/14; 1240 / 8/25/14; 1445	**Drilling Method:**	cased wash boring	**Core Barrel:**	NQ2
Boring Location:	Sta 591+79, 8.7 LT	**Casing ID/OD:**	NW (3") - spun	**Water Level*:**	

IN-SITU SAMPLING AND TESTING:
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
MV = Unsuccessful Insitu Vane Shear Test attempt

ADDITIONAL DEFINITIONS:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOH = weight of 140lb. hammer
WOR = weight of rods
-- = not recorded

BOREHOLE ADVANCEMENT METHOD:
SSA= solid stem auger / RC=roller cone

LABORATORY TEST RESULTS:
LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
0						SSA			12 inches HMA	
	1D	24/13	1.0 - 3.0	9-7-4-3	11		931.9		1.0 — 1D: Brown, damp, m. dense, Gravelly fine to coarse SAND, trace to little Silt (GRANULAR FILL);	
							930.3		2.6 — Changing at 2.6 ft. to brown, damp, fine to medium SAND, some Gravel, little Silt, trace coarse Sand (TILL FILL).	
	2D	24/13	4.0 - 6.0	2-3-2-3	5				2D: Brown, damp, loose, fine to medium SAND, some Gravel, little Silt, trace coarse Sand (TILL FILL).	
5										
							925.4		7.5	
	3D	22/11	9.0 - 10.8	2-2-3-15/4"	5				3D: Brown with rust mottling, damp, loose, fine to medium SAND, some Gravel, little Silt, trace coarse Sand (TILL). Broken rock in tip of spoon.	
10										
	R1	60/59	11.3 - 16.3	RQD = 47%			921.6		11.3 — R1: Hard, fresh to slightly weathered, aphanitic to fine grained, light grey, HORNFELS; little evidence of relic bedding. Very close to moderately spaced, low and high angle fractures; undulating, rough, typically slightly discolored, and open to moderately wide with occasional mud infilling. Very close fracture spacing from 12.8 to 13.6 feet and from 14.5 to 15.0 feet. Core times: 1:10, 1:00, 1:00, 1:15, 1:20 min:sec/ft.	
15										
	R2	60/60	16.3 - 21.3	RQD = 93%					R2: Same as R1, except moderately spaced fractures; 3 horizontal and 1 moderately dipping; no infilling. Core times: 1:10, 1:00, 1:00, 1:10, 1:05 min:sec/ft.	
20										
							911.6		21.3 — **Bottom of Exploration at 21.3 feet below ground surface.**	
25										
30										
35										

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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid Township, Maine	**Boring No.:** BB-MSR-103 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	933.0	**Auger ID/OD:**	solid stem auger to 9.0 ft.
Operator:	Enos/Dube	**Datum:**	NAVD88	**Sampler:**	standard split-spoon
Logged By:	Schonewald	**Rig Type:**	Mobile Drill B-51	**Hammer Wt./Fall:**	rope and cathead; 140#/30"
Date Start/Finish:	8/25/14; 1455 / 8/26/14; 1030	**Drilling Method:**	cased wash boring	**Core Barrel:**	NQ2
Boring Location:	Sta 590+89, 7.3 RT	**Casing ID/OD:**	NW (3") - driven	**Water Level*:**	

IN-SITU SAMPLING AND TESTING:
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
MV = Unsuccessful Insitu Vane Shear Test attempt

ADDITIONAL DEFINITIONS:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOH = weight of 140lb. hammer
WOR = weight of rods
-- = not recorded

BOREHOLE ADVANCEMENT METHOD:
SSA= solid stem auger / RC=roller cone

LABORATORY TEST RESULTS:
LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
0						SSA			14 inches HMA	
	1D	24/14	1.0 - 3.0	10-9-7-4	16		932.0		1.0 — 1D: Black changing to brown, damp, m. dense, fine to medium SAND, some Gravel, trace to little Silt, trace coarse Sand (GRANULAR FILL);	
							930.1		2.9 — Changing at 2.9 ft. to brown, damp, fine to medium SAND, little Gravel, little Silt (TILL FILL).	
	2D	24/14	4.0 - 6.0	2-1-2-1	3				2D: Brown, damp, v. loose, fine to medium SAND, little to some Gravel, little Silt, trace coarse Sand (TILL FILL).	
5										
	3D	24/10	9.0 - 11.0	1-1-3-3	4	2			3D: Brown, moist, v. loose, fine to medium SAND, some Gravel, little Silt, trace coarse Sand (TILL FILL).	
10						2				
						9				
						19	920.5		12.5	
						39				
	4D	24/7	14.5 - 16.5	16-22-23-28	45	18			4D: Brown, dense, fine to medium Sandy GRAVEL, trace to little Silt, trace coarse Sand (TILL).	
15						50				
						55				
						50/4"			casing bouncing at 17.3 ft.	
						42				
	5D	17/11	19.0 - 20.4	17-34-50/5"	>84	91			5D: Brown, v. dense, fine to coarse Sandy GRAVEL, trace to little Silt (TILL).	
20	MR	60/5	20.3 - 25.3			48			possible cobble; driller notes drilling like "rotten" rock	
						28				
						30				
						49				
						44				
25	MR	8/6	25.3 - 26.0			50			predominately gravel; variety of rock types	
	6D	14/5	26.0 - 27.2	24-28-50/2"	>78				6D: Grey-brown, v. dense, GRAVEL, little fine to coarse Sand, trace Silt.	
	R1	53/53	27.2 - 31.6	RQD = 100%			905.8		27.2 — R1: Hard, fresh, aphanitic to fine grained, light grey, HORNFELS, with one 1-inch thick, high-angle, fine to medium grained seam that is likely relic bedding. One drill break. Core times: 1:55, 1:20, 1:20, 1:10, -- min:sec/ft.	
30										
	R2	60/60	31.6 - 36.6	RQD = 100%					R2: same as R1, except minor visible relic bedding and one 1/2-inch thick, low-angle granite intrusions and two 1/ 16-inch thick, high-angle quartz veins. Two drill breaks and one low angle fracture; planar, rough, slightly discolored and open. Core times: not recorded, 1:15, 1:15, 1:05, 1:05 min:sec/ft.	
35										
							896.4		36.6 — **Bottom of Exploration at 36.6 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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid Township, Maine	**Boring No.:** BB-MSR-104 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	932.9	**Auger ID/OD:**	solid stem auger to 14.0 ft.
Operator:	Enos/Dube	**Datum:**	NAVD88	**Sampler:**	standard split-spoon
Logged By:	Schonewald	**Rig Type:**	Mobile Drill B-51	**Hammer Wt./Fall:**	rope and cathead; 140#/30"
Date Start/Finish:	8/26/14; 1035 / 8/26/14; 1420	**Drilling Method:**	cased wash boring	**Core Barrel:**	NQ2
Boring Location:	Sta 591+01, 7.5 LT	**Casing ID/OD:**	NW (3") - driven	**Water Level*:**	13.0 ft. (open hole)

IN-SITU SAMPLING AND TESTING:	**ADDITIONAL DEFINITIONS:**	**BOREHOLE ADVANCEMENT METHOD:**
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	SSA= solid stem auger / RC=roller cone
MD = Unsuccessful Split Spoon Sample attempt	R = Rock Core Sample	**LABORATORY TEST RESULTS:**
U = Thin Wall Tube Sample	RQD = Rock Quality Designation (%)	LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
MU = Unsuccessful Thin Wall Tube Sample attempt	WOH = weight of 140lb. hammer	WC = water content, percent
V = Insitu Vane Shear Test	WOR = weight of rods	-#200 = percent fines from grain size analysis
MV = Unsuccessful Insitu Vane Shear Test attempt	-- = not recorded	UCT qp = peak compressive strength of rock

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
0						SSA			15 inches HMA	
							931.9		1.0	
	1D	24/12	1.0 - 3.0	24-19-11-8	30				Black, dry, weathered asphalt;	
							930.4		2.5	
									Changing at 2.5 ft. to 1D: brown, dry, Gravelly fine to coarse SAND, trace to little Silt (GRANULAR FILL).	
5	2D	24/7	4.0 - 6.0	3-5-5-3	10				2D: Brown, damp, loose, fine to medium SAND, some Gravel, little Silt, trace coarse Sand (TILL FILL).	
10	3D	24/13	9.0 - 11.0	2-11-11-6	22				3D: Brown, damp, m. dense, fine to medium SAND, some Gravel, little Silt, trace coarse Sand with one 5-inch layer grey broken rock at bottom of sample (TILL FILL). Till in tip of spoon.	
							921.9		11.0	
15	4D	24/13	14.0 - 16.0	7-10-11-11	21	26			4D: Brown, wet, m. dense, GRAVEL, some fine to coarse Sand, trace Silt.	
						48				
						48				
						65				
						109				
20	5D	24/9	19.0 - 21.0	29-28-28-30	56	61			5D: Greyish-brown, v. dense, GRAVEL, some fine to coarse Sand, trace Silt.	
						59				
						63				
						77				
						72				
25	6D	24/16	24.0 - 26.0	34-39-55-41	94	--			6D: Brownish-grey, v. dense, GRAVEL, some fine to coarse Sand, trace Silt.	
						--				
						--				
							905.3		27.6	
30	R1	60/58	27.8 - 32.8	RQD = 82%					R1: Hard, fresh, aphanitic to fine grained, light grey, HORNFELS with one 1/4-inch folded (loop), one 1/2-inch high angle, and one 1/16-inch high angle quartz to granite intrusions; minor evidence of relic bedding. Close, low and high angle fractures; typically undulating, rough, discolored, and open. One vertical fracture from 29.8 to 32.0 ft. Core times: 1:20, 1:05, 1:05, 1:05, 1:05 min:sec/ft.	
35	R2	60/60	32.0 - 32.6 32.8 - 37.8	SAMPLE RQD = 77%					R2: same as R1, except no intrusions. No fractures from 32.8 to 36.2 ft, and very close to close, predominately horizontal with lesser vertical fractures resulting in a blocky structure to the bottom of the core. Core times: 1:00, 1:05, 1:15, 1:10, 1:15 min:sec/ft.	UCT qp = 17,515 psi
							895.1		37.8	
									Bottom of Exploration at 37.8 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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

APPENDIX B

ROCK CORE PHOTOGRAPHS

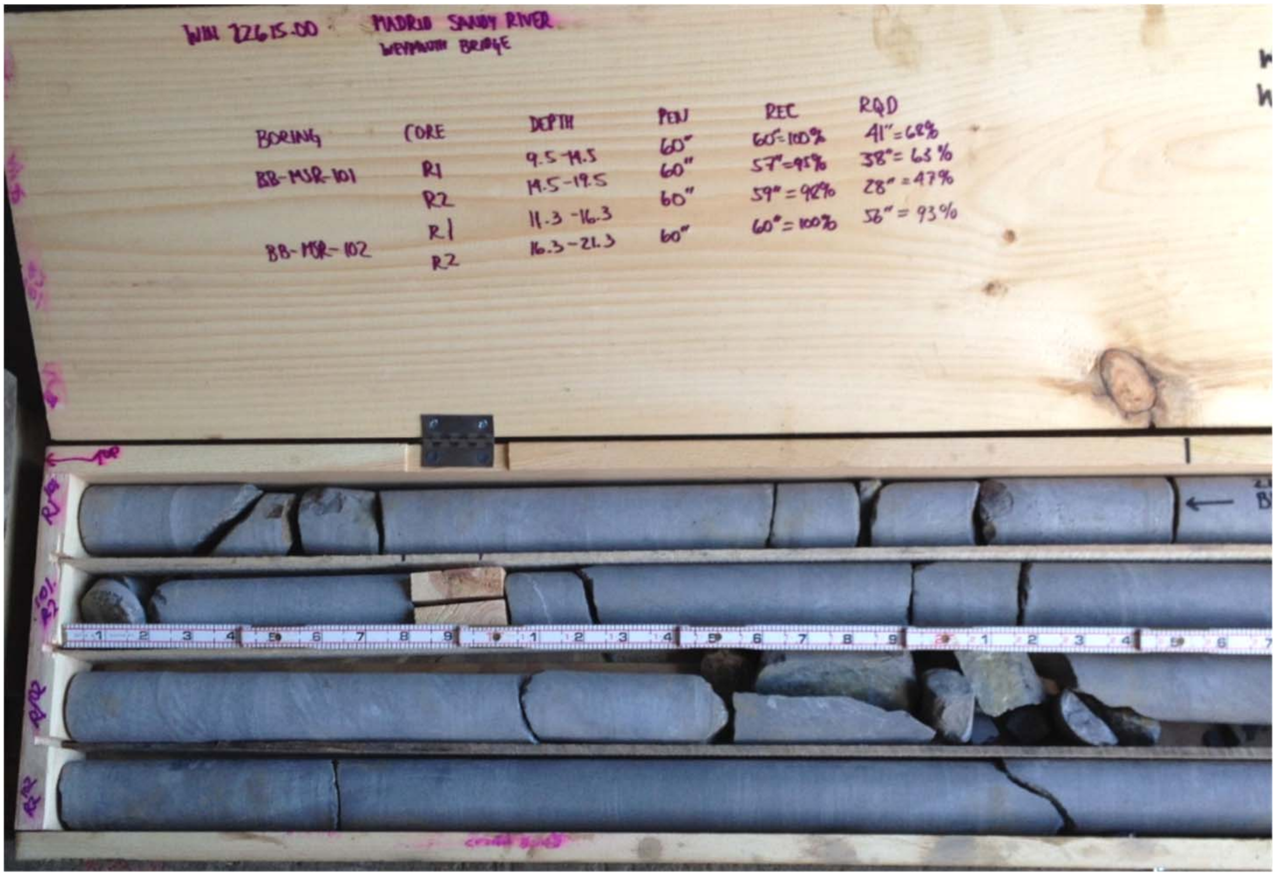


Photo 1: Core box containing rock core from test borings BB-MSR-101 and BB-MSR-102- left side of core box (top portion of cores).
From top to bottom 1) BB-MSR-101, R-1; 2) BB-MSR-101, R-2; 3) BB-MSR-102, R-1; 4) BB-MSR-102, R-2.

Photo 2: Core box containing rock core from test borings BB-MSR-101 and BB-MSR-102 – right side of core box (bottom portion of cores).
From top to bottom 1) BB-MSR-101, R-1; 2) BB-MSR-101, R-2; 3) BB-MSR-102, R-1; 4) BB-MSR-102, R-2.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
TEST BORINGS BB-MSR-101 & BB-MSR-102
WEYMOUTH BRIDGE REPLACEMENT
MADRID TOWNSHIP, MAINE

Sheet No.:

1 of 2

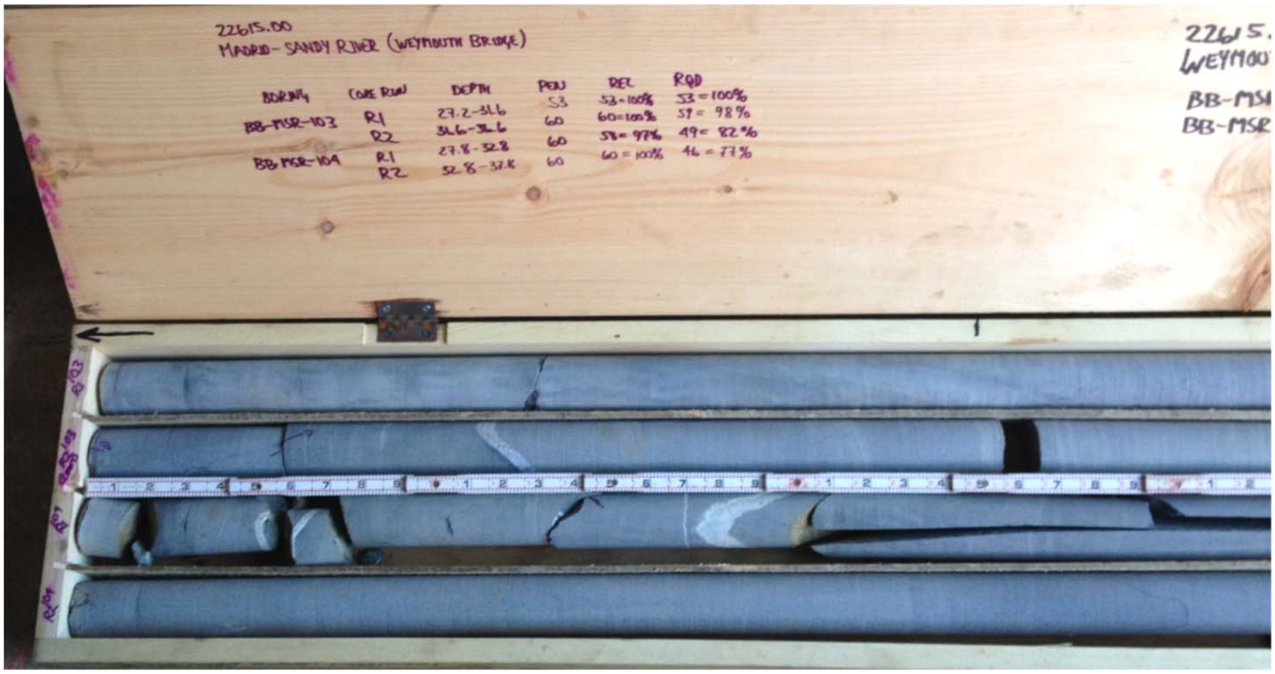


Photo 3: Core box containing rock core from test borings BB-MSR-103 and BB-MSR-104- left side of core box (top portion of cores).
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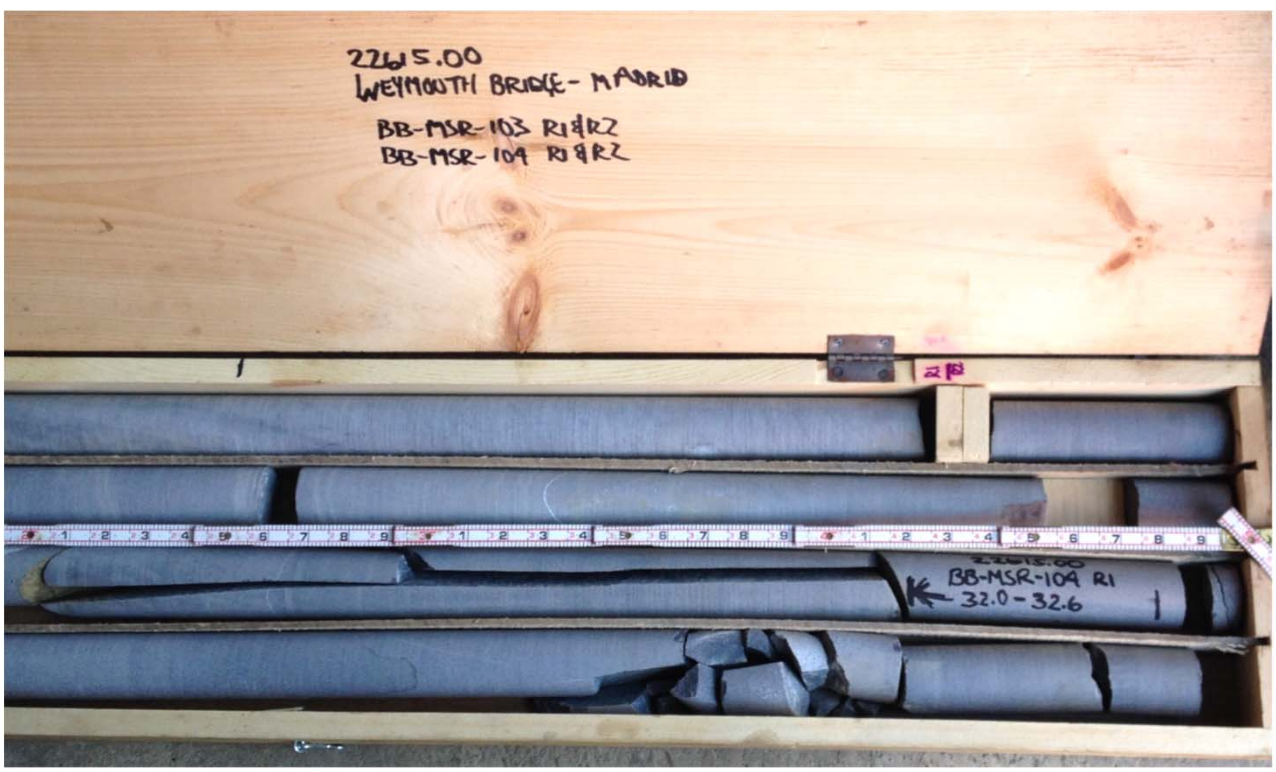


Photo 4: Core box containing rock core from test borings BB-MSR-103 and BB-MSR-104- right side of core box (bottom portion of cores).
From top to bottom 1) BB-MSR-103, R-1; 2) BB-MSR-103, R-2; 3) BB-MSR-104, R-1; 4) BB-MSR-104, R-2.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
TEST BORINGS BB-MSR-103 & BB-MSR-104
WEYMOUTH BRIDGE REPLACEMENT
MADRID TOWNSHIP, MAINE

Sheet No.:

2 of 2

SCHONEWALD ENGINEERING ASSOCIATES, INC.

APPENDIX C

LABORATORY TEST REPORT

LABORATORY TESTING DATA SHEET

Project Name MaineDOT Weymouth & Wing Bridges | Location Madrid and Phillips, ME | Reviewed By [signature]

Schonewald Engineering Assoc. No. 14-113 74-14-0003.18 | Assigned By B. Schonewald

Project Engineer B. Schonewald | Report Date 10/21/2014 | Date Reviewed 10/22/2012

Boring	Sample No.	Depth Ft.	Lab No.	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests (ASTM D7012 D) (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ KSI	Rock Formation or Description or Remarks
BB-MSR-101	R1	12.0-12.4	1	1.978	4.584	169.0			U	18,751	0.58		2.88	0.45		
BB-MSR-104	R1	32.0-32.4	2	1.978	4.615	169.5			U	17,515	0.31		5.20	1.28		

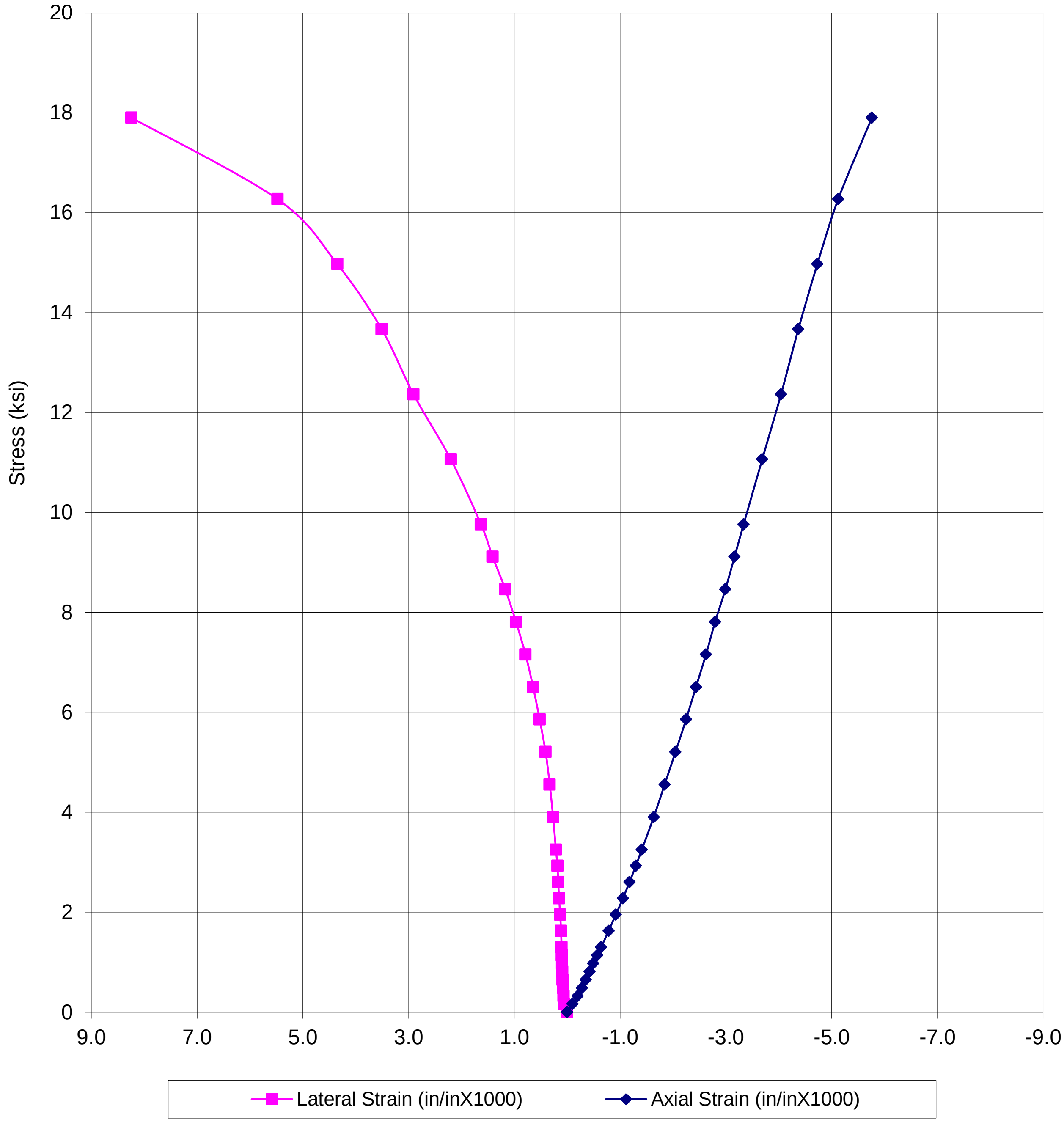
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

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

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

THIELSCH ENGINEERING
195 Frances Avenue
Cranston, RI 02910 401-467-6454

MaineDOT Weymouth & Wing Bridges
Madrid and Phillips, ME



Rock Testing

Schonewald EA	14-113	Test Method	ASTM D7012
Boring No.	BB-MSR-101	File No.	CTS-74-14-0003.18
Sample No.	R1	Date:	10/20/14
Depth:	12.0-12.4'	Test No.	U 1

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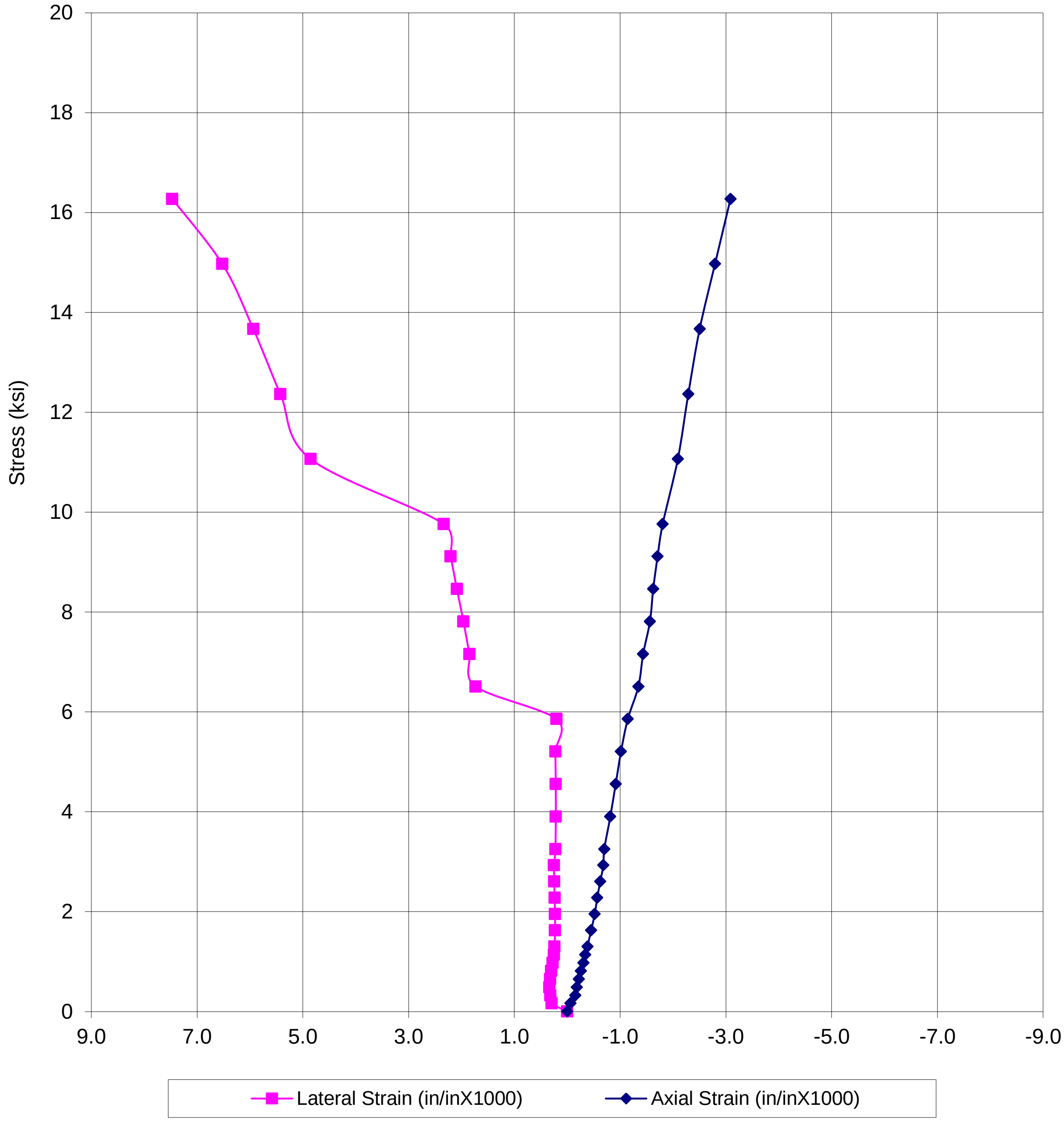
MaineDOT Weymouth & Wing Bridges
Madrid and Phillips, ME



Rock Testing

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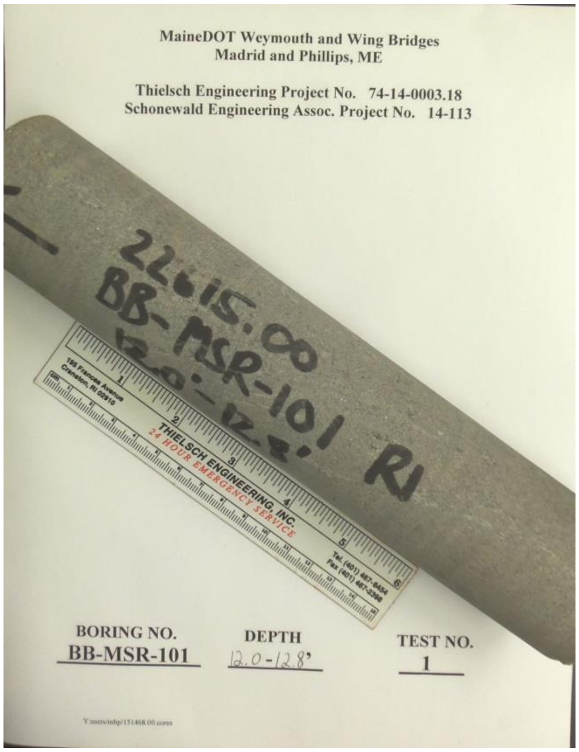
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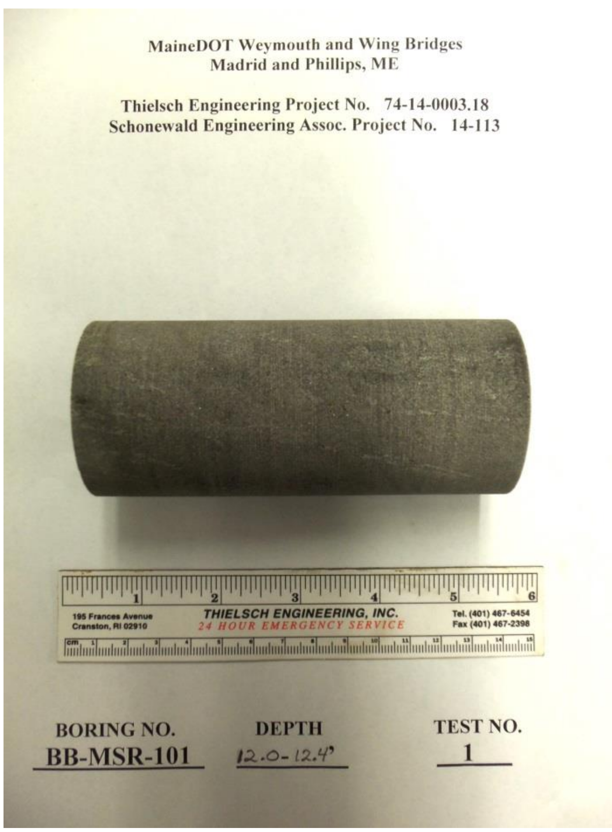
Madrid and Phillips, ME
Thielsch Engineering Project No. 74-14-0003.18
Schonewald Engineering Assoc. Project No. 14-113
22615.00
BB-MSR-101
R1
THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE
BORING NO.
BB-MSR-101
DEPTH
12.0-12.8'
TEST NO.
1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Madrid and Phillips, ME
Thielsch Engineering Project No. 74-14-0003.18
Schonewald Engineering Assoc. Project No. 14-113
195 Frances Avenue
Cranston, RI 02910
THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE
Tel. (401) 467-6454
Fax (401) 467-2398
BORING NO.
BB-MSR-101
DEPTH
12.0-12.4'
TEST NO.
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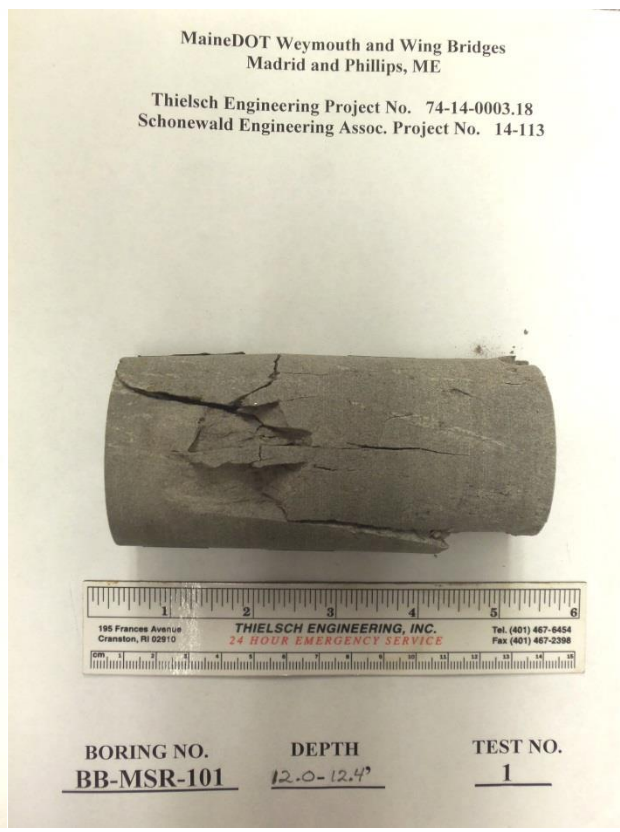
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Madrid and Phillips, ME
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195 Frances Avenue
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THIELSCH ENGINEERING, INC.
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DEPTH
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TEST NO.
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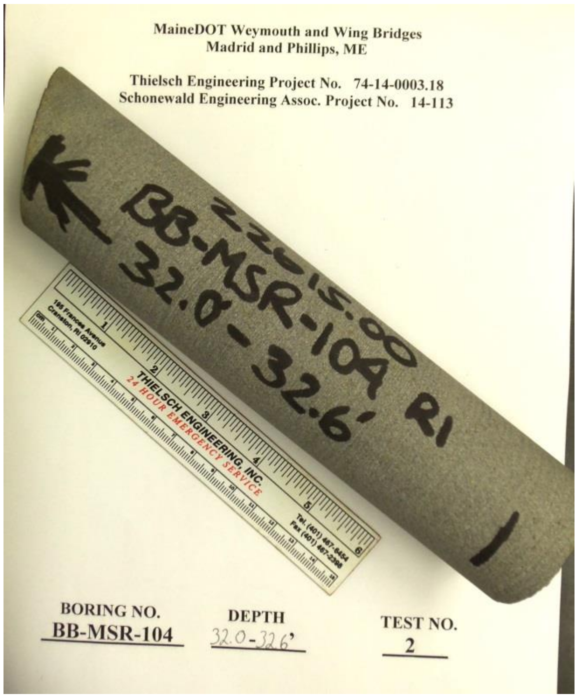
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Madrid and Phillips, ME
Thielsch Engineering Project No. 74-14-0003.18
Schonewald Engineering Assoc. Project No. 14-113
22015.00
BB-MSR-104 R1
32.0-32.6'
195 Frances Avenue
Cranston, RI 02910
THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE
Tel. (401) 467-6454
Fax (401) 467-2398
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DEPTH
32.0-32.6'
TEST NO.
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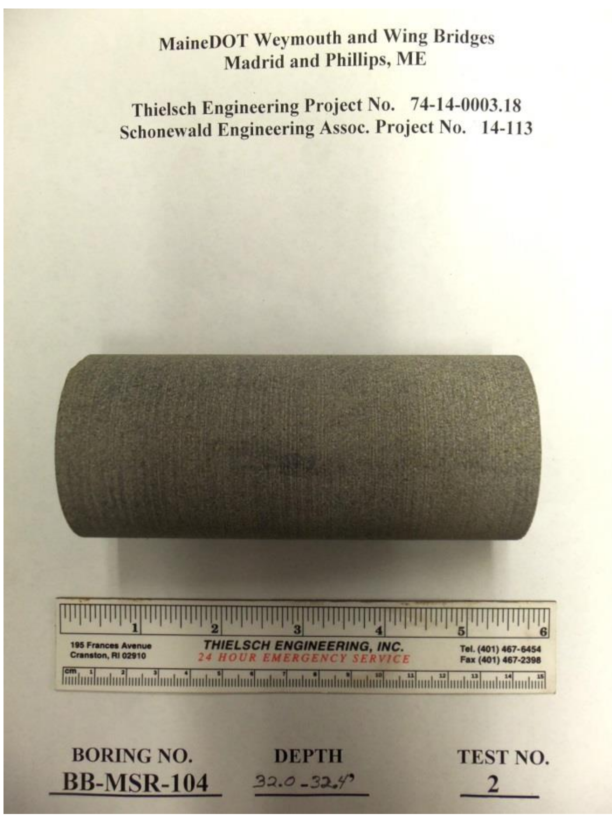
MaineDOT Weymouth and Wing Bridges
Madrid and Phillips, ME
Thielsch Engineering Project No. 74-14-0003.18
Schonewald Engineering Assoc. Project No. 14-113
195 Frances Avenue
Cranston, RI 02910
THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE
Tel. (401) 467-6454
Fax (401) 467-2398
BORING NO.
BB-MSR-104
DEPTH
32.0-32.4'
TEST NO.
2


MaineDOT Weymouth and Wing Bridges
Madrid and Phillips, ME
Thielsch Engineering Project No. 74-14-0003.18
Schonewald Engineering Assoc. Project No. 14-113
195 Frances Avenue
Cranston, RI 02910
THIELSCH ENGINEERING, INC.
24 HOUR EMERGENCY SERVICE
Tel. (401) 467-6454
Fax (401) 467-2398
BORING NO.
BB-MSR-104
DEPTH
32.0-32.4'
TEST NO.
2

Schonewald Engineering Associates, Inc.

APPENDIX D

HRG'S 2014 GEOPHYSICAL REPORT

HAGER-RICHTER
GEOSCIENCE, INC.

GEOPHYSICAL SURVEY
WEYMOUTH BRIDGE
MADRID TOWNSHIP, MAINE
PIN 22615.00

MAINE DOT CONTRACT
NO. 20110613000000006486

Prepared for:

Maine Department of Transportation
Highway Program
16 State House Station
Augusta, Maine 04333-0016

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 14J61
December, 2014

0. EXECUTIVE SUMMARY

Hager-Richter Geoscience, Inc. conducted a geophysical survey of approaches to the Weymouth Bridge, which carries State Route 4 over the Sandy River, in Madrid Township, Maine for the Maine Department of Transportation (MaineDOT) in September, 2014. The geophysical survey was performed in support of a geotechnical investigation of the Site by the Maine Department of Transportation (MaineDOT) for a project to replace portions of Route 4 between Madrid and Phillips, Maine.

The geophysical survey was conducted using the seismic refraction and ground penetrating radar (GPR) methods. Seismic refraction profiling was conducted along two 295-foot long seismic lines located along each shoulder of the roadway. The GPR survey was conducted on the roadway approximately between highway stationing 589+00 and 593+25 in an approximately 425-foot by 25-foot area centered on the bridge.

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc. along State Route 4 in the vicinity of the approaches to the Weymouth Bridge over the Sandy River in Madrid, Maine for the Maine Department of Transportation (MaineDOT) in September, 2014, we conclude the following:

- Based on the available boring and seismic data, bedrock is lowest under the west end of the bridge.

- The depth of bedrock along the seismic lines varies from about 8 and 29 feet below ground surface.

- The elevation of competent bedrock in the locations surveyed varies between 904 and 925 feet for a total relief of 21 feet.

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TABLES

FIGURES

APPENDIX

1. INTRODUCTION

Hager-Richter Geoscience, Inc. conducted a geophysical survey of approaches to the Weymouth Bridge, which carries State Route 4 over the Sandy River, in Madrid Township, Maine for the Maine Department of Transportation (MaineDOT) in September, 2014. Two geophysical methods were used: seismic refraction and ground penetrating radar. The geophysical survey was performed in support of a geotechnical investigation of the Site by the Maine Department of Transportation (MaineDOT) for a project to replace portions of Route 4 between Madrid and Phillips, Maine. A similar geophysical survey of the approaches to the Wing Bridge in Phillips Township, Maine was also conducted as part of the project.[1]

The general location of the Site is shown in Figure 1. Four borings (BB-MSR-101 to BB-MSR-104) were drilled in the Route 4 roadway, two borings on either side of the Weymouth Bridge within approximately 10 to 15 feet of the bridge. The borings encountered bedrock at 27-28 feet depth on the west side of the bridge and 10-11 feet on east side of the bridge. MaineDOT required information on the depth of bedrock in the vicinity of the bridge.

Hager-Richter conducted seismic refraction profiling along two 295-foot long seismic lines - one line on each side of the road and each line centered and extending over the bridge and approaches. In addition, Hager-Richter conducted a GPR survey on the roadway approximately between highway stationing 589+00 and 593+25 in an approximately 425-foot by 25-foot area centered on the bridge. Figure 2 is a modified site plan showing the locations of the seismic lines and the area covered by the GPR survey.

Route 4 is a two-lane road with paved and gravel shoulders. The natural ground surface in the area of interest dips toward the river. The Route 4 road surface maintains a relatively consistent elevation as it crosses the raised approaches and bridge deck. According to plans provided by MaineDOT, the river bed is approximately 10 to 16 feet below the top of the bridge deck. Bedrock outcrops occur in the river bed in the vicinity of the bridge.

Hager-Richter personnel were on-site on September 9, 2014. Jeffrey Reid, P.G., and Steven Grant, P.G., conducted the survey. The project was coordinated with Ms. Kitty Breskin, P.E., of MaineDOT. Ms. Meredith Kirkmann, P.E., also of MaineDOT was onsite for a portion of the fieldwork. MaineDOT personnel were onsite during the field work to provide lane closures and traffic control. MaineDOT provided plans showing site features, surface

[1] **MaineDOT Contract No. 20110613000000006486,** Geophysical Survey, Wing Bridge, Phillips Township, Maine, PIN 22616.00, HR File 14J061, December, 2014

topography, and the locations of borings. Data analysis and interpretation were completed at the Hager-Richter offices. Original data and field notes will be retained in the Hager-Richter files for a minimum of three years.

2. EQUIPMENT AND PROCEDURES

2.1 SEISMIC REFRACTION

2.1.1 Equipment. We used two Geometrics Geode units connected to, and controlled by, a notebook PC computer. The software provides for the acquisition, display, plotting, filtering and storage of seismic data. The seismogram image presented in real time on the notebook screen allows the operator to verify the quality of the data. The stored digital data are transferred to our server at the end of the field day for storage, backup, and future data processing.

The Geodes were coupled to two 24-element seismic spread cables for a total of up to 48 geophones. Each deployment of (up to) 48 co-linear geophones is called a spread, and multiple end-to-end spreads can be conducted to survey long transects. The geophones measure only the vertical component, and their resonant frequency is 12 Hz.

Seismic energy is provided by a 12-lb sledge hammer striking an aluminum base plate, an EWG, or a Betsy seisgun. The Betsy seisgun uses a shotgun blank as the seismic source and is not classified as a weapon or explosive under Federal regulations. The EWG is an accelerated weight drop, using industrial elastics to accelerate the weight. The number of stacks per shot point is variable, and the quality of the stacked seismic signal for each shot point was verified in the field. Six to nine shot points were used for each 48-geophone spread -- one off each end of the cable, one at each end of the cable, and two to five internal to the spread. This configuration provides reversed profiles.

2.1.2 Data Analysis and Interpretation. The seismic data are analyzed using the Generalized Reciprocal Method (GRM) of seismic refraction interpretation. The method is described in detail in Palmer (1980).[2] GRM allows for some variation in the surface topography as well as lateral variation in the seismic velocity of the upper layers. The method uses the principle of migration whereby the refractor need only be planar over a short distance, thus allowing the calculation of depth to an undulating interface. In addition, GRM is relatively insensitive to dip angles as high as 20^o, unlike most other methods that can be sensitive to dips as low as 5^o. GRM also allows for the calculation of depth below each geophone instead of below only the shot points as in the Time-Intercept and Crossover Distance methods. The GRM software that we use for data analysis (IXRefraX by Interpex) contains several internal tests for data consistency.

[2]Palmer, Derecke (1980) The Generalized Reciprocal Method of Seismic Refraction Interpretation, Society of Exploration Geophysicists, 104 p.

The results are used to construct an interpreted velocity profile of the subsurface for each seismic line. The velocities of seismic waves are functions of the types of geologic material through which they pass. One can thus infer the general subsurface stratigraphy from the velocities determined. Seismic velocities are expressed in feet per second (ft/s).

A widespread misconception about the seismic refraction method is that one cannot detect velocity inversions (layers of lower velocity material underlying higher velocity material) or hidden layers (layers of intermediate velocity too thin to produce first arrival signals), common conditions in stratified sediments. If present and undetected, such layers can cause large errors in the depths calculated for the various layers. However, using GRM, the presence of such layers can be inferred readily, and more importantly, the method uses average velocities for the detected and undetected layers to determine accurate depths to the refractors that are detected.

2.1.3 Limitations of the Method. The accuracy (standard deviation) of the apparent *depths* of relatively competent bedrock determined by the seismic refraction survey is about ± 10% of the apparent depth of bedrock, or ± 2 feet, whichever is greater. **The bedrock model shown as a profile, bedrock elevation contour plot, or listed as tabular data should not be used for contract bedrock removal quantities.** Like all geophysical methods, the seismic refraction method is based on the assumption that the local geology is uncomplicated. In particular, the seismic refraction method assumes that interfaces between geologic materials correlate with sharp increases in seismic velocity and that the interfaces are relatively flat-lying. The method is not very sensitive to lateral variations within layers, and relatively subtle features such as fracture zones within bedrock are generally difficult to detect unless there is a topographic expression of the feature. The accuracy of the method is degraded in areas with strong topographic relief at the surface and/or where the interfaces have apparent dips greater than about 20°.

Where two materials do not exhibit contrasting velocities, or where velocities gradually increase with depth, a clear refracted signal is not generated, and the seismic refraction method cannot be used to distinguish the two materials. In some cases, the "geophysical contact" between materials with contrasting velocities does not correlate exactly with the "geologic contact." For example, where a highly weathered bedrock is overlain by a dense material such as till, the velocity range of the weathered bedrock might overlap or approach the velocity range of the till, and the two materials cannot be distinguished seismically. In such cases, the depth determined by seismic refraction is the depth of *competent* bedrock, which might be located at some depth below the geologic contact.

The depth relations of the water table and bedrock may constitute a significant problem for the seismic refraction technique. This problem is that of a "blind layer." A blind layer occurs where the thickness of the saturated overburden is less than about half the depth of bedrock. In such cases, the water-saturated material immediately above bedrock is "blind" in the sense that no refracted seismic energy from it will be received as a first arrival of seismic energy, and all methods used to reduce the seismic data to determine the depth of bedrock, the objective of this survey, use *only* first arrivals. Thus, the saturated layer will not be detected where it is close to bedrock, and most methods of seismic data reduction will indicate that bedrock is considerably deeper than it actually is. Although GRM, the method used by Hager-Richter to reduce the seismic refraction data, does not use first arrivals through the water saturated zone (because there is none to use) in such cases, GRM determines the depth of bedrock correctly by using the *average* velocity of the saturated and unsaturated zones.

A "hidden layer" occurs where a lower velocity material underlies a higher velocity material, a common situation in stratified sediments. An example is where sands are present under layers of clay or till. As in the case of a "blind layer," most methods of seismic refraction data reduction will indicate that bedrock is shallower than it actually is, if a hidden layer is present but not detected. Internal tests in the seismic refraction data reduction software that we use (IXRefraX by Interpex) indicate that such layers might be present, and an average velocity of the two layers is used to determine the depth of bedrock.

2.1.4 Site Specific. The seismic refraction survey consisted of two parallel 295-foot lines, one located on each shoulder of Route 4. Each line consisted of two 115-foot long sections along the approaches containing active geophones, separated by a 65-foot gap across the bridge. MaineDOT provided site plans showing site features, surface topography, and boring locations. Hager-Richter located the seismic lines in the field using site features such as the bridge, boreholes, and roadway. The locations of the seismic lines are shown on Figure 2. Elevations along the seismic line were determined from site plans provided by MaineDOT.

A geophone spacing of 5 feet was used for the seismic lines. The seismic source was a 12-pound sledge hammer striking an aluminum plate. Seven shot points were used for each line -- four internal shot points, one shot point at each end of the line, and one external shot located in-line but offset from each end of the line. The photograph below shows the setup of Seismic Line 2.

2.2 GPR

2.2.1 General. The GPR survey was conducted using our GSSI UtilityScan DF subsurface imaging radar system. Data were recorded digitally, and the GPR data were reviewed in the field. The system includes a survey wheel that triggers the recording of the data at fixed intervals, thereby increasing the accuracy of the locations of features detected along the survey lines. The UtilityScan DF acquires data simultaneously from an 800 MHZ and a 300 MHZ antenna. The GPR data were processed using RADAN 7™ software licensed by Geophysical Survey Systems, Inc.

Photograph of Line 2 across the Weymouth Bridge, looking east. Geophones are planted in the gravel shoulders on either side of the bridge. The seismograph and operator are located in the center of the bridge.

2.2.2 Limitations of the Method. There are limitations of the GPR technique as used to detect and/or locate targets such as those of the objectives of this survey: (1) surface conditions, (2) electrical conductivity of the ground, (3) contrast of the electrical properties of the target and the surrounding soil, and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us.

The condition of the ground surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. Sites covered with snow piles, high grass, bushes, landscape structures, debris, obstacles, soil mounds, etc. limit the survey access and the coupling of the GPR antenna with the ground. In many cases, the GPR signal will not penetrate below concrete pavement, especially inside buildings, and a target may not be detectable. The GPR method also commonly does not provide useful data under canopies found at some facilities.

The electrical conductivity of the ground determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. For example, the GPR signal does not penetrate clay-rich soils, and targets buried in clay might not be detected.

A definite contrast in the electrical conductivities of the surrounding ground and the target material is required to obtain a reflection of the GPR signal. If the contrast is too small then the reflection may be too weak to recognize, possibly due to deeply corroded metal in the target, the target can be missed.

2.2.3 Site Specific. GPR traverses were conducted in one direction approximately between highway stationing 589+00 and 593+25 in the 425-foot by 25-foot area of interest. GPR traverses oriented approximately parallel to the roadway were spaced 4 or 6 feet apart. MaineDOT provided site plans showing site features, surface topography, and boring and probe locations. Hager-Richter located the GPR traverses in the field using site features such as the road, lines on the roadway, and the bridge structure. The location of the GPR survey area is shown on Figure 2.

3. RESULTS AND DISCUSSIONS

3.1 GENERAL

The geophysical survey of the roadway in the vicinity of the Weymouth Bridge consisted of seismic refraction profiling along two 295-foot long lines located approximately along the road shoulders of Route 4 and a GPR survey in a 425-foot by 25-foot survey area located on the roadway and centered on the bridge. The locations of the seismic refraction traverses and the GPR survey area are shown in Figure 2.

3.2 SEISMIC REFRACTION

3.2.1 General. The seismic refraction survey consisted of two transects designated as Seismic Lines 1 and 2. The results of the seismic refraction survey are shown in profile form in Figure 3 and as a bedrock elevation contour plot in Figure 4, and are listed in Table 1.

3.2.2 Data Quality. The quality of the seismic refraction data was excellent. A measure of the accuracy of the data can be obtained by comparing the depths determined seismically with depths reported from nearby borings that intersect bedrock, and the internal consistency of the data can be assessed by comparison of the depths determined at intersecting seismic lines or with results from other geophysical methods. For the present survey, four borings (BB-MSR-101 to 104) were drilled in the roadway in the vicinity of the east and west ends of the bridge and Table 2 compares the depths of bedrock determined from (A) the boring data and (B) seismically. The borings on the west side of the bridge (BB-MSR-103 and 104) encountered bedrock at elevations of 905-906 feet and borings on the east side of the bridge (BB-MSR-101 and 102) encountered bedrock at elevations of 922-924 feet. Seismically determined elevations for the top of bedrock for Seismic Line 1, located within 5 feet of BB-MSR-101 and 103 and Seismic Line 2, located within 5 feet of BB-MSR-102 and 104, differ by 2 feet or less from bedrock elevations reported in the boring logs.

For the present survey, the seismic lines are parallel and do not intersect, and GPR signal penetration was insufficient to detect bedrock. However, based on the results of comparing seismically determined elevations with the refusal depth reported for nearby borings, and on the results from other similar seismic refraction surveys, we estimate the accuracy (standard deviation) of the *depths* of competent bedrock determined by the seismic refraction survey to be about ± 10% of the depth of bedrock, or ± 2 feet), whichever is greater.

3.2.3 Interpretation of Velocities. Materials with two distinct velocity ranges were detected at the Site. The upper material exhibits a velocity range of 1,100 ft/s to 1,850 ft/s and is interpreted to consist of mostly unsaturated soils and fill materials.

The lower material exhibits a velocity range of 12,900 to 16,000 ft/s and is interpreted to be competent bedrock. Where the top of bedrock is highly fractured and/or deeply weathered, it might exhibit lower velocities that cannot be detected as a distinct layer on the basis of the seismic refraction data. Thus, the top of rock determined on the basis of seismic refraction data generally is the top of *competent* bedrock, which might be located somewhat below the geologic contact between the overburden and bedrock.

3.2.4 Bedrock Depths and Configuration. The results of the seismic refraction survey are listed in Table 1, presented as seismic profiles in Figure 3, and presented as a bedrock elevation contour plot in Figure 4. Seismic Lines 1 and 2 each have a 65-foot gap in the bridge section where seismic data could not be acquired. The depth of competent bedrock along the seismic lines varies between about 8 and 29 feet below ground surface. The elevation of competent bedrock in the locations surveyed varies between 904 and 925 feet for a total relief of 21 feet.

Figure 4 shows a color contour plot of the bedrock elevation model generated from the seismic refraction data and the bedrock boring data. The contours represent interpolations based on the seismic data and available boring information. The contours shown represent non-unique models for bedrock elevation and depth, respectively, (i.e., different valid conceptual models can be developed to fit the data set), and the elevation and depth of competent bedrock at any particular location may differ from that shown. Bedrock elevations and depths based on additional data, such as additional borings or seismic data, may differ significantly from those shown on the plates. **The bedrock model shown as profiles or listed as tabular data should not be used for contract bedrock removal quantities.**

Examination of the seismic profiles and the bedrock topographic model shows that the bedrock surface dips toward the Sandy River along the western portion of the seismic lines for which data could be acquired. The bedrock surface is relatively flat and generally shallower along the eastern portion of the seismic lines. Based on the available boring and seismic data, bedrock is lowest under the west end of the bridge.

3.3 GPR

3.3.1 General. The GPR survey of the 425-foot by 25-foot area of interest consisted of GPR traverses oriented approximately parallel to the roadway and spaced 4 or 6 feet apart. The location of the GPR survey area is shown in Figure 2.

Apparent GPR signal penetration was greatest along the gravel shoulders of Route 4. In such areas two-way traveltime reflections were received from no more than about 55 ns. Based on site-specific time-to-depth conversions for the GPR signal, the GPR signal penetration is estimated to have been no more than about 6 feet for the Highway shoulders. GPR signal penetration was generally less in the central portion of the Route 4 travel lanes, with GPR signal penetration limited to no more than 30 ns, or about 3.5 feet.

3.3.2 Data Quality and Interpretation. Strong GPR reflections consistent with those expected for the top of bedrock were not detected in the GPR records for the subject site. GPR signal penetration for the survey area was not sufficient to produce reflections from bedrock, which is consistent with the results of the seismic refraction survey that determined bedrock depth in the area varies between approximately 8 and 29 feet.

4. CONCLUSIONS

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc. along State Route 4 in the vicinity of the approaches to the Weymouth Bridge over the Sandy River in Madrid, Maine for the Maine Department of Transportation (MaineDOT) in September, 2014, we conclude the following:

- Based on the available boring and seismic data, bedrock is lowest under the west end of the bridge.
- The depth of bedrock along the seismic lines varies from about 8 and 29 feet below ground surface.
- The elevation of competent bedrock in the locations surveyed varies between 904 and 925 feet for a total relief of 21 feet.

5. LIMITATIONS

This report was prepared for the exclusive use of the Maine Department of Transportation (Client). No other party shall be entitled to rely on this Report or any information, documents, records, data, interpretations, advice or opinions given to Client by Hager-Richter Geoscience, Inc. (H-R) in the performance of its work. The Report relates solely to the specific project for which H-R has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of H-R. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to H-R.

H-R has used reasonable care, skill, competence and judgment in the preparation of this Report consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by H-R should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and are not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

Except as expressly provided in this limitations section, H-R makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed.

HAGER-RICHTER
GEOSCIENCE, INC.

MaineDOT Contract No. 20110613000000006486
Geophysical Survey
Weymouth Bridge
Madrid Township, Maine
PIN 22615.00 HR File 14J061 December, 2014

TABLE 1
SEISMIC REFRACTION RESULTS
WEYMOUTH BRIDGE
MADRID TOWNSHIP, MAINE

Line	Location (ft)	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
1	0+00	955476.7	737895.3	933	13	920
1	0+05	955481.4	737897.1	933	13	920
1	0+10	955486.1	737898.8	933	13	919
1	0+15	955490.8	737900.6	933	14	919
1	0+20	955495.4	737902.4	933	14	919
1	0+25	955500.1	737904.1	933	14	919
1	0+30	955504.8	737905.9	933	14	919
1	0+35	955509.5	737907.6	933	14	919
1	0+40	955514.2	737909.4	933	14	918
1	0+45	955518.8	737911.2	933	15	918
1	0+50	955523.5	737912.9	933	15	918
1	0+55	955528.2	737914.7	933	16	917
1	0+60	955532.9	737916.5	933	17	915
1	0+65	955537.6	737918.2	933	18	915
1	0+70	955542.2	737920.0	933	19	914
1	0+75	955546.9	737921.8	933	19	914

Line	Location	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
1	0+80	955551.6	737923.5	933	22	911
1	0+85	955556.3	737925.3	933	22	911
1	0+90	955561.0	737927.0	933	22	910
1	0+95	955565.6	737928.8	933	24	909
1	1+00	955570.3	737930.6	933	24	909
1	1+05	955575.0	737932.3	933	27	906
1	1+10	955579.7	737934.1	933	28	905
1	1+15	955584.4	737935.9	933	28	904
65-foot gap for bridge						
1	1+80	955645.2	737958.8	932	11	921
1	1+85	955649.9	737960.5	932	11	922
1	1+90	955654.5	737962.3	932	10	922
1	1+95	955659.2	737964.1	933	10	922
1	2+00	955663.9	737965.8	933	11	922
1	2+05	955668.6	737967.6	933	11	922
1	2+10	955673.2	737969.3	933	11	922

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. Easting and northing coordinates and elevations for the seismic lines were determined from topographic plans provided by MaineDOT

MaineDOT Contract No. 20110613000000006486
Geophysical Survey
Weymouth Bridge
Madrid Township, Maine
PIN 22615.00 HR File 14J061 December, 2014

TABLE 1 (CONTINUED)
SEISMIC REFRACTION RESULTS

Line	Location (ft)	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
1	2+15	955677.9	737971.1	933	11	922
1	2+20	955682.6	737972.9	933	11	922
1	2+25	955687.3	737974.6	933	11	922
1	2+30	955692.0	737976.4	933	11	922
1	2+35	955696.6	737978.2	933	11	922
1	2+40	955701.3	737979.9	933	11	922
1	2+45	955706.0	737981.7	933	11	922
1	2+50	955710.7	737983.5	933	10	923
1	2+55	955715.4	737985.2	933	10	924
1	2+60	955720.0	737987.0	933	9	924
1	2+65	955724.7	737988.7	933	9	924
1	2+70	955729.4	737990.5	933	9	924
1	2+75	955734.1	737992.3	933	9	924
1	2+80	955738.8	737994.0	933	9	924
1	2+85	955743.4	737995.8	933	9	924
1	2+90	955748.1	737997.6	933	9	924
1	2+95	955752.8	737999.3	933	8	925
2	0+10	955477.1	737922.0	933	18	915
2	0+15	955481.8	737923.8	933	18	915

Line	Location	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
2	0+20	955486.5	737925.5	933	19	914
2	0+25	955491.2	737927.2	933	19	914
2	0+30	955495.9	737928.9	933	19	914
2	0+35	955500.6	737930.7	933	19	914
2	0+40	955505.3	737932.4	933	19	914
2	0+45	955510.0	737934.1	933	19	914
2	0+50	955514.7	737935.8	933	19	914
2	0+55	955519.4	737937.5	933	20	914
2	0+60	955524.1	737939.3	933	20	913
2	0+65	955528.8	737941.0	933	20	913
2	0+70	955533.5	737942.7	933	20	913
2	0+75	955538.1	737944.4	933	21	912
2	0+80	955542.8	737946.1	933	21	912
2	0+85	955547.5	737947.9	933	22	911
2	0+90	955552.2	737949.6	933	23	910
2	0+95	955556.9	737951.3	933	24	909
2	1+00	955561.6	737953.0	933	26	907
2	1+05	955566.3	737954.8	933	28	905
2	1+10	955571.0	737956.5	933	29	904

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. Easting and northing coordinates and elevations for the seismic lines were determined from topographic plans provided by MaineDOT

HAGER-RICHTER
GEOSCIENCE, INC.

MaineDOT Contract No. 20110613000000006486
Geophysical Survey
Weymouth Bridge
Madrid Township, Maine
PIN 22615.00 HR File 14J061 December, 2014

TABLE 1 (CONTINUED)
SEISMIC REFRACTION RESULTS

Line	Location (ft)	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
2	1+15	955575.7	737958.2	933	28	905
2	1+20	955580.4	737959.9	932	28	905
2	1+25	955585.1	737961.6	932	27	906
65-foot gap for bridge						
2	1+90	955646.1	737984.0	933	12	921
2	1+95	955650.8	737985.7	933	12	921
2	2+00	955655.5	737987.5	933	12	921
2	2+05	955660.2	737989.2	933	13	920
2	2+10	955664.9	737990.9	933	13	920
2	2+15	955669.6	737992.6	933	14	919
2	2+20	955674.3	737994.3	933	14	919
2	2+25	955679.0	737996.1	933	14	919
2	2+30	955683.7	737997.8	933	14	920
2	2+35	955688.4	737999.5	933	13	920
2	2+40	955693.1	738001.2	933	13	920
2	2+45	955697.8	738003.0	933	13	920
2	2+50	955702.5	738004.7	933	13	921
2	2+55	955707.1	738006.4	933	12	921
2	2+60	955711.8	738008.1	933	12	921

Line	Location	Easting (ft)	Northing (ft)	Surface Elevation (ft)	Bedrock Depth (ft)	Bedrock Elevation (ft)
2	2+65	955716.5	738009.8	933	12	922
2	2+70	955721.2	738011.6	933	11	922
2	2+75	955725.9	738013.3	933	11	922
2	2+80	955730.6	738015.0	933	11	922
2	2+85	955735.3	738016.7	933	11	922
2	2+90	955740.0	738018.4	933	12	921
2	2+95	955744.7	738020.2	933	12	921
2	3+00	955749.4	738021.9	933	13	920
2	3+05	955754.1	738023.6	933	13	920

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. Easting and northing coordinates and elevations for the seismic lines were determined from topographic plans provided by MaineDOT

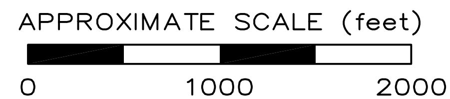
MaineDOT Contract No. 20110613000000006486
Geophysical Survey
Weymouth Bridge
Madrid Township, Maine
PIN 22615.00 HR File 14J061 December, 2014

TABLE 2
COMPARISON OF BEDROCK ELEVATIONS
WEYMOUTH BRIDGE
MADRID TOWNSHIP, MAINE

Comparison of Seismically Determined Bedrock Elevations with Bedrock Elevations Reported in Boring Logs

Seismic Line and Location	/	Boring	Distance from Seismic Line to Boring	Bedrock Elevations (feet)		Bedrock Depth from Boring (feet)	Difference	
				Seismic Line	Boring		Feet	Percent
SL1 1+09	/	BB-MSR-103	4.6' N	905	906	27	1	4
SL1 1+89	/	BB-MSR-101	5' N	922	924	10	2	20
SL2 1+20	/	BB-MSR-104	4.7' S	905	905	28	0	0
SL2 1+99	/	BB-MSR-102	3' S	921	922	11	1	9
						Average	1	8
						Standard Deviation	1	9

Boring information provided by MaineDOT. The absolute differences in feet reflect the absolute difference between bedrock elevation determined for a location on a seismic line and bedrock elevation reported for a nearby boring. The percentage differences were calculated by dividing the absolute differences in feet by the bedrock depth reported in the boring log.



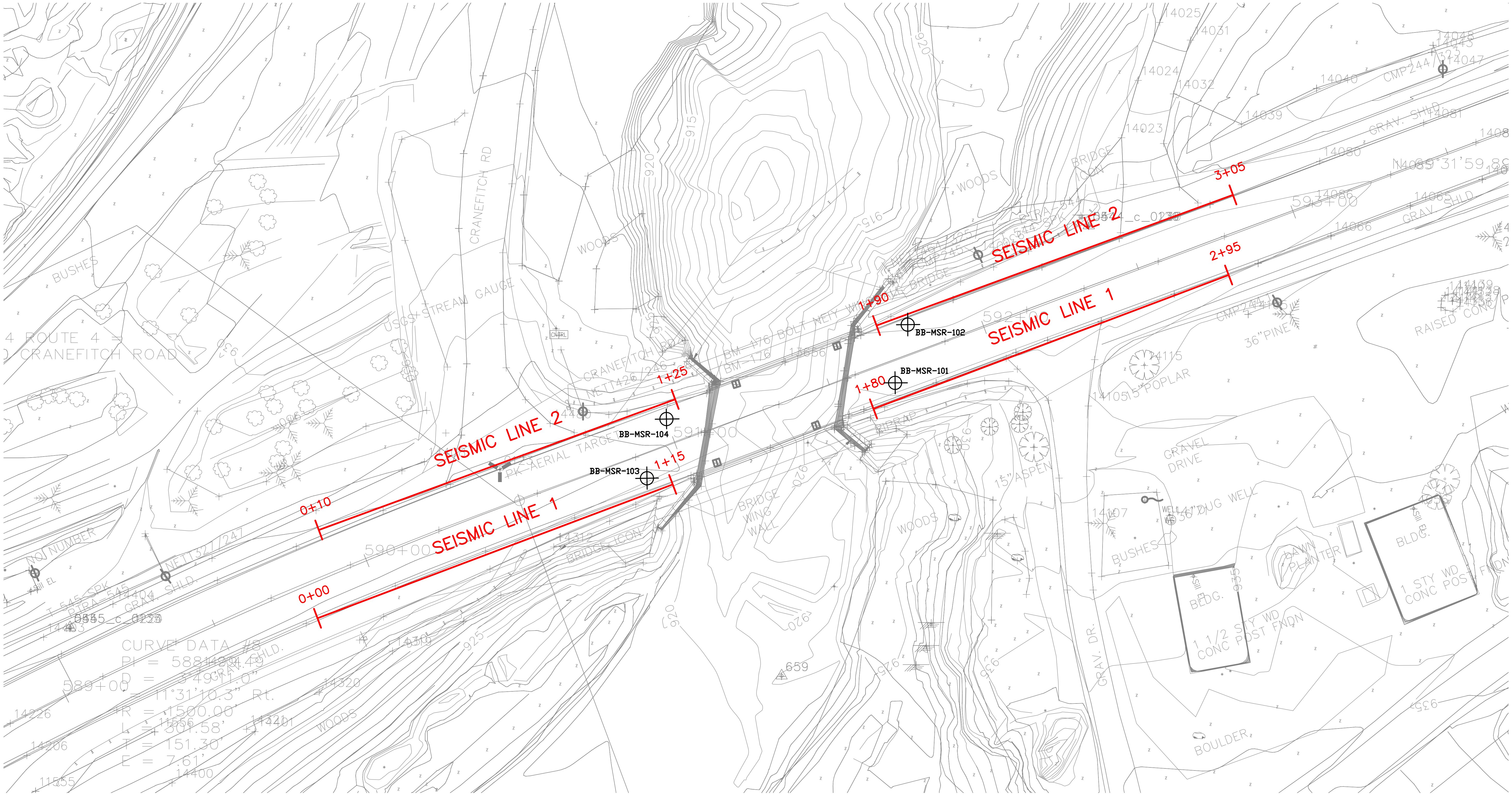
NOTE:

Modified from Google Earth Pro aerial photograph.

Figure 1
General Site Location
Weymouth Bridge
Madrid Townships, Maine
PIN 22615.00

File 14J61	December, 2014

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



LEGEND

SEISMIC LINE

BORING

SCALE (feet)

0 30 60

N

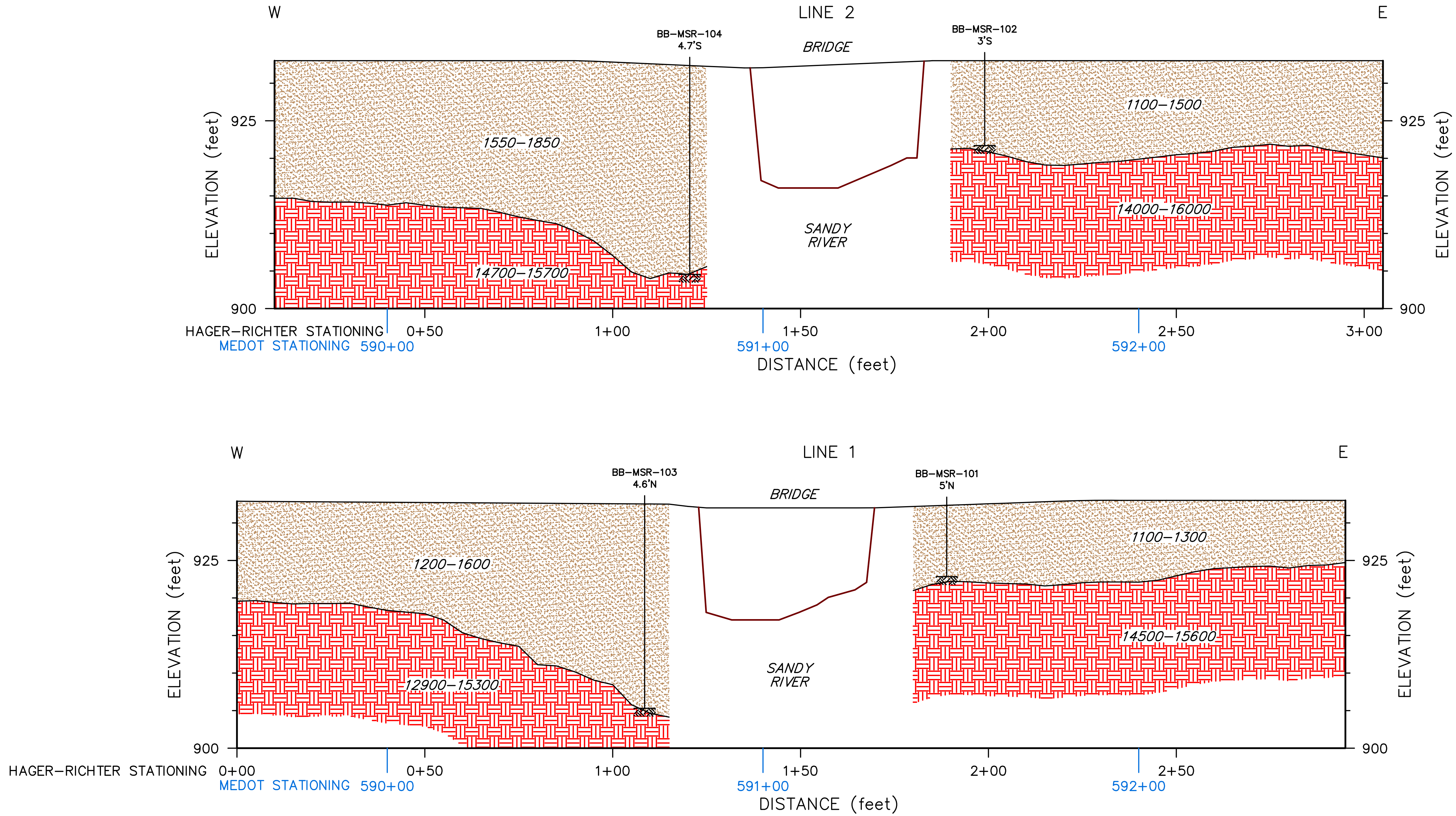
NOTE:

Modified from site plan provided by Maine Department of Transportation, identified as ALIGNMENTS.dgn.

Figure 2
Site Plan
Weymouth Bridge
Madrid Townships, Maine
PIN 22615.00

File 14J61	December, 2014

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



NOTES:

1. Estimated accuracy (standard deviation) of depth of bedrock is ±10% or 2 feet, whichever is greater.
2. The depths determined for bedrock are depths of competent rock; weathered and/or fractured bedrock might occur at shallower depths.
3. Surface elevations determined from plans provided by Maine Department of Transportation.
4. Data were analyzed using the Generalized Reciprocal Method.

LEGEND

Unsaturated soils

Competent bedrock

12900-15300 Velocity (fps)

Interface determined from seismic refraction data

BB-MRS-104
4.7'S

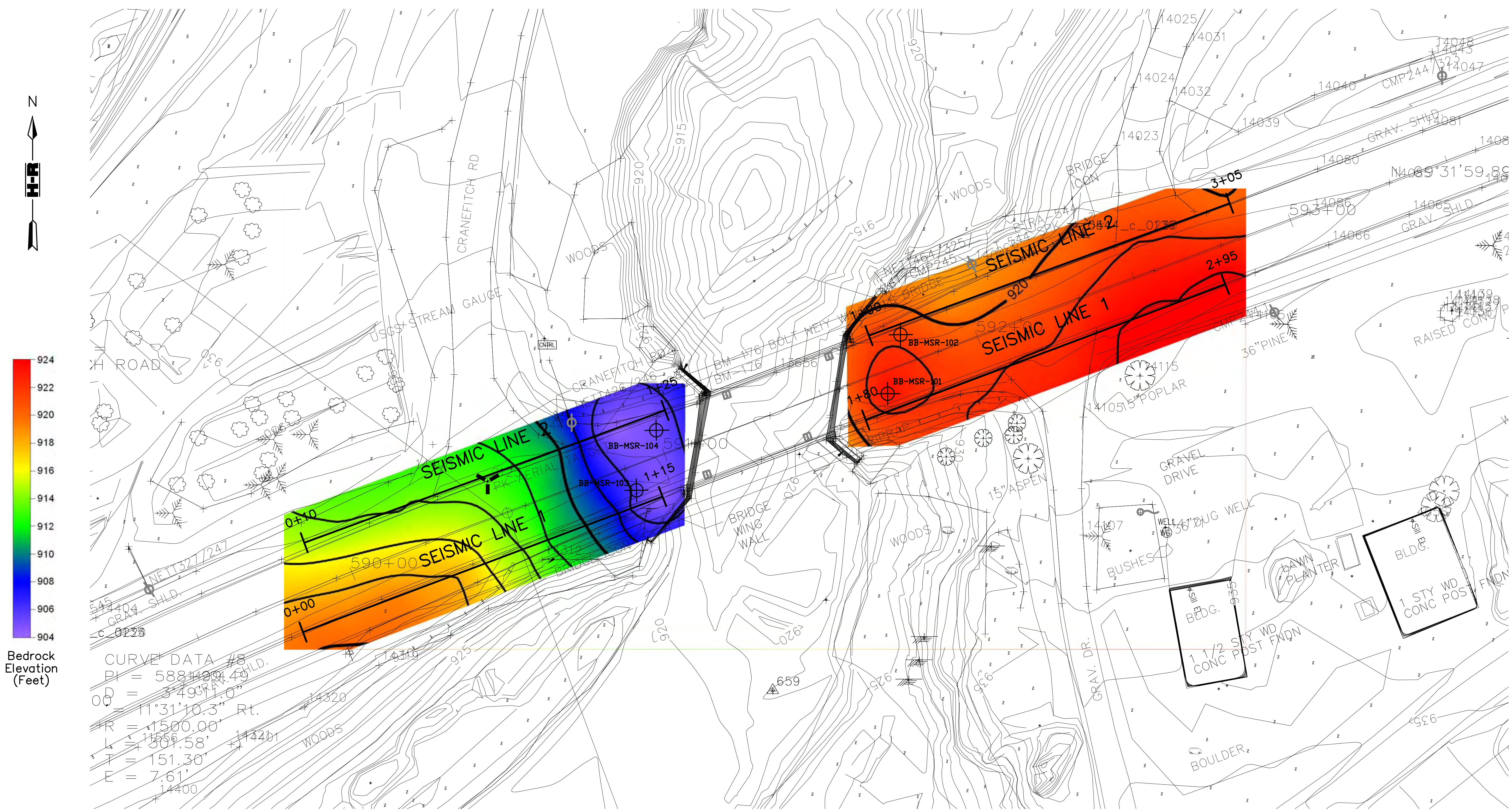
Boring with identification, distance from traverse, and depth of bedrock based on logs provided by Maine Department of Transportation

HORIZONTAL SCALE (feet)
0 30 60
Vertical Exaggeration = 2X

Figure 3
Seismic Lines 1 & 2
Weymouth Bridge
Madrid Townships, Maine
PIN 22615.00

File 14J61 | December, 2014

HAGER-RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



NOTES:

1. Modified from site plan provided by Maine Department of Transportation, identified as ALIGNMENTS.dgn.

2. Contour Interval = 2 Feet.

3. Bedrock elevation shown on this plot is a non-unique model. The contours represent interpolations based on seismic refraction data. The elevations of the bedrock surface at any particular location may differ from that shown. Bedrock elevation based on additional data may differ significantly.

LEGEND

SEISMIC LINE

BORING

SCALE (feet)

0 30 60

Figure 4
Bedrock Elevation
Weymouth Bridge
Madrid Townships, Maine
PIN 22615.00

File 14J61 | December, 2014

HAGER-RICHTER GEOSCIENCE, INC.
Salem, New Hampshire

APPENDIX
BORING LOGS

SCHONEWALD ENGINEERING ASSOCIATES, INC.

PROJECT: Weymouth Bridge
Route 4 over Sandy River

LOCATION: Madrid, Maine

Boring No.: BB-MSR-101

WIN: 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	933.1	**Auger ID/OD:**	
Operator:	Enos/Dube	**Datum:**		**Sampler:**	
Logged By:	Schonewald	**Rig Type:**		**Hammer Wt./Fall:**	
Date Start/Finish:	8/25/14; 1000 / 8/25/14; 1220	**Drilling Method:**		**Core Barrel:**	
Boring Location:	Sta 591+67, 6.5 RT	**Casing ID/OD:**		**Water Level*:**	

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
MV = Unsuccessful Insitu Vane Shear Test attempt

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOC = weight of casing
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
-- = not recorded
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
WC = water content, percent
G = grain size analysis

Depth (ft.)	Sample Information						Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows				
0									13.5 inches HMA	
	1D	24/13	1.0 - 3.0	13-14-11-9	25					
	2D	24/13	4.0 - 6.0	6-9-4-3	13					
5										
	R1	60/60	9.5 - 14.5	RQD = 68%%					R1: Hard, fresh, aphanitic to fine grained, light gray, HORNFELS; little to no evidence of relic bedding. Close to moderately spaced, predominately horizontal fractures; undulating, rough, fresh to slightly discolored, and partially open with occasional mud infilling.	
10	~~3D~~	~~3/3~~	~~9.5 - 9.8~~	~~20/3" (bounce)~~						
	R2	60/57	14.5 - 19.5	RQD = 63%%					R2: Same as R1, except very close fractures from 14.5 to 14.8 feet and 19.0 to 19.5 feet.	
15										
							913.6		19.5 **Bottom of Exploration at 19.5 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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid, Maine	**Boring No.:** BB-MSR-102 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	932.9	**Auger ID/OD:**	
Operator:	Enos/Dube	**Datum:**		**Sampler:**	
Logged By:	Schonewald	**Rig Type:**		**Hammer Wt./Fall:**	
Date Start/Finish:	8/25/14; 1240 / 8/25/14; 1445	**Drilling Method:**		**Core Barrel:**	
Boring Location:	Sta 591+79, 8.1 LT	**Casing ID/OD:**		**Water Level*:**	

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
MV = Unsuccessful Insitu Vane Shear Test attempt

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOC = weight of casing
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
-- = not recorded
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
WC = water content, percent
G = grain size analysis

Depth (ft.)	Sample Information								Visual Description and Remarks	Lab. Testing Results
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log		
0									12 inches HMA	
	1D	24/13	1.0 - 3.0	9-7-4-3	11					
	2D	24/13	4.0 - 6.0	2-3-2-3	5					
5										
	3D	22/11	9.0 - 10.8	2-2-3-15/4"	5					
10										
	R1	60/59	11.3 - 16.3	RQD = 47%%					R1: Hard, fresh, aphanitic to fine grained, light gray, HORNFELS; little to no evidence of relic bedding. Very close to moderately spaced, low and high angle fractures; undulating, rough, typically discolored, and open to moderately wide with occasional mud infilling. Very close fracture spacing from 12.8 to 13.6 feet and from 14.5 to 15.0 feet.	
15										
	R2	60/60	16.3 - 21.3	RQD = 93%%					R2: Same as R1, except moderately spaced fractures; 3 horizontal and 1 moderately dipping; no infilling.	
20										
							911.6		21.3 **Bottom of Exploration at 21.3 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.

Schonewald Engineering Associates, Inc.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid, Maine	**Boring No.:** BB-MSR-103 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	933.0	**Auger ID/OD:**	
Operator:	Enos/Dube	**Datum:**		**Sampler:**	
Logged By:	Schonewald	**Rig Type:**		**Hammer Wt./Fall:**	
Date Start/Finish:	8/25/14; 1455 / 8/26/14; 1030	**Drilling Method:**		**Core Barrel:**	
Boring Location:	Sta 590+89, 7.8 RT	**Casing ID/OD:**		**Water Level*:**	

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
MV = Unsuccessful Insitu Vane Shear Test attempt

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOC = weight of casing
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
-- = not recorded
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
WC = water content, percent
G = grain size analysis

Depth (ft.)	Sample Information								Visual Description and Remarks	Lab. Testing Results
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log		
0									14 inches HMA	
	1D	24/14	1.0 - 3.0	10-9-7-4	16					
	2D	24/14	4.0 - 6.0	2-1-2-1	3					
5										
	3D	24/10	9.0 - 11.0	1-1-3-3	4					
10										
	4D	24/7	14.5 - 16.5	16-22-23-28	45					
15										
	5D	17/11	19.0 - 20.4	17-34-50/5"	>84					
20	MR	60/5	20.3 - 25.3						possible cobble; driller notes drilling like "rotten" rock	
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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid, Maine	**Boring No.:** BB-MSR-103 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	933.0	**Auger ID/OD:**	
Operator:	Enos/Dube	**Datum:**		**Sampler:**	
Logged By:	Schonewald	**Rig Type:**		**Hammer Wt./Fall:**	
Date Start/Finish:	8/25/14; 1455 / 8/26/14; 1030	**Drilling Method:**		**Core Barrel:**	
Boring Location:	Sta 590+89, 7.8 RT	**Casing ID/OD:**		**Water Level*:**	

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
MV = Unsuccessful Insitu Vane Shear Test attempt

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOC = weight of casing
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
-- = not recorded
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
WC = water content, percent
G = grain size analysis

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
25	MR	8/6	25.3 - 26.0							
	6D	14/5	26.0 - 27.2	24-28-50/2"	>78					
	R1	53/53	27.2 - 31.6	RQD = 100%%					R1: Hard, fresh, aphanitic to fine grained, light gray, HORNFELS; little to no evidence of relic bedding. (TO BE CONT'D)	
30										
	R2	60/60	31.6 - 36.6	RQD = 100%%					R2: same as R1	
35										
							896.4		36.6 **Bottom of Exploration at 36.6 feet below ground surface.**	
40										
45										
50										

Remarks:

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

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

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid, Maine	**Boring No.:** BB-MSR-104 **WIN:** 22615.00

Driller:	Maine Test Borings	**Elevation (ft.)**	932.9	**Auger ID/OD:**	
Operator:	Enos/Dube	**Datum:**		**Sampler:**	
Logged By:	Schonewald	**Rig Type:**		**Hammer Wt./Fall:**	
Date Start/Finish:	8/26/14; 1035 / 8/26/14; 1420	**Drilling Method:**		**Core Barrel:**	
Boring Location:	Sta 591+01, 7.1 LT	**Casing ID/OD:**		**Water Level*:**	

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
MV = Unsuccessful Insitu Vane Shear Test attempt

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)
WOC = weight of casing
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
-- = not recorded
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
WC = water content, percent
G = grain size analysis

Depth (ft.)	Sample Information								Visual Description and Remarks	Lab. Testing Results
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log		
0									15 inches HMA	
	1D	24/12	1.0 - 3.0	24-19-11-8	30					
	2D	24/7	4.0 - 6.0	3-5-5-3	10					
5										
	3D	24/13	9.0 - 11.0	2-11-11-6	22					
10										
	4D	24/13	14.0 - 16.0	7-10-11-11	21					
15										
	5D	24/9	19.0 - 21.0	29-28-28-30	56					
20										
	6D	24/16	24.0 - 26.0	34-39-55-41	94					
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.

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Weymouth Bridge Route 4 over Sandy River **LOCATION:** Madrid, Maine	**Boring No.:** BB-MSR-104 **WIN:** 22615.00
Driller: Maine Test Borings	**Elevation (ft.)** 932.9	**Auger ID/OD:**
Operator: Enos/Dube	**Datum:**	**Sampler:**
Logged By: Schonewald	**Rig Type:**	**Hammer Wt./Fall:**
Date Start/Finish: 8/26/14; 1035 / 8/26/14; 1420	**Drilling Method:**	**Core Barrel:**
Boring Location: Sta 591+01, 7.1 LT	**Casing ID/OD:**	**Water Level*:**

Definitions:	Definitions:	Definitions:
D = Split Spoon Sample	S_u = Insitu Field Vane Shear Strength (psf)	-- = not recorded
MD = Unsuccessful Split Spoon Sample attempt	R = Rock Core Sample	LL = Liquid Limit
U = Thin Wall Tube Sample	RQD = Rock Quality Designation (%)	PL = Plastic Limit
MU = Unsuccessful Thin Wall Tube Sample attempt	WOC = weight of casing	PI = Plasticity Index
V = Insitu Vane Shear Test	WOH = weight of 140lb. hammer	WC = water content, percent
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR = weight of rods	G = grain size analysis

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Lab. Testing Results
25										
	R1	60/58	27.8 - 32.8	RQD = 82%%					R1: Hard, fresh, aphanitic to fine grained, light gray, HORNFELS; little to no evidence of relic bedding. (TO BE CONT'D)	
30										
	R2	60/60	32.8 - 37.8	RQD = 77%%						
35										
							895.1		37.8 **Bottom of Exploration at 37.8 feet below ground surface.**	
40										
45										
50										

Remarks:

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

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

APPENDIX E

ROCK MASS RATING AND ROCK MASS BEARING CAPACITY CALCULATIONS

SCHONEWALD ENGINEERING ASSOCIATES, INC.

Project: MaineDOT Weymouth Bridge Replacement		WIN 22615.00		**Proj. No.**	14-113
Location: Madrid Township, ME		**Last updated:**	7/7/15	**By**	IVS
Subject:	Geotechnical Calculations	**Checked:**	9/2/15	**By**	SJR
	Rock Mass Rating			**By**	

Objective:

Evaluate the strength and rating of the bedrock underlying the site.

Reference:

2010 LRFD Manual - Section 10.4.6.4 "Rock Mass Strength", including Tables 10.6.4.4-1 and 10.6.4.4-3
NAVFAC DM 7.1 - Chapter 1, Figure 3 (Strength Classification)
FHWA Geotechnical Engineering Circular No. 5, Section 6 "Interpretation of Rock Properties"

Data Sources:

Published bedrock geological information
Site-specific test boring logs, specifically rock core descriptions and RQDs.
Site-specific laboratory test results: unconfined compression tests on rock core samples.

TEST BORINGS: Rock observed underlying the Weymouth Bridge site is a hard, typically fresh, aphanitic to fine grained HORNFELS. Protolith rock is a pelite that underwent alteration as a result of a large granitic intrusion that is mapped in close proximity to the site.

LABORATORY TESTS: 2 laboratory uniaxial compressive strength tests, as follows

BB-MSR-101	12.0 to 12.8 ft BGS	UCT q_p = 2,700 ksf
BB-MSR-104	32.0 to 32.6 ft BGS	UCT q_p = 2,522 ksf

average peak uniaxial compressive strength: 2,611 ksf
strength is classified as " EXTREMELY STRONG" per NAVFAC DM7.1-Fig 3

RQDs FROM ROCK CORES OBTAINED AT WEYMOUTH BRIDGE:

BORING	ROCK CORE NO.	RQD
BB-MSR-101	R1	68
	R2	63
BB-MSR-102	R1	47
	R2	93
BB-MSR-103	R1	100
	R2	100
BB-MSR-104	R1	82
	R2	77
	$RQD_{average}$ =	79

Project: MaineDOT Weymouth Bridge Replacement	WIN 22615.00		**Proj. No.**	14-113	
Location: Madrid Township, ME	**Last updated:**	7/7/15	**By**	IVS	
Subject: Geotechnical Calculations	**Checked:**	9/2/15	**By**	SJR	
Rock Mass Rating			**By**		

ROCK MASS RATING MATRIX (Table 10.4.6.4-1)

ITEM #	DESCRIPTION	VALUE	REL RATING
1	strength of intact rock	avg uniaxial compressive strength: 2,611 ksf	10
2	drill core quality (RQD)	average RQD of 8 core runs = 79%	15
3	spacing of joints	typ. moderately spaced, with some close spacing	13
4	condition of joints	typically hard joint wall rock	18
		typically rough and undulating	
5	groundwater conditions	typically interstitial water	7
		RAW RMR (sum):	63

RMR ADJUSTMENT FOR JOINT ORIENTATION (Table 10.4.6.4-2)

-4 predominatley low angle joints are favorable to fair with respect to foundations

ADJUSTED ROCK CLASS NUMBER (Table 10.4.6.4-3)

RMR 59

Class No. III

Description Fair

Project: MaineDOT Weymouth Bridge Replacement	WIN 22615.00			**Proj. No.**	14-113
Location: Madrid Township, ME	**Last updated:**	7/7/15	**By**	IVS	
Subject: Geotechnical Calculations	**Checked:**	9/2/15	**By**	SJR	
Bearing Resistance of CIP Footing on Fractured Rock			**By**		

Service Limit State Bearing Resistance

Nominal Bearing Resistance = Factored Bearing Resistance (LRFD Manual Section 10.5.5.1)

Presumptive Bearing Resistance for Service Limit State ONLY
LRFD Manual Table C10.6.2.6.1-1 with:

Type of Bearing Material - massive crystalline metamorphic rock
Consistency in Place - hard sound rock

Bearing Resistance (ordinary range):	120 to 200 ksf	
Recommended Bearing Resistance:	**160 ksf**	SERVICE LIMIT STATE ONLY

(Note: use lesser of Recommended Bearing Resistance or Nominal Resistance of Concrete ($0.3f'_c$))

Project: MaineDOT Weymouth Bridge Replacement		WIN 22615.00		**Proj. No.**	14-113
Location: Madrid Township, ME		**Last updated:**	7/7/15	**By**	IVS
Subject:	Geotechnical Calculations	**Checked:**	9/2/15	**By**	SJR
	Bearing Resistance of CIP Footing on Fractured Rock (cont'd)			**By**	

Strength Limit State Bearing Resistance

Resistance Factor (φ_b) = 0.45 (LRFD Manual Table 10.5.5.2.2-1)

C_o (uniaxial compressive strength) = 2,611 ksf

Determine Rock Mass Rating: refer to Rock Mass Rating worksheets

ROCK CLASS NUMBER (Table 10.4.6.4-3)

RMR: 59

Class No.: III

Description: Fair

Determine Rock Type (2010 LRFD Manual Table 10.4.6.4-4)

Hornfels is a fine-grained dense rock produced by thermal metamorphism of phyllites as a result of contact with an intrusive igneous body

Hornfels is best characterized as Rock Type D (fine grained polyminerallic crystalline rc)

For use in 2014 LRFD Manual T10.4.6.4-1: metamorphic, nonfoliated, hornfels

Determine Rock Mass Material Parameters (2014 LRFD Manual T10.4.6.4-1)

$m = m_i$ exp((RMR-100)/14), where m_i = 19 for Intact Rock Type D m = 1.016

s = exp ((RMR-100)/6) s = 0.00108

Determine Rock Mass Nominal Bearing Resistance (q_{nom})

using LRFD methodology (Reference: Wyllie "Foundations on Rock" Eq. 5.4, Pg 138)

q_{nom} = C_{f1} * s^0.5 * C_o * [1 + (ms^-0.5 + 1)^0.5], where

q_{nom} is the ultimate bearing capacity of footings on broken or jointed rock

C_{f1} is a correction factor for the shape of the foundation (Wyllie Table 5.4) = 1.0 for strip

m and s are functions of rock type (D) and rock mass quality (Good) (see above)

q_{nom} = (1.0) * (0.00108)^0.5 * 2,611 * [1 + ((0.909)(0.00108)^-0.5 + 1)^0.5]

q_{nom} = 570.1 ksf

Determine Rock Mass Factored Bearing Resistance

q_{nom}= 570 ksf and φ_b = 0.45

qr = 257 ksf

Recommended Bearing Resistance: **250 ksf** STRENGTH LIMIT STATE