FINAL GEOTECHNICAL DESIGN REPORT
U.S. ROUTE 1 BYPASS BRIDGE OVER ROUTE 236
MAINEDOT WIN 19283.00, BRIDGE NO. 03860
KITTERY, MAINE

by

Haley & Aldrich, Inc.
Portland, Maine

for

McFarland-Johnson, Inc.
Concord, New Hampshire

File No. 38239-100
22 November 2013

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Attention: Gene McCarthy, P.E.
Project Manager

Subject: Final Geotechnical Design Report
U.S. Route 1 Bypass Bridge over Route 236
MaineDOT PIN 19283.00, Bridge No. 03860
Kittery, Maine

Dear Mr. McCarthy:

We are pleased to submit our report titled, "Final Geotechnical Design Report, U.S. Route 1 Bypass Bridge over Route 236, MaineDOT PIN 19283.00, Bridge No. 03860, Kittery, Maine" prepared in accordance with our proposal dated 11 January 2013, revised 21 February 2013, and your subsequent authorization.

This Final Geotechnical Design Report (GDR) is a compilation of the results of subsurface investigations and geotechnical laboratory testing programs and provides final geotechnical design recommendations in support of the subject project.

Thank you for the opportunity to help support McFarland-Johnson, Inc. (M-J) and the Maine Department of Transportation (MaineDOT) on this project. We look forward to providing continued assistance to M-J during the contract document preparation, bidding and construction phases of the project.

Sincerely yours,
HALEY & ALDRICH, INC.

Erin F. Wood, P.E.
Senior Geotechnical Engineer | Project Manager

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Lead Geotechnical Engineer | Vice President

Enclosures
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1. INTRODUCTION

This report presents the results of preliminary and final design phase geotechnical investigations, laboratory testing, engineering evaluations and geotechnical design recommendations prepared by Haley & Aldrich, Inc. (Haley & Aldrich) for McFarland-Johnson (M-J) for the proposed replacement of the U.S. Route 1 Bypass Bridge over Route 236 in Kittery, Maine (refer to Sheet 1, Project Locus).

1.1 Existing Site Conditions

1.1.1 Existing Bridge Structure

The existing bridge carries U.S. Route 1 Bypass (four lanes) over Route 236 (two lanes), immediately northwest of the traffic circle at the intersection of Route 236 and Route 1. Based on our review of historic bridge plans provided by M-J (refer to Appendix C), the existing bridge structure was constructed in 1942 and is approximately 47-ft long and 71 to 74-ft wide. The bridge structure is a single span, supported on approximately 19-ft tall abutments. We understand that the abutments are supported by shallow "footings" bearing on bedrock. These footings appear to have been constructed by placing concrete in depressions or trenches in the rock. Approximately 25-ft long wingwalls retain embankment fill behind the abutments.

According to the historic drawings, the existing Route 236 roadway section consists of 18 in. of gravel base, constructed directly on bedrock and "ledge debris."

1.1.2 Terrain

U.S. Route 1 Bypass is located on 5 to 10-ft high embankments approaching the existing bridge. The existing ground surface along the top of the embankments is relatively level, ranging from approximately El. 45 to El 46. The embankments slope down to approximately El. 35 to El. 40, at 1.5H:1V to 2H:1V slopes. The ground surface slopes down again to Route 236, which is at approximately El. 27 to El. 28 beneath the bridge.

Refer to Sheet 2, Site and Subsurface Exploration Location Plan, for existing site conditions, and Sheet 3, Interpretive Subsurface Profile, for a graphic interpretation of existing ground surface along the U.S. Route 1 Bypass baseline.

1.2 Proposed Bridge Structure

The replacement bridge will be a single-span structure, constructed along the same alignment as the existing bridge. The bridge length will be increased to 60 ft to accommodate potential future sidewalks on either side of Route 236 and wider lanes within Route 236. The bridge width will be increased from approximately 71 ft to approximately 90 ft.

The vertical profile of U.S. Route 1 Bypass roadway will be raised slightly in the vicinity of the new bridge, to accommodate the depth of the new bridge structure and to increase the vertical clearance requirements beneath the bridge. The current planned maximum raise-in-grade is approximately 3 ft. Currently no changes in vertical profile are planned for the portion of Route 236 adjacent to and below the existing bridge.

1.3 Horizontal Coordinate System and Elevation Datum

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone (Refer to Sheet 4). Elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

2. GEOLOGIC SETTING

According to the Maine Geological Survey's Kittery Quadrangle, Maine (1999), surficial geologic units mapped within the site vicinity and the region consist largely of Marine Nearshore Deposits and Presumpscot Formation. However, the primary soils encountered in the test borings conducted at the site consisted of man-placed fill.

The man-placed fill generally consisted of well graded sand with gravel and varying amounts of silt. Fill soils were encountered in all explorations conducted for this study. Blasted rock fill was encountered in one exploration overlying bedrock.

Glaciomarine sediments of the Presumpscot Formation consisting of silt and clay with sand lenses were encountered beneath the fill in five of the test borings. A thin layer of dense glacial till composed of a heterogeneous mix of sand, silt, and gravel was encountered beneath the Presumpscot Formation in two of the test borings. The glacial till directly overlies the bedrock surface.

Bedrock at the site is mapped as Late Ordovician to Early Silurian age rocks of the Kittery Formation, part of the Merrimack Group. These rocks are composed of metamorphosed shale, siltstone and sandstone, with variably thin to thick bedding and other well developed structural features such as folding and graded bedding (Hussey 1962). Phyllite and slate were encountered in subsurface explorations at the site. There are no mapped faults within the site and vicinity. The closest mapped fault is the Portsmouth fault of Permian age, located approximately 2 miles south of the site (Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine and New Hampshire, 2008).

3. SUBSURFACE EXPLORATION PROGRAMS

3.1 Historic Explorations by Others

No subsurface explorations are shown on the historic plans for the original construction of the bridge (refer to Appendix C). However the historic plans do indicate that relatively shallow bedrock was present at the existing abutments, and the bedrock may have been excavated or blasted to construct the existing bridge and Route 236 roadway section.

3.2 Preliminary Phase Explorations by Haley & Aldrich

Haley & Aldrich conducted a preliminary phase geotechnical exploration program at the site. All test borings were drilled by Maine Test Borings of Hermon, Maine. In total, seven borings were drilled along the existing and proposed bridge alignment in order to identify general subsurface conditions. "As-drilled" locations of the borings are shown on Sheet 2. Coordinate location data and ground surface elevation at exploration locations are provided on individual test boring logs provided in Appendix A and are summarized in Table I and II (Sheet 4 and 5). All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. The samples that were not submitted for laboratory testing are available for review upon request. Soil and bedrock samples are being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

A total of five borings (BB-KDH-101, BB-KDH-101A, BB-KDH-102, BB-KDH-105 and BB-KDH-106) were drilled in U.S. Route 1 Bypass. Two borings (BB-KDH-103 and BB-KDH-104) were drilled in Route 236, as close as possible to the existing bridge. Boring locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features. "As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using optical survey equipment.

Subsurface explorations were drilled using a truck-mounted Mobile B53 drill rig. Test borings were drilled to depths ranging from approximately 3 to 32 ft below ground surface (BGS) using 3.0-in. (NW-size) inside diameter (ID) steel casing or hollow stem augers (for shallow borings). Soil samples were generally collected at standard, 5-ft intervals by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. Soil samples were collected continuously in borings conducted for the approach roadways and for the Route 236 pavement design. Drilling and sampling were performed in accordance with MaineDOT specifications.

Each drill rig was equipped with a standard rope and cathead and safety hammer per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

The two borings at the proposed bridge abutments (BB-KDH-101 and BB-KDH-106) and one of the borings in Route 236 (BB-KDH-103) sampled at least 5 ft of bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel. Borings BB-KDH-101 and BB-KDH-106 were extended to depths of 13 to 14 ft into bedrock to investigate the presence of fractured zones within the rock.

Soil samples obtained during explorations were screened with a photoionization detector (PID) to assess the presence of volatile organic compounds (VOCs). No elevated PID readings were detected in screened soil samples collected from these explorations.

3.3 Final Design Phase Explorations by Haley & Aldrich

Haley & Aldrich also conducted a final design phase geotechnical exploration program at the site after completion of preliminary design. All test borings were drilled by Maine Test Borings of Hermon, Maine. In total, four borings, fourteen auger probes and four hand auger probes were drilled along the existing and proposed bridge alignment in order to identify general subsurface conditions. "As-drilled" locations of the borings are shown on Sheet 2. Coordinate location data and ground surface elevation at exploration locations are provided on individual test boring logs provided in Appendix A and are summarized in Table I and II (Sheet 4 and Sheet 5). All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. The samples that were not submitted for laboratory testing are available for review upon request. Soil and bedrock samples are being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

Two borings (BB-KDH-202 and BB-KDH-203) and fourteen auger probes (BP-KDH-205 through BP-KDH-211 and BP-KDH-214 through BP-KDH-219A) were drilled in U.S. Route 1 Bypass. Two borings (BB-KDH-201 and BB-KDH-204) were drilled at proposed southwest and northeast wing wall locations respectively. Four hand auger probes (BP-KDH-212, BP-KDH-213 and BP-KDH-221, BP-KDH-222) were conducted at proposed southeast and northwest wing wall locations respectively. Exploration locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features. "As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using optical survey equipment.

Subsurface explorations were drilled using a track-mounted Mobile B53 drill rig. Test borings were drilled to depths ranging from approximately 6 to 17 ft below ground surface (BGS) using 3.0-in. (NW-size) inside diameter (ID) steel casing. Soil samples were generally collected at standard, 5-ft intervals by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. Drilling and sampling were performed in accordance with MaineDOT specifications.

Each drill rig was equipped with a standard rope and cathead and safety hammer per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

Bedrock core samples were obtained in borings BB-KDH-201 through BB-KDH-204 using a 2.0-in. (NQ-size) ID diamond-tipped core barrel. Approximately 4.5 to 5.0 ft of rock core was recovered at each test boring location to assess the near surface condition of the bedrock. Refer to Appendix A for details of the bedrock core.

Soil samples obtained during explorations were screened with a photoionization detector (PID) to assess the presence of volatile organic compounds (VOCs). No elevated PID readings were detected in screened soil samples collected from these explorations.

4. GENERALIZED SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site consist of the following geologic units presented in order of increasing depth below ground surface: bituminous concrete/Portland cement concrete/fill, blasted rock, marine clay deposit, glacial till and bedrock. Refer to Sheet 3 for a graphic interpretation of the subsurface soil conditions along the proposed project alignment and Table II (Sheet 5) for a summary of the soil units and encountered thicknesses. A description of each soil unit is provided separately, below. Detailed soil descriptions are provided on the test boring logs in Appendix A.

Please note that the soil descriptions provided on the test boring logs, summarized below and shown on the geologic profile (Sheet 3) do not represent actual field conditions other than at the specific test boring locations. The actual conditions will vary from those described and shown herein.

4.1 Soil Unit and Bedrock Descriptions

4.1.1 Bituminous Concrete/Portland Cement Concrete/Fill

Bituminous concrete was encountered at ground surface in the Route 1 Bypass and Route 236 roadway borings and probes was approximately 10 to 13 in. thick. Portland cement concrete was encountered in the Route 1 Bypass roadway borings below the bituminous concrete and was approximately 7 to 14 in. thick. It is possible that the upper bituminous concrete is a pavement overlay installed sometime after initial roadway/bridge construction.

Man-placed fill was encountered in the Route 1 Bypass and Route 236 roadway borings and likely in the roadway auger probes beneath the bituminous concrete and Portland cement concrete. Man-placed fill was also encountered in all wingwall borings, auger probes and hand probes at ground surface. The fill consisted primarily of fine to medium SAND, little to trace coarse sand, some to trace gravel, trace silt OR silty SAND, trace coarse sand, trace gravel OR sandy GRAVEL. The only cohesive fill encountered was an approximate 1.5-ft thick fill layer containing reworked silty CLAY, little fine sand, trace coarse sand and gravel encountered beneath the granular fill in boring BB-KDH-201.

Topsoil was encountered in borings BB-KDH-201 and BB-KDH-204 (overlying granular fill) and consisted of an approximately 0.5-ft thick layer of medium stiff to stiff SILT, little to trace fine sand. The thickness of the fill ranged from approximately 2 to 3 ft in Route 236, 9 to 16 ft in U.S. Route 1 Bypass test borings and from 1 to 6 ft in the wing wall explorations. The fill was loose to dense with SPT N-values ranging from 8 to 65.

4.1.2 Blasted Rock

Highly fractured rock was encountered in boring BB-KDH-103 drilled in Route 236. It is likely, in our opinion, that the fractured rock was caused by blasting activities that were needed to construct this section of Route 236, or to construct the existing abutments. The “overblasted” fractured rock was 1.5 ft thick and contained rock pieces with little sand and trace silt.

4.1.3 Marine Clay Deposit

A thin deposit of marine clay was encountered in borings BB-KDH-105, BB-KDH-201 through BB-KDH-204 drilled in the existing Route 1 Bypass bridge, roadway, and proposed east and west wing wall locations. The deposit was approximately 2 to 5 ft thick, consisting of medium stiff silty CLAY

with sand lenses to hard, silty CLAY, little fine to coarse sand, little gravel, with occasional sand layers OR soft, SILT, trace fine to medium sand, trace clay, trace organics and SPT N-values ranging from 4 to 58.

4.1.4 Glacial Till

Glacial till was encountered in boring BB-KDH-105 drilled in Route 1 Bypass approximately 100 ft north of the existing bridge and boring BB-KDH-201 drilled at the proposed west wing wall location. The deposit consisted of fine to medium SAND with little coarse sand, little gravel, little to trace silt. The deposit was approximately 2-ft thick, medium dense to very dense, with an SPT N-value ranging from 22 to 56.

4.1.5 Weathered Bedrock

A thin layer (0.2 to 2-ft thick) of weathered bedrock was encountered in six borings and three auger probes. The presence of weathered rock was identified primarily by drilling behavior and the presence of rock fragments in the drill wash water.

4.1.6 Bedrock

Bedrock or probable bedrock was encountered in each test boring, except BB-KDH-101A which was terminated on a shallow obstruction. Bedrock was cored in four borings drilled at Route 1 Bypass bridge abutments (BB-KDH-101, BB-KDH-106, BB-KDH-202 and BB-KDH-203), one of the borings in Route 236 (BB-KDH-103) and in two borings drilled at the east and west wing wall locations (BB-KDH-201 and BB-KDH-204).

The top of the bedrock surface in the test borings ranged from approximately 3 to 5 ft BGS (El. 22.6 to El. 25) in Route 236 and 10 to 19 ft BGS (El. 26.8 to El. 33.5) in U.S. Route 1 Bypass, and 6 to 12 ft BGS (El. 24.8 to El. 25.7) at the east and west wing wall locations. Bedrock encountered at the site consists of moderately hard to hard SLATE and hard to soft PHYLLITE. Highly fractured zones were encountered within the bedrock.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the core run. RQD values for bedrock encountered at the site typically ranged between 30 and 85 percent. However RQD values less than 30 percent (i.e. 0 to 24 percent) were measured within 2 to 5 ft of the soil/rock interface in the borings drilled in the area we suspect blasting was previously conducted (i.e. BB-KDH-103 and BB-KDH-106). Low RQD values (0 to 14 percent) were also recorded in Boring BB-KDH-204 at the proposed east wing wall location which encountered a highly fractured, moderately severely weathered zone within 4.5 ft of the bedrock surface.

It is our opinion that the presence of vertical and high angle joints contributes to the low RQD values.

4.2 Groundwater Conditions

Water was used in the drilling process; therefore actual groundwater levels were not determined in the explorations. Groundwater levels can be expected to fluctuate, subject to seasonal variation, local soil conditions, topography and precipitation. Water levels encountered during construction may differ from those observed in the test borings or observation wells.

5. LABORATORY TESTING PROGRAM

A preliminary laboratory testing program was undertaken to assist in soil classification/identification and to determine engineering properties of representative rock samples collected during the field investigations. In general, laboratory testing was performed on disturbed soil samples collected during SPT sampling or rock core samples. All laboratory soil testing was performed by GeoTesting Express of Acton, Massachusetts. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. Laboratory testing and results are summarized below. Laboratory test results are provided in Appendix B.

5.1 Preliminary Phase Laboratory Testing and Results

The preliminary phase testing program included four grain size analyses (sieve only) and four unconfined compressive strength tests including elastic modulus determination on bedrock samples. A summary of laboratory test results is provided below.

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Percent Finer than #200 Sieve (%)	USCS Classification	Strata
BB-KDH-101	2D	5.0 to 7.0	15.1	SW	Fill
BB-KDH-102	2D	2.5 to 4.5	24.6	SM	Fill
BB-KDH-105	2D	3.5 to 5.0	15.5	GM	Fill
BB-KDH-106	2D	5.0 to 7.0	19.5	SM	Fill

Test Boring No.	Rock Core No.	Core Depth (ft)	Peak Compressive Stress (psi)	Young's Modulus (psi)[1]	Bedrock Type
BB-KDH-101	R1	12.8 to 13.1	41,002	12,400,000	Slate
BB-KDH-101	R2	15.7 to 16.0	30,720	12,200,000	Slate
BB-KDH-106	R2	22.5 to 22.9	12,670	11,200,000	Phyllite
BB-KDH-106	R5	30.3 to 30.7	14,797	13,400,000	Phyllite

Note 1: Young's modulus reported above is that calculated for low strains. Young's modulus values at other strains are provided in Appendix B.

6. GEOTECHNICAL DESIGN RECOMMENDATIONS

Geotechnical design recommendations for the subject project were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Sixth Edition, 2012 with June 2012 errata; and
- MaineDOT Bridge Design Guide (BDG), August 2003 with Interim Revisions through August 2008.

6.1 Approach Embankment Design Considerations

The maximum raise-in-grade of approach embankments is approximately 3 ft. Bedrock at the site is relatively shallow and compressible materials are generally not present. A thin (4.5-ft thick maximum) layer of marine clay and silt is present at several locations. The material is typically medium stiff. Post construction settlement of the embankments due to consolidation of the marine clay is anticipated to be less than ¼ in. Refer to Appendix D, Calculations, for supporting documentation.

Elastic settlement of the existing granular materials present at the approach embankments will occur rapidly during placement of new embankment fill, prior to construction of the roadway section. Therefore we do not consider special embankment construction, such as lightweight fill, prefabricated vertical drain installation or a preload/surcharge program, to be required to limit post-construction settlements.

6.2 Seismic Design/Liquefaction Considerations

Based on the corrected SPT N_{60} values obtained from the test borings (since the soils are primarily granular), the site is considered Site Class "C" in accordance with LRFD Table 3.10.3.1-1. Refer to Appendix D, Calculations, for supporting documentation.

Since the deposits consist primarily of granular materials that are not saturated, it is our opinion that the site soils are not liquefaction susceptible.

6.3 Bridge Abutment Foundation Design Recommendations

As shown on the interpretive subsurface profile (Sheet 3), the subsurface conditions primarily consist of existing fill overlying bedrock. The bedrock is considered suitable for support of the bridge superstructure. Based on the depth to the suitable foundation bearing strata, we consider spread footings on intact bedrock to be the most practicable and cost effective foundation alternative.

We recommend that soil, weathered rock, and disturbed rock (if encountered) be removed from beneath the proposed footings and replaced with lean concrete as needed prior to footing construction.

Specific foundation design recommendations follow (refer to Appendix D, Calculations, for supporting documentation):

- Bearing Resistance
  - For the service limit state, mass concrete footings should be designed such that footing contact pressures do not exceed 30 kips per square foot (ksf). At this pressure, it is estimated that settlement of footings bearing on bedrock will be less than 1 in. as stated in LRFD Article 10.6.2.6.1.
  - For the strength limit state, footings should be designed for a factored bearing resistance (based on Rock Mass Rating and Rock Type) of 45 ksf, using a resistance factor of 0.45.
  - For the extreme limit state, footings should be designed for a factored bearing resistance of 100 ksf, using a resistance factor of 1.0.
- Sliding resistance between the base of the concrete and rock subgrade should be calculated using a coefficient of sliding friction of 0.7.
  - Resistance factor for the strength limit state should be 0.85
  - Resistance factor for the extreme event limit state should be 1.0
  - Values provided are in accordance with LRFD Table 10.5.5.2.2-1
- Mass footings on bedrock require no minimum embedment depth in accordance with the BDG.

6.4 Wingwall Foundation Design Recommendations

The proposed wingwalls will consist of precast concrete modular gravity walls (e.g. T-Wall or DoubleWal). We recommend existing fill be removed to expose the bedrock surface, and granular borrow be placed/compacted up to the proposed wingwall bearing elevation, if needed.

Specific foundation design recommendations follow (refer to Appendix D, Calculations, for supporting documentation):

- Bearing Resistance
  - For the service limit state, mass concrete footings should be designed such that footing contact pressures do not exceed 8 kips per square foot (ksf). At this pressure, it is estimated that settlement of footings bearing on bedrock will be less than 1 in. as stated in LRFD Article 10.6.2.6.1.
  - For the strength limit state, footings should be designed for a factored bearing resistance (based on Rock Mass Rating and Rock Type) of 8 ksf for footing widths up to 8 ft, and 10 ksf for footing widths up to 14 ft, using a resistance factor of 0.45.
  - For the extreme limit state, footings should be designed for a factored bearing resistance of 18 ksf for footing widths up to 8 ft, and 22 ksf for footing widths up to 14 ft, using a resistance factor of 1.0.

- Sliding resistance between the base of the concrete and granular fill subgrade should be calculated using a coefficient of sliding friction of 0.55.
  - Resistance factor for the strength limit state should be 0.8
  - Resistance factor for the extreme event limit state should be 1.0
  - Values provided are in accordance with LRFD Table 10.5.5.2.2-1

6.5 Abutment and Wingwall Lateral Earth Pressure Recommendations

Based on discussions with M-J and their evaluation of anticipated magnitude of lateral movement of the abutment, we recommend the abutment be designed using active earth pressures. We recommend that abutments and wingwalls be designed for static lateral earth pressures using an equivalent fluid unit weight of 34 pounds per cubic foot (pcf) which assumes an active earth pressure coefficient of 0.275 and a soil unit weight of 125 pcf. This recommendation assumes the granular soil behind abutments and wingwalls will be drained and no unbalanced hydrostatic pressures will develop behind the abutments and wingwalls.

In addition, walls should be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should be applied to abutments and walls to account for the live load surcharge.

If determined applicable by M-J, the walls and abutments should be designed for a uniform lateral load to account for seismic soil loading in accordance with LRFD Article A.11.1. Based on the seismic site class (Site Class "C"), we recommend a uniform lateral load (in psf) equal to 19 times the wall height (in ft). Note that this soil pressure includes both the static and seismic lateral earth loads.

Refer to Appendix D, Calculations, for supporting documentation.

7. CONSTRUCTION CONSIDERATIONS

7.1 Subgrade Preparation

Prior to construction of the mass concrete footings bearing on rock, all existing soil, weathered bedrock, and disturbed bedrock should be removed. If the intact bedrock is lower than the design bottom of foundation, backfill to the design elevation with concrete fill or structural concrete.

Bedrock footing subgrades should be prepared as flat as practical, with all areas of the subgrade flatter than 6 horizontal to 1 vertical (6H:1V). Where the slope of the bedrock surface exceeds 6H:1V, the bedrock surface should be benched.

7.2 Bedrock Removal

Up to 9 ft of bedrock removal will be required to construct the new abutments and wingwalls. It is our opinion that either hoe ramming or controlled blasting methods are feasible for rock removal. It is our understanding that the bridge will be taken out of service during the accelerated 30-calendar day construction duration. Because of this, the maximum amount of time available to complete bedrock removal and maintain the accelerated schedule is less than ten days. Based on our understanding of the plan and profile limits of rock removal required, it is our opinion that rock removal using controlled blasting methods could be completed close to, but slightly over, the desired 10-day time period. In general it is our experience that hoe ramming will be slower than blasting and therefore we anticipate that rock removal using this method will require more than 10 days to complete.

7.3 Construction Monitoring

The geotechnical design and earthwork recommendations summarized herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation and approach embankment construction activities is required to enable the geotechnical engineer to keep in contact with procedures and techniques used during construction. Therefore, we recommend that an individual representing the design team, qualified by geotechnical training and experience be present at the site to provide monitoring during the approach embankment and foundation construction activities listed below:

- Preparation of the footing bearing surfaces.
- Placement and compaction of compacted fills.

8. LIMITATIONS OF RECOMMENDATIONS

This report is prepared for the exclusive use of M-J and the MaineDOT relative to the U.S. Route 1 Bypass Bridge over Route 236 in Kittery, Maine. There are no intended beneficiaries other than M-J and the MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than M-J and the MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from M-J, MaineDOT, and from Haley & Aldrich. Use of this report by such other person or entity without the written authorization of M-J, MaineDOT, and Haley & Aldrich shall be at such other person's or entity's sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

Use of this Report by any person or entity, including by M-J and the MaineDOT, for a purpose other than the U.S. Route 1 Bypass Replacement Bridge over Route 236 in Kittery, Maine is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the Report is adequate for such other use. Use of this Report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

REFERENCES

1. Bothner, Wallace A., Hussey, Arthur M. and Peter J. Thompson (2008), *Kittery Quadrangle, Maine-New Hampshire, Bedrock Geology*, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 08-78, 2008.

2. Hussey, Arthur M., *The Geology of Southern York County, Maine*, Maine Geological Survey, Department of Economic Development, Augusta, Maine, Special Geologic Studies Series No. 4,1962.

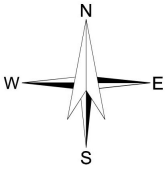
3. O'Toole, Patrick B., and J. Michael Clinch, *Surficial Geology of the Kittery 7.5-minute Quadrangle, York County, Maine*, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 99-119, 1999.

4. Haley & Aldrich, Inc., *Preliminary Geotechnical Design Report, U.S. Route 1 Bypass Bridge Over Route 236, MaineDOT PIN 19283.00, Kittery, Maine*,

G:\PROJECTS\38239 - Kittery Overpass Bridge\100\Final GDR\2013-1122-HAI-Final GDR-F.docx

SHEETS

SITE COORDINATES:43°6'4"N 70°44'43"W



U.S.G.S. QUADRANGLE: KITTERY, ME

HALEY & ALDRICH

U.S. ROUTE 1 BYPASS BRIDGE OVER ROUTE 236
MAINEDOT WIN 19283.00
KITTERY, MAINE

PROJECT LOCUS

SCALE: 1:24,000
NOVEMBER 2013

SHEET 1

38239-000 1.PDF

NOTES:

1. EXISTING SITE CONDITIONS, CONTOURS OF EXISTING GROUND SURFACE ELEVATIONS, LOCATION OF EXISTING SITE FEATURES AND PROPOSED BRIDGE LOCATION ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY MCFARLAND-JOHNSON, DATED 18 SEPTEMBER 2013.

2. AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING OPTICAL SURVEY EQUIPMENT.

3. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).

4. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH PERSONNEL.

5. SUBSURFACE EXPLORATIONS WERE DRILLED BY MAINE TEST BORINGS OF HERMAN, MAINE.

6. REFER TO APPENDIX A FOR LOGS OF TEST BORINGS.

7. REFER TO SHEET 3 FOR INTERPRETIVE SUBSURFACE PROFILE.

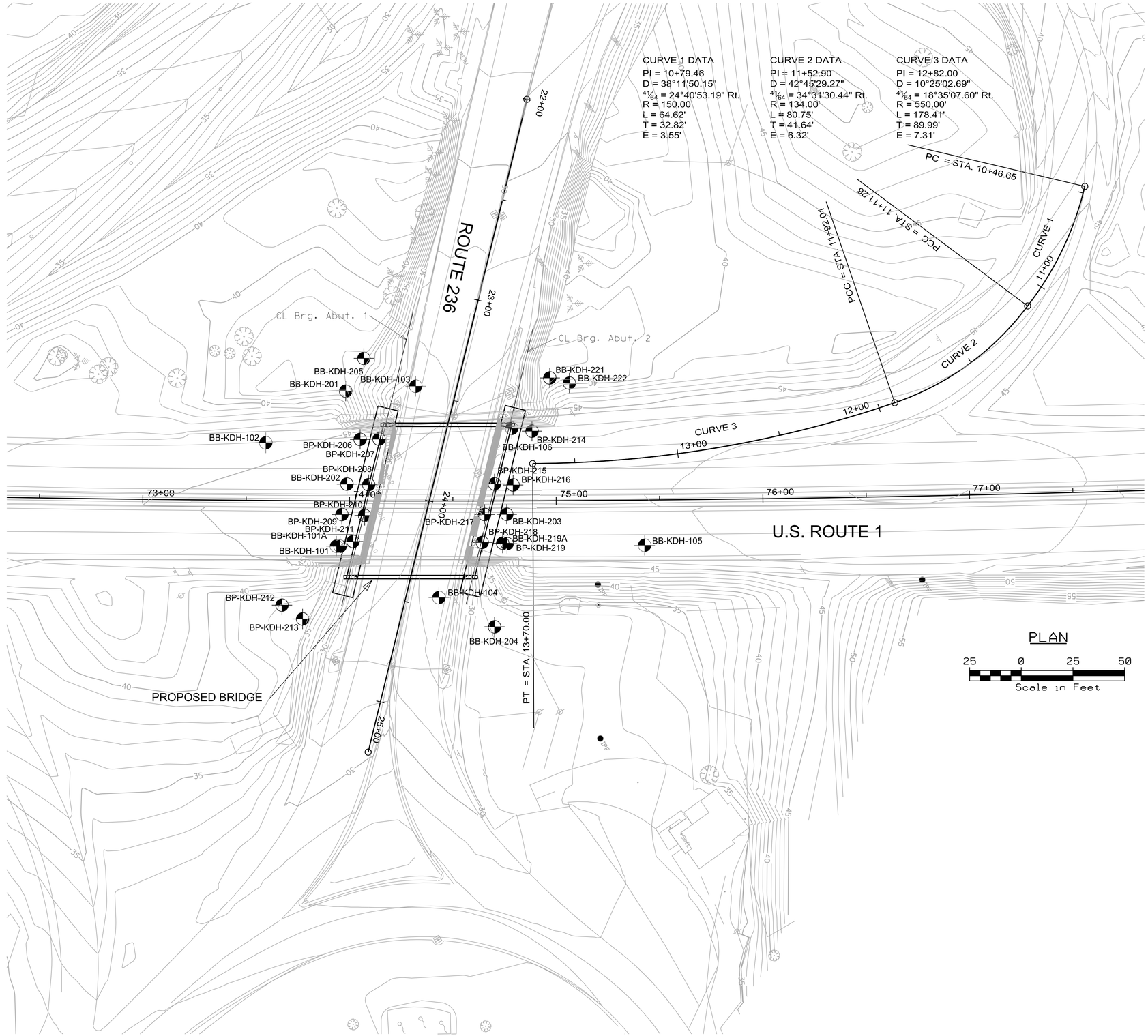
LEGEND:

BB-KDH-103 — DESIGNATION AND LOCATION OF TEST BORING DRILLED IN JANUARY 2012

BB-KDH-202 — DESIGNATION AND LOCATION OF TEST BORING DRILLED IN AUGUST 2013

DESIGNATION AND LOCATION OF TEST PROBE DRILLED IN AUGUST 2013

40 — ELEVATION CONTOUR OF EXISTING GROUND SURFACE



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 19283.00
BRIDGE NO. 03860
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	L. TIMBERLAKE		
DESIGN-DETAILED	MLS	PGL	9/13
CHECKED-REVIEWED	EFW	WAC	11/13
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

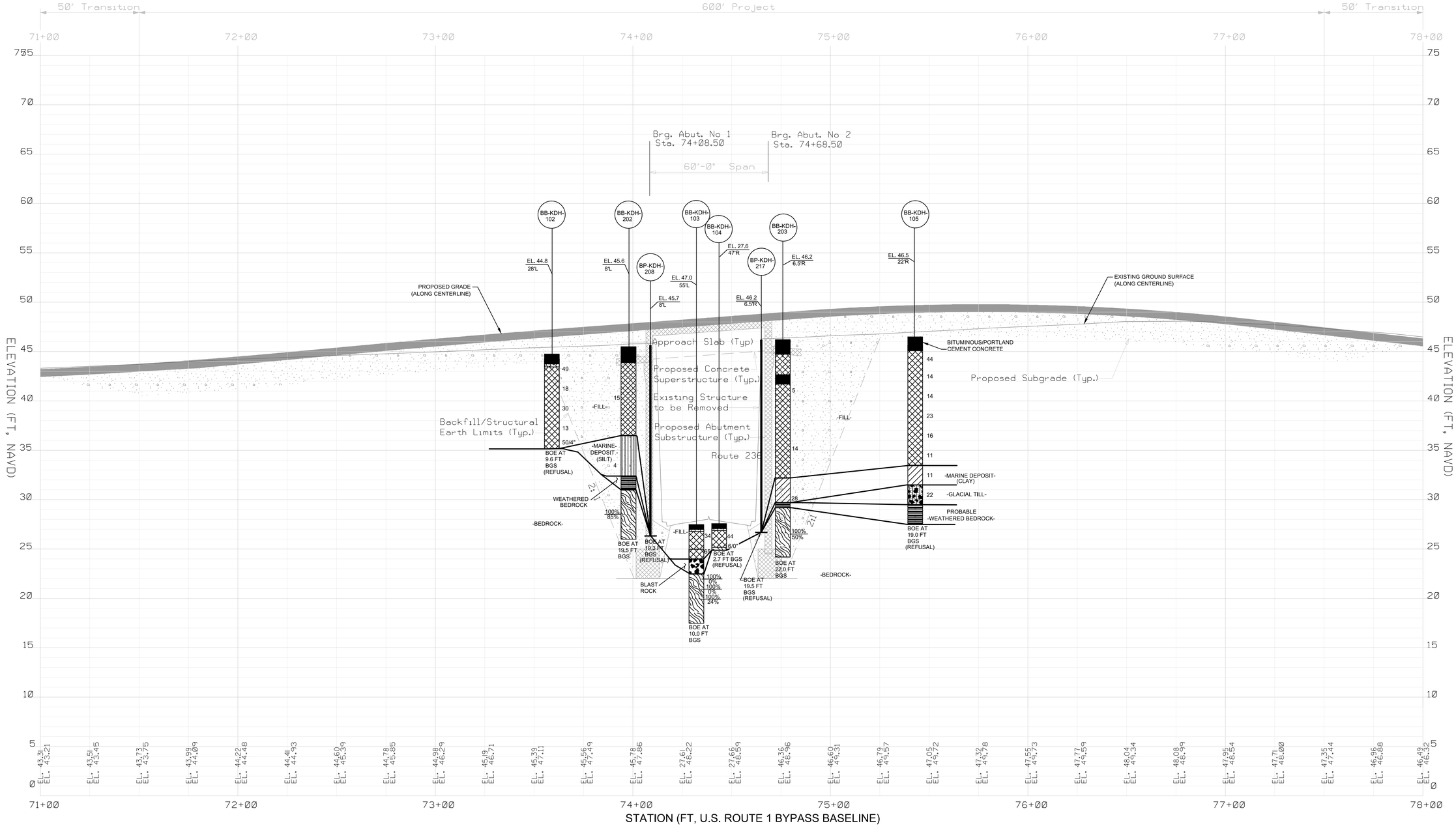
DATE

KITTERY OVERPASS BRIDGE
OVER STATE ROUTE 236
KITTERY YORK COUNTY

SITE AND SUBSURFACE
EXPLORATION LOCATION PLAN

SHEET NUMBER
2
OF 5

Filename: 38239-100-0007.dgn Division: HA Username: pgl Date: 2013-1106



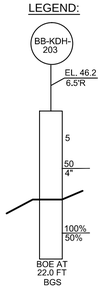
PROFILE

HORIZ. 25 0 25 50

VERT. 5 0 5 10

SCALE

LEGEND:



TEST BORING OR PROBE DESIGNATION

GROUND SURFACE ELEVATION AT TEST BORING OR PROBE
APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND DIRECTION MEASURED FROM PROPOSED CENTERLINE

CORRECTED STANDARD PENETRATION TEST (SPT) N-VALUE (N60 BLOWS PER FOOT)

50 BLOWS, 4 INCHES OF PENETRATION

STRATA INTERFACE

PERCENT RECOVERY
PERCENT ROCK QUALITY DESIGNATION (RQD)

DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE DEPTH SHOWN

NOTES:

1. THE SUBSURFACE PROFILE IS LOCATED ALONG THE PROPOSED ROUTE 1 BYPASS CENTERLINE.
2. EXISTING AND PROPOSED GROUND SURFACE ELEVATIONS ALONG THE SUBSURFACE PROFILE LINE WERE TAKEN FROM THE ELECTRONIC FILE ENTITLED "PROFILE H&A.dgn," PROVIDED BY MCFARLAND-JOHNSON ON 18 SEPTEMBER 2013.
3. THE "AS-DRILLED" TEST BORING LOCATIONS AND GROUND SURFACE ELEVATIONS WERE DETERMINED BY MAINEDOT USING OPTICAL SURVEY EQUIPMENT UPON COMPLETION OF DRILLING.
4. LINES REPRESENTING CHANGES IN STRATA ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS. THESE INTERPRETED STRATA LINES DO NOT REPRESENT ACTUAL FIELD CONDITIONS OTHER THAN AT SPECIFIC EXPLORATION LOCATIONS. ACTUAL FIELD CONDITIONS ALONG THE SUBSURFACE PROFILE WILL VARY FROM THOSE SHOWN.
5. REFER TO APPENDIX A FOR TEST BORING LOGS.
6. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 19283.00

BRIDGE PLANS

BRIDGE NO. 03860

PROJ. MANAGER	L. TIMBERLAKE	BY	DATE
DESIGN-DETAILED	MLS	PGL	10/08/13
CHECKED-REVIEWED	EPW	WAC	10/28/13
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

KITTERY OVERPASS BRIDGE
OVER STATE ROUTE 236
KITTERY YORK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

Date: 19 September 2013 Username: Division: Filename: J:\BENTLEY\WORKSPACE\PROJECTS\18239_MEDOT\18239-100006_PROF.DWG

TABLE I
SUMMARY OF SUBSURFACE EXPLORATIONS - LOCATION DATA
U.S. ROUTE 1 BYPASS BRIDGE OVER ROUTE 236
MAINEDOT WIN 19283.00
KITTERY, MAINE

Test Boring No.[1]	Coordinates[2]	
	Easting	Northing
BB-KDH-101	885,366.8	97,820.9
BB-KDH-101A	885,367.8	97,822.4
BB-KDH-102	885,306.3	97,817.7
BB-KDH-103	885,320.6	97,893.9
BB-KDH-104	885,413.8	97,850.3
BB-KDH-105	885,443.9	97,948.5
BB-KDH-106	885,362.2	97,922.9
BB-KDH-201	885,303.2	97,863.0
BB-KDH-202	885,342.7	97,839.6
BB-KDH-203	885,396.4	97,899.3
BB-KDH-204	885,440.8	97,865.3
BP-KDH-205	885,294.0	97,879.0
BP-KDH-206	885,327.2	97,856.6
BP-KDH-207	885,331.9	97,864.7
BP-KDH-208	885,348.5	97,848.6
BP-KDH-209	885,354.2	97,829.7
BP-KDH-210	885,360.5	97,839.0
BP-KDH-211	885,368.5	97,827.5
BP-KDH-212	885,377.1	97,781.2
BP-KDH-213	885,388.2	97,786.3
BP-KDH-214	885,367.8	97,931.3
BP-KDH-215	885,380.5	97,901.9
BP-KDH-216	885,385.7	97,909.5
BP-KDH-217	885,390.9	97,890.1
BP-KDH-218	885,402.0	97,881.7
BP-KDH-219	885,408.9	97,891.9
BP-KDH-219A	885,407.6	97,890.2
BP-KDH-221	885,349.9	97,952.4
BP-KDH-222	885,357.1	97,959.4

NOTES:
1. As-drilled test boring locations are shown on Sheet 2, Site and Subsurface Exploration Location Plan.
2. As-drilled coordinates of test borings were determined by MaineDOT using optical survey equipment and reference NAD83, Maine 2000 West Zone coordinate system.

Prepared By:	MLS	9/3/2013
Checked By:	EFW	11/5/2013
Reviewed By:	WAC	11/22/2013

TABLE II
SUMMARY OF SUBSURFACE EXPLORATIONS - SUBSURFACE DATA
U.S. ROUTE 1 BYPASS BRIDGE OVER ROUTE 236
MAINEDOT WIN 19283.00
KITTERY, MAINE

Test Boring No.[1]	Ground Surface Elevation[2,3]	Type of Exploration	Approximate Strata Thickness[4,5,6,7,8] (ft)						Depth to Refusal (ft)	Approximate Elevation of Top of Bedrock/ Refusal[2,3]	Approximate Elevation of Bottom of Exploration[2,3]
			Bituminous/ Portland Cement Concrete	Fill	Marine Deposit (Silt/Clay)	Glacial Till	Blasted Rock	Weathered Bedrock			
BB-KDH-101	45.3	Test Boring	1.6	10.2	NE	NE	NE	NE	11.8	33.5	19.3
BB-KDH-101A	45.3	Test Boring	1.6	>1.4 (BOE)	--	--	--	--	--	--	42.3
BB-KDH-102	44.8	Test Boring	1.0	8.6	NE	NE	NE	NE	9.6	35.2	35.2
BB-KDH-103	27.6	Test Boring	0.5	3.0	NE	NE	1.5	NE	5.0	22.6	17.6
BB-KDH-104	27.7	Test Boring	0.5	1.9	NE	NE	NE	0.3	2.7	25.0	25.0
BB-KDH-105	46.5	Test Boring	1.4	11.6	2.0	2.0	NE	2.0	19.0	27.5	27.5
BB-KDH-106	45.7	Test Boring	1.2	16.3	NE	NE	NE	1.4	18.9	26.8	14.0
BB-KDH-201	37.2	Test Boring	NE	5.5	4.5	2.2	NE	0.2	12.4	24.8	20.1
BB-KDH-202	45.6	Test Boring	1.6	7.4	4.1	NE	NE	1.4	14.5	31.1	26.1
BB-KDH-203	46.2	Test Boring	2.5	11.5	2.5	NE	NE	0.5	17.0	29.2	24.2
BB-KDH-204	32.1	Test Boring	NE	4.0	2.4	NE	NE	NE	6.4	25.7	21.2
BP-KDH-205	36.7	Auger Probe	NE	ND	ND	ND	ND	ND	12.2	24.5	24.5
BP-KDH-206	45.3	Auger Probe	1.9	ND	ND	ND	ND	ND	14.2	31.1	31.1
BP-KDH-207	45.4	Auger Probe	2.2	ND	ND	ND	ND	ND	19.3	26.1	26.1
BP-KDH-208	45.7	Auger Probe	2.0	ND	ND	ND	ND	0.6	19.3	26.4	26.4
BP-KDH-209	45.4	Auger Probe	1.5	ND	ND	ND	ND	1.7	15.2	30.2	30.2
BP-KDH-210	45.6	Auger Probe	2.0	ND	ND	ND	ND	ND	19.8	25.8	25.8
BP-KDH-211	45.4	Auger Probe	2.6	ND	ND	ND	ND	ND	18.2	27.2	27.2
BP-KDH-212	37.4	Hand Probe	NE	1.3	NE	NE	NE	NE	1.3	36.1	36.1
BP-KDH-213	36.8	Hand Probe	NE	0.8	NE	NE	NE	NE	0.8	36.0	36.0
BP-KDH-214	45.9	Auger Probe	1.5	ND	ND	ND	ND	ND	13.7	32.2	32.2
BP-KDH-215	46.3	Auger Probe	2.0	ND	ND	ND	ND	ND	16.8	29.5	29.5
BP-KDH-216	46.3	Auger Probe	1.5	ND	ND	ND	ND	0.3	16.5	29.8	29.8
BP-KDH-217	46.2	Auger Probe	2.0	ND	ND	ND	ND	ND	19.5	26.7	26.7
BP-KDH-218	46.1	Auger Probe	2.6	ND	ND	ND	ND	ND	17.6	28.5	28.5
BP-KDH-219	46.1	Auger Probe	1.5	ND	ND	ND	ND	ND	17.8	28.3	28.3
BP-KDH-219A	46.1	Auger Probe	4.1 (BOE)	--	--	--	--	--	--	--	42.0
BP-KDH-221	38.2	Hand Probe	NE	0.7	NE	NE	NE	NE	0.7	37.5	37.5
BP-KDH-222	39.4	Hand Probe	NE	1.2	NE	NE	NE	NE	1.2	38.2	38.2

NOTES:
1. As-drilled test boring locations are shown on Sheet 2, Site and Subsurface Exploration Location Plan.
2. Ground surface elevations at test boring locations were determined in the field by MaineDOT using optical survey equipment
3. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
4. "NE" indicates stratum was not encountered in test boring or probe.
5. ">" indicates stratum was not fully penetrated in test boring or probe.
6. BOE indicates bottom of exploration, obstruction encountered.
7. ND indicates stratum thickness not determined, no samples were collected from the probes.
8. Strata thickness reported at probe locations was estimated based on drilling behavior.

Prepared By:	MLS	9/5/2013
Checked By:	EFW	11/5/2013
Reviewed By:	WAC	11/22/2013

APPENDIX A

Test Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-101
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-4-12 / 1-4-12	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885367, N97821 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		-BITUMINOUS CONCRETE-	
								44.3	1.0	
								43.7	-PORTLAND CEMENT CONCRETE- 1.6	
	1D	24/10	2.0 - 4.0	3/9/12/15	21	21			Brown, dry, medium dense, fine to medium SAND, some gravel, little coarse sand, trace silt -FILL-(SP)	
5	2D	24/10	5.0 - 7.0	3/5/4/5	9	9	6		Brown, moist, loose, fine to coarse SAND, some gravel, little silt -FILL-(SW)	G#230402 A-1-b, SW
							8			
							8			
							20			
							19			
10	3D	22/6	10.0 - 11.8	7/6/4/30(4")	10	10	11		Brown, wet, loose, fine to medium SAND, little coarse sand, silt and gravel, greater silt content from 11.6 to 11.8 ft -FILL-(SW)	
							NQ Core			
								33.5	11.8	
	R1	36/36	12.0 - 15.0	RQD = 61%					Top of Bedrock at El. 33.5 Gray, aphanitic SLATE. Hard, fresh to slightly weathered, occasional moderately weathered joints. Joints dipping at low to steep angles, very close to close, tight to open, planar to stepped, smooth to rough, highly fractured zone from approximately 14.8 to 15.0 ft. Rock Mass Quality=Fair Recovery=100% -KITTERY FORMATION- R1 Core Times (min:sec): 12.0-13.0' (4:00), 13.0-14.0' (2:00), 14.0-15.0' (2:00)	UC qp=41,002 psi
15	R2	24/24	15.0 - 17.0	RQD = 79%					Gray, aphanitic SLATE. Hard, fresh to slightly weathered, occasional moderately weathered joints. Primary joints low angle with steep secondary joints. One vertical joint at approximately 16.5 ft. Oxidation on some joint surfaces, very close to close, tight to open, planar to stepped, smooth to rough. Rock Mass Quality=Good Recovery=100% -KITTERY FORMATION- R2 Core Times (min:sec): 15.0-16.0' (3:00), 16.0-17.0' (2:00)	UC qp=30,720 psi
	R3	35/35	17.0 - 19.9	RQD = 23%						
	R4	25/25	19.9 - 22.0	RQD = 32%						
20										
	R5	48/46	22.0 - 26.0	RQD = 65%					Gray, aphanitic SLATE. Hard, fresh to slightly weathered. Joints dipping at low and vertical angles, very close to close (highly fractured), planar to stepped, smooth to rough, tight to open. Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION- R3 Core Times (min:sec): 17.0-18.0' (3:00), 18.0-19.0' (2:00), 19.0-19.9' (2:00)	
25										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. UC=unconfined compressive strength; modulus; psi=pounds per square in.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-101
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-4-12 / 1-4-12	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885367, N97821 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.2
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ Core	19.3		Gray, aphanitic SLATE. Hard, fresh to slightly weathered. Primary joints dipping at low to moderate angles, very close to close, planar to stepped, smooth to rough, tight to open. Secondary vertical joints. Oxidized joint surfaces. Rock Mass Quality=Poor Recovery=100% -KITTERY FORMATION- R4 Core Times (min:sec): 19.9-21.0' (2:00), 21.0-21.0' (3:00) Gray, aphanitic SLATE. Moderately hard, fresh to slightly weathered. Joints dipping at low and steep angles, planar to stepped, smooth to rough, tight to open. Oxidation on some joint surfaces, occasional calcite coatings on joint surfaces. Rock Mass Quality=Fair Recovery=96% -KITTERY FORMATION- R5 Core Times (min:sec): 22.0-23.0' (3:00), 23.0-24.0' (2:00), 24.0-25.0' (3:00), 25.0-26.0' (2:00) 26.0 **Bottom of Exploration at 26.0 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. UC=unconfined compressive strength; modulus; psi=pounds per square in.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-101A
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.3	**Auger ID/OD:**	2.5 in.
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-4-12 / 1-4-12	**Drilling Method:**	SSA Drive	**Core Barrel:**	--
Boring Location:	E885368, N97822 (See Plan)	**Casing ID/OD:**	--	**Water Level*:**	Dry
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			-BITUMINOUS CONCRETE-	
								44.3		1.0	
								43.7		-PORTLAND CEMENT CONCRETE-	
										1.6	
	1D	12/3	2.0 - 3.0	3/19/5(0.0)						Brown, dry, dense, fine to medium SAND, trace coarse sand, trace silt, little gravel (concrete pieces) -FILL-(SP)	
								42.3		3.0	
5											
10											
15											
20											
25											

Bottom of Exploration at 3.0 feet below ground surface.

Note: Resistance at 3.0 ft (probable concrete), moved boring location approximately 2.0 ft south. See BB-KDH-101 test boring log.

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-102
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	44.8	**Auger ID/OD:**	2.5 ID
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-2.375/1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	S-140/30
Date Start/Finish:	1-3-12 / 1-3-12	**Drilling Method:**	HSA Drive	**Core Barrel:**	--
Boring Location:	E885306, N97818 (See Plan)	**Casing ID/OD:**	--	**Water Level*:**	Dry
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D*	24/18	0.5 - 2.5	31/28/21/19	49	49		44.3		-BITUMINOUS CONCRETE- 0.5	
								43.8		Black, dry, dense, gravel size asphalt pieces 1.0	
								43.5		Gray, crushed stone - 0.75 in. stone size 1.3	
	2D*	24/12	2.5 - 4.5	17/11/7/5	18	18				Brown, moist, dense, fine to coarse SAND, some silt and gravel, grading to little gravel at 3 to 3.5 ft -FILL-(SM)	G#230403 A-1-b, SM
	3D	24/5	4.5 - 6.5	6/15/15/7	30	30				Brown, moist, dense, fine to coarse SAND, some gravel, trace silt -FILL-(SW)	
5	4D	24/7	6.5 - 8.5	7/7/6/5	13	13				Brown, moist, fine to coarse SAND, some gravel, trace silt -FILL-(SW)	
	5D	10/11	8.5 - 9.3	9/50(4")						Brown, moist, fine to medium SAND, some silt, little coarse sand, some gravel grading to little gravel at 8.5 to 8.9 ft -FILL-(SP-SM)	
								35.2		Note: Auger refusal on probable bedrock at 9.6 ft. 9.6	
10										**Bottom of Exploration at 9.6 feet below ground surface.**	
15											
20											
25											

Remarks:

1. *Indicates sample obtained with 3-in. OD split spoon driven with 140 lb. hammer.
2. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
3. Hammer consisted of rope and cathead and safety hammer.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-103
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.6	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-2.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-5-12 / 1-5-12	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885321, N97894 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D*	18/14	0.5 - 2.0	26/20/14	34	34		27.1	-BITUMINOUS CONCRETE- 0.5	
								26.9	0.75 to 2 in. crushed stone 0.7	
	2D*	23/17	2.0 - 3.9	13/20/45/57	65	65		25.1	Brown, dry, dense, fine to coarse SAND, little gravel, little to trace silt -FILL-(SW)	
								24.1	Brown, dry, dense, fine to coarse SAND, some gravel, trace silt -FILL-(SW) 2.5	
5	R1	12/12	5.2 - 6.2	RQD = 0%			NQ Core	22.6	Brown, wet, dense, fine to coarse SAND, some silt, trace gravel -FILL-(SP) 3.5	
	R2	12/12	6.2 - 7.2	RQD = 0%					Blasted rock pieces, little fine to coarse sand, trace silt -BLAST ROCK- Note: Auger refusal at 5.0 ft. 5.0	
	R3	34/34	7.2 - 10.0	RQD = 24%					Top of Bedrock at El. 22.6 Gray, aphanitic SLATE, moderately hard, slightly weathered, highly fractured, low angle and steep to vertical joints, planar to undulating, smooth to rough, open. Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION-	
10								17.6	R1 Core Times (min:sec): 5.2-6.2' (2:00) Gray, aphanitic SLATE, moderately hard, slightly weathered, highly fractured, low angle and vertical joints, planar to undulating, smooth to rough, open. Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION- R2 Core Times (min:sec): 6.2-7.2' (4:00)	
15									Gray, aphanitic SLATE, moderately hard, slightly to moderately severely weathered, highly fractured zone from approximately 9.5 to 10.0 ft. Joints dipping at low to steep angles with secondary vertical joints, planar to undulating, smooth to rough, tight to open, oxidation on some joint surfaces. Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION- R3 Core Times (min:sec): 7.2-8.2' (3:00), 8.2-9.2' (3:00), 9.2-10.0' (3:00) 10.0	
20									**Bottom of Exploration at 10.0 feet below ground surface.**	
25										

Remarks:

1. *Indicates sample obtained with 3-in. OD split spoon driven with 140 lb. hammer.
2. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
3. Hammer consisted of rope and cathead and safety hammer.
4. No elevated PID readings were detected during sample screening operations.
5. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-104
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.7	**Auger ID/OD:**	2.5 in.
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-2.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	S-300/30
Date Start/Finish:	1-5-12 / 1-5-12	**Drilling Method:**	SSA Drive	**Core Barrel:**	--
Boring Location:	E885414, N97850 (See Plan)	**Casing ID/OD:**	--	**Water Level*:**	Dry
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D*	18/15	0.5 - 2.0	25/20/24	44	44		27.2		-BITUMINOUS CONCRETE- 0.5	
								27.0		0.75 to 1 in. crushed stone 0.7	
	*2D	5/5	2.0 - 2.4	24(5")/6(0")				25.3		Brown, dry, dense, fine to medium SAND, little coarse sand, some gravel, trace silt -FILL-(SW)	
								25.0		Brown, dry, dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW) 2.4	
5										Note: Split spoon refusal at 2.4 ft, auger refusal on probable bedrock at 2.7 ft. 2.7	
										Bottom of Exploration at 2.7 feet below ground surface.	
10											
15											
20											
25											

Remarks:

1. *Indicates sample obtained with 3-in. OD split spoon driven with 140 lb. hammer.
2. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
3. Hammer consisted of rope and cathead and safety hammer.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-105
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	46.5	**Auger ID/OD:**	2.5 in.
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375/2.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	S-140/30
Date Start/Finish:	1-4-12 / 1-4-12	**Drilling Method:**	HSA Drive	**Core Barrel:**	--
Boring Location:	E885444, N97949 (See Plan)	**Casing ID/OD:**	--	**Water Level*:**	Dry
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0								45.6	-BITUMINOUS CONCRETE- 0.9	
	1D*	18/10	1.5 - 3.0	20/23/21	44	44		45.1	-PORTLAND CEMENT CONCRETE- 1.4	
									Brown, dry, dense, fine to coarse SAND, little gravel (one 3 in. diameter piece), trace silt -FILL-(SW)	
	2D*	24/16	3.0 - 5.0	14/8/6/12	14	14			Brown, moist, medium dense, sandy GRAVEL, little silt -FILL-(GM)	G#23404 A-1-b, GM
5	3D	24/10	5.0 - 7.0	9/7/7/18	14	14			Brown, dry, medium dense, GRAVEL, some fine to coarse sand, trace silt -FILL-(GP)	
	4D	24/3	7.0 - 9.0	11/13/10/11	23	23			Brown, dry, medium dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW) Note: Missample, redrive spoon from 9 to 11 ft.	
	5D	24/12	9.0 - 11.0	7/7/9/4	16	16			Gray, dry, medium dense, GRAVEL pieces, poor recovery -FILL-(GP)	
10	6D	24/10	11.0 - 13.0	4/7/4/5	11	11			Brown, moist, medium dense, fine to medium SAND, little silt, little gravel, trace coarse sand -FILL-(SP-SM) Brown, moist, medium dense, fine to medium SAND, little silt, little gravel, trace coarse sand -FILL-(SP-SM)	
	7D	24/20	13.0 - 15.0	5/4/7/17	11	11		33.5	13.0 Gray-brown, mottled, moist, medium stiff, silty CLAY, few gray fine sand lenses -MARINE DEPOSIT-(CL)	
15	8D	24/18	15.0 - 17.0	11/11/11/23	22	22		31.5	15.0 Brown, moist, medium dense, fine to medium SAND, little coarse sand, little gravel, trace silt -GLACIAL TILL-(SP-SM)	
								29.5	17.0 Note: Drill action and spoon refusal indicate strata change at 17.0 ft. -PROBABLE WEATHERED BEDROCK- Note: Auger refusal on probable bedrock at 19.0 ft.	
20								27.5	19.0 **Bottom of Exploration at 19.0 feet below ground surface.**	
25										

Remarks:

1. *Indicates sample obtained with 3-in. OD split spoon driven with 140 lb. hammer.
2. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
3. Hammer consisted of rope and cathead and safety hammer.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-106
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.7	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-3-12 / 1-3-12	**Drilling Method:**	SSA to 5.0 ft; NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885362, N97923 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		-BITUMINOUS CONCRETE-	
								44.7	1.0	
	1D	24/15	1.0 - 3.0	25/22/15/12	37	37			Gray-brown, dry, dense, fine to medium sand, trace silt -FILL-(SP)	
								43.7	2.0	
								43.5	-BITUMINOUS CONCRETE-	
								43.2	2.2	
									Gray-brown, dry, dense, GRAVEL (0.75 in.), trace fine to coarse sand, trace silt -FILL-(GP)	
									2.5	
5	2D	24/6	5.0 - 7.0	6/6/5/4	11	11	11		Brown, dry, dense, fine to coarse SAND, trace silt -FILL-(SW)	
							12		Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt -FILL-(SM)	G#230405 A-1-b, SM
							13			
							13			
							13			
10	3D	24/5	10.0 - 12.0	5/5/4/7	9	9	5		Brown, wet, loose, fine to medium SAND, little coarse sand, little gravel, little silt -FILL-(SP)	
							10			
							15			
							21			
							23		Note: Pushed on cobble at 15.0 ft.	
15	4D	24/3	15.0 - 17.0	46/7/6/5	13	13	45		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW)	
							10		Note: Washed ahead of casing 15.0 to 19.0 ft.	
							10	28.2	17.5	
							39		Note: Drill action and rock chips in wash water indicate strata change at 17.5 ft.	
							NQ	26.8	-PROBABLE WEATHERED BEDROCK-	
	R1	24/24	19.0 - 21.0	RQD = 0%					18.9	
20									Top of Bedrock at El. 26.8 Gray, fine-grained to aphanitic, PHYLLITE. Hard, slightly to moderately weathered, highly fractured joints dipping at moderate to steep angles, secondary vertical joints, very close to close, planar to undulating, smooth to rough, tight to open, occasional calcite veins Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION- R1 Core Times (min:sec): 19.0-20.0' (4:00), 20.0-21.0' (5:00)	
	R2	41/34	21.0 - 24.4	RQD = 39%						UC qp=12,670 psi
25	R3	25/25	24.4 - 26.5	RQD = 32%					Gray, fine grained to aphanitic, PHYLLITE. Hard, slightly to moderately weathered, highly fractured, joints dipping at low to	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. UC=unconfined compressive strength; psi=pounds per square in.
4. No elevated PID readings were detected during sample screening operations.
5. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-106
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.7	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B53 Truck	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	1-3-12 / 1-3-12	**Drilling Method:**	SSA to 5.0 ft; NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885362, N97923 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ Core			moderate angles, secondary vertical joints, very close to close, planar to undulating, smooth to rough, tight to open, occasional calcite veins. Rock Mass Quality=Poor Recovery=83% -KITTERY FORMATION- R2 Core Times (min:sec): 21.0-22.0' (5:00), 22.0-23.0' (5:00), 23.0-24.0' (4:00)	
	R4	31/27	26.5 - 29.1	RQD = 36%						Gray, fine grained to aphanitic, PHYLLITE. Hard, slightly to moderately weathered, highly fractured, joints dipping at low to moderate angles, very close to close, planar to undulating, smooth to rough, tight to open, occasional calcite veins. Secondary vertical joints tight to open. Rock Mass Quality=Poor Recovery=100% -KITTERY FORMATION- R3 Core Times (min:sec): 24.4-25.4' (4:00), 25.4-26.5' (5:00)	UC qp=14,797 psi
	R5	31/28	29.1 - 31.7	RQD = 77%						Gray, fine grained to aphanitic, PHYLLITE. Hard, slightly to moderately weathered, highly fractured, joints dipping at low to moderate angles, very close to close, planar to undulating, smooth to rough, tight to open, occasional calcite veins, one near vertical secondary joint, discoloration on joint surfaces. Rock Mass Quality=Poor Recovery=87% -KITTERY FORMATION- R4 Core Times (min:sec): 26.5-27.5' (4:00), 27.5-28.5' (4:00), 28.5-29.1' (5:00)	
30								14.0		Gray, fine grained to aphanitic PHYLLITE. Hard, slightly to moderately weathered, highly fractured from approximately 29.1 to 29.5 ft. Joints dipping at low to moderate angles, moderately close, planar to undulating, smooth to rough, open, frequent calcite veins, discoloration on joint surfaces. Rock Mass Quality=Good Recovery=90% -KITTERY FORMATION- R5 Core Times (min:sec): 29.1-30.1' (5:00), 30.1-31.7 (5:00)	
35											
40											
45											
50											

31.7

Bottom of Exploration at 31.7 feet below ground surface.

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. UC=unconfined compressive strength; psi=pounds per square in.
4. No elevated PID readings were detected during sample screening operations.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-201
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	37.2	**Auger ID/OD:**	--
Operator:	T. Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	8-19-13/8-19-13	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885303, N97863 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	2/5/3/3	8	8	2	36.5	Dark brown, moist, medium stiff, SILT, little find sand -TOPSOIL-(OH)	
							5		0.7	
							6		Brown, moist, fine to medium silty SAND, trace coarse sand, trace gravel -FILL-(SM)	
							10			
							21	33.2	4.0	
5	2D	24/14	5.0 - 7.0	7/12/11/17	23	23	38	31.7	Brown-rust-brown-gray, wet, very stiff, mottled, silty CLAY, little fine sand, trace coarse sand and gravel -FILL-(CL) (appears reworked)	
							44		5.5	
							48		Brown-rust-brown-gray, wet, very stiff, mottled, silty CLAY with brown fine sand layers (3-in. thick) -MARINE DEPOSIT-(CL)	
							67			
							67			
10	3D	24/12	10.0 - 12.0	15/25/31/52	56	56	41	27.2	10.0	
							47		Brown-rust-brown-gray, wet, very dense, fine to medium SAND, little coarse sand, little silt, little gravel -GLACIAL TILL-(SW)	
	R1	56/56	12.4 - 17.1	RQD = 55%			NQ Core	25.0	12.2	
								24.8	-WEATHERED BEDROCK-	
									12.4	
15									Top of Bedrock at El. 24.8 Gray, aphanitic PHYLLITE. Hard, fresh to slightly weathered. Joints dipping at low and steep angles, very close to close, tight to open, planar to stepped, smooth to rough, iron oxide on some joint surfaces, occasional calcite veins. Rock Mass Quality=Fair Recovery=100% -KITTERY FORMATION- R1 Core Times (min:sec): 12.4-13.4' (1:46), 13.4-14.4' (1:44), 14.4-15.4' (1:54), 15.4-16.4' (2:13), 16.4-17.1' (1:51)	
								20.1		
20									17.1	
									Bottom of Exploration at 17.1 feet below ground surface.	
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. No elevated PID readings were detected during sample screening operations.
4. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-202
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	45.6	**Auger ID/OD:**	--
Operator:	T. Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	8-15-13/8-15-13	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885343, N97840 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	44.6	-BITUMINOUS CONCRETE- 1.0	
								44.0	-CONCRETE-(with rebar) 1.6	
									Note: Description from auger cuttings. Brown, dry, fine to coarse SAND, some gravel, trace silt -FILL-(SW)	
5	1D	24/12	5.0 - 7.0	3/3/12/30	15	15	7		Brown, moist, medium dense, fine to medium SAND, some gravel (blast rock), trace coarse sand, trace silt -FILL-(SW)	
							41			
							34			
							33			
							17	36.6	9.0	
10	2D	24/10	10.0 - 12.0	2/2/2/4	4	4	11		Dark brown, moist, soft, SILT, trace fine to medium sand, trace clay, trace organics -MARINE DEPOSIT-(ML)	
							8			
							15			
							30(1.0")	32.5	13.1	
	R1	60/60	14.5 - 19.5	RQD = 85%			NQ Core	31.1	Note: Advanced roller bit to 14.5 ft. Seat casing at 14.0 ft. -WEATHERED BEDROCK- Note: Weathered rock chips observed in wash water from 13.1 to 14.5 ft. 14.5	
15									Top of Bedrock at El. 31.1 Gray, aphanitic PHYLLITE. Hard, fresh to slightly weathered, oxidation observed on some joint surfaces. Joints dipping at low to steep angles, very close to moderately close, tight to open, planar to undulating, smooth to rough, frequent calcite veins, occasional healed joints.	
								26.1	Rock Mass Quality=Good Recovery=100% -KITTERY FORMATION- R1 Core Times (min:sec): 14.5-15.5' (2:00), 15.5-16.5' (1:49), 16.5-17.5' (2:33), 17.5-18.5' (1:45), 18.5-19.5' (1:45) 19.5	
20									**Bottom of Exploration at 19.5 feet below ground surface.**	
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. No elevated PID readings were detected during sample screening operations.
4. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-203
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	46.2	**Auger ID/OD:**	--
Operator:	T. Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	8-14-13/8-14-13	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885396, N97899 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	45.2	-BITUMINOUS CONCRETE- 1.0	
								44.7	-CONCRETE-(with rebar) 1.5	
									Note: Description from auger cuttings: Brown, dry, fine to medium SAND, some gravel, little coarse sand, trace silt	
								42.7	-FILL-(SW)	
								41.7	-CONCRETE- 3.5	
5	1D	24/12	5.0 - 7.0	3/2/3/3	5	5	5		4.5	
							6		Brown, moist, loose, fine to medium SAND, little coarse sand, little silt, little gravel -FILL-(SW)	
							20			
							36			
							20			
10	2D	24/4	10.0 - 12.0	16/7/7/8	14	14	33		Brown, moist, medium dense, medium to coarse SAND, some gravel, trace fine sand, trace silt -FILL-(SW)	
							23			
							14			
							14			
							29	32.2	14.0	
15	3D	24/20	15.0 - 17.0	10/9/19/50(5.0")	28	28	42		Brown-gray mottled, wet, very stiff, silty CLAY, little fine to coarse sand, little gravel (bedrock fragments) -MARINE DEPOSIT-(CL)	
							45	29.7	16.5	
	R1	60/60	17.0 - 22.0	RQD = 50%			NQ Core	29.2	-WEATHERED BEDROCK- 17.0	
									Top of Bedrock at El. 29.2 Note: Advanced roller bit to 17.0 ft. Seat casing at 17.0 ft. Gray, aphanitic PHYLLITE. Hard, fresh to slightly weathered. Joints dipping at low to steep angles, very close to close, tight to open, planar to undulating, smooth to rough. Rock Mass Quality=Poor Recovery=100%	
20								24.2	-KITTERY FORMATION- R1 Core Times (min:sec): 17.0-18.0' (3:45), 18.0-19.0' (2:26), 19.0-20.0' (3:04), 20.0-21.0' (3:45), 21.0-22.0' (2:48) 22.0	
25									**Bottom of Exploration at 22.0 feet below ground surface.**	
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. No elevated PID readings were detected during sample screening operations.
4. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: US Route 1 Bypass over Route 236 Bridge
Location: Kittery, Maine

Boring No.: BB-KDH-204
PIN: 19283.00

Driller:	Maine Test Borings	**Elevation (ft.)**	32.1	**Auger ID/OD:**	--
Operator:	T. Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile B-53 Bombardier	**Hammer Wt./Fall:**	NW-300/24; S-140/30
Date Start/Finish:	8-15-13/8-15-13	**Drilling Method:**	NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	E885441, N97865 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	2/5/7/9	12	12	PUSH	31.6	Dark brown, dry, stiff SILT, trace fine sand with roots -TOPSOIL-(OH)	
									0.5 -FILL-(SM)	
							20		Note: Silt, sand and gravel observed in wash water 0-4.0 ft. Clay observed in wash water 4.0-5.0 ft.	
							38	28.1	4.0	
5	2D	18/8	5.0 - 6.5	5/8/50	58	58	43		Brown, wet, hard, silty CLAY, trace fine and coarse sand (rock piece in spoon tip) -MARINE DEPOSIT-(CL)	
	R1	42/42	6.4 - 9.9	RQD = 14%			29(6.0")	25.7	6.4 Top of Bedrock at El. 25.7 Gray, aphanitic PHYLLITE. Hard to soft, moderately severely weathered, highly fractured, joints dipping at low and steep angles, very close to close, open, planar to stepped, smooth to rough. Rock Mass Quality=Very Poor Recovery=100% R1 Core Times (min:sec): 6.4-7.4' (1:32), 7.4-8.4' (1:35), 8.4-9.4' (1:42), 9.4-9.9' (1:58)	
							NQ Core			
	R2	12/12	9.9 - 10.9	RQD = 0%						
10								21.2	Gray, aphanitic PHYLLITE. Hard, moderately severely weathered, highly fractured, joints dipping at low and steep angles, very close, open, planar to stepped, smooth to rough. Rock Mass Quality=Very Poor Recovery=100% -KITTERY FORMATION- R2 Core Times (min:sec): 9.9-10.9' (1:47)	
15									10.9 **Bottom of Exploration at 10.9 feet below ground surface.**	
20										
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD 83(96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. No elevated PID readings were detected during sample screening operations.
4. Water was introduced during the drilling process, therefore actual groundwater conditions were not observed at the time of drilling.

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Row 1 (Top): BB-KDH-101, R1: 12.0-15.0 ft below ground surface (BGS)/ BB-KDH-101, R2 (Top): 15.0-17.0 ft BGS
Row 2: BB-KDH-101, R2 (Bottom) 15.0-17.0 ft BGS/ BB-KDH-101, R3: 17.0-19.9 ft BGS
Row 3: BB-KDH-101, R4: 19.9-22.0 ft BGS
Row 4: BB-KDH-101, R5: 22.0-26.0 ft BGS

Row 1: BB-KDH-103, R1: 5.2-6.2 ft BGS, BB-KDH-103, R2: 6.2-7.2 ft BGS, BB-KDH-103, R3: 7.2-10.0 ft BGS

Row 1: BB-KDH-106, R1: 19.0-21.0 ft BGS, BB-KDH-106, R2: 21.0-24.4 ft BGS
Row 2: BB-KDH-106, R3: 24.4-26.5 ft BGS, BB-KDH-106, R4:26.5-29.1 ft BGS
Row 3: BB-KDH-106, R5: 29.1-31.7 ft BGS

Row 1: BB-KDH-203, R1: 17.0-22.0 ft BGS

Row 2: BB-KDH-202, R1: 14.5-19.5 ft BGS

Row 3: BB-KDH-204, R1: 6.4-9.9 ft BGS, BB-KDH-204, R2: 9.9-10.9 ft BGS

Row 4: BB-KDH-201, R1: 12.4-17.1 ft BGS

APPENDIX B

Laboratory Test Results

GeoTesting EXPRESS

125 Nagog Park
Acton, MA 01720
978 635 0424 Tel
978 635 0266 Fax

Transmittal

TO:
Erin Wood
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 2/23/2012	GTX NO: 11547
RE: U.S. Route 1 Bypass over Route 236 Bridge	

COPIES	DATE	DESCRIPTION
	2/23/2012	**February 2012 Laboratory Test Report**

REMARKS:

SIGNED:
Joe Tomei, Laboratory Manager

CC:

APPROVED BY:
Nancy Hubbard, Project Manager

GeoTesting
EXPRESS

Boston
Atlanta
New York

www.geotesting.com

February 23, 2012

Erin Wood
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

RE: U.S. Route 1 Bypass over Route 236 Bridge, Kittery, ME (GTX-11547)

Dear Erin:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received four samples from you on 2/15/2012. These samples were labeled as follows:

Boring	Sample	Depth
BB-KDH-101	S2	5-7 ft
BB-KDH-102	S2	2.5-4.5 ft
BB-KDH-105	S2	3.5-5.0 ft
BB-KDH-106	S2	5-7 ft

GTX performed the following test on each of these samples:

ASTM D 422 – Grain Size Analysis (sieve only)

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

125 Nagog Park
Acton, MA 01720
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

2/23/2012

GTX-11547
U.S. Route 1 Bypass over Route 236 Bridge Project

Kittery, ME

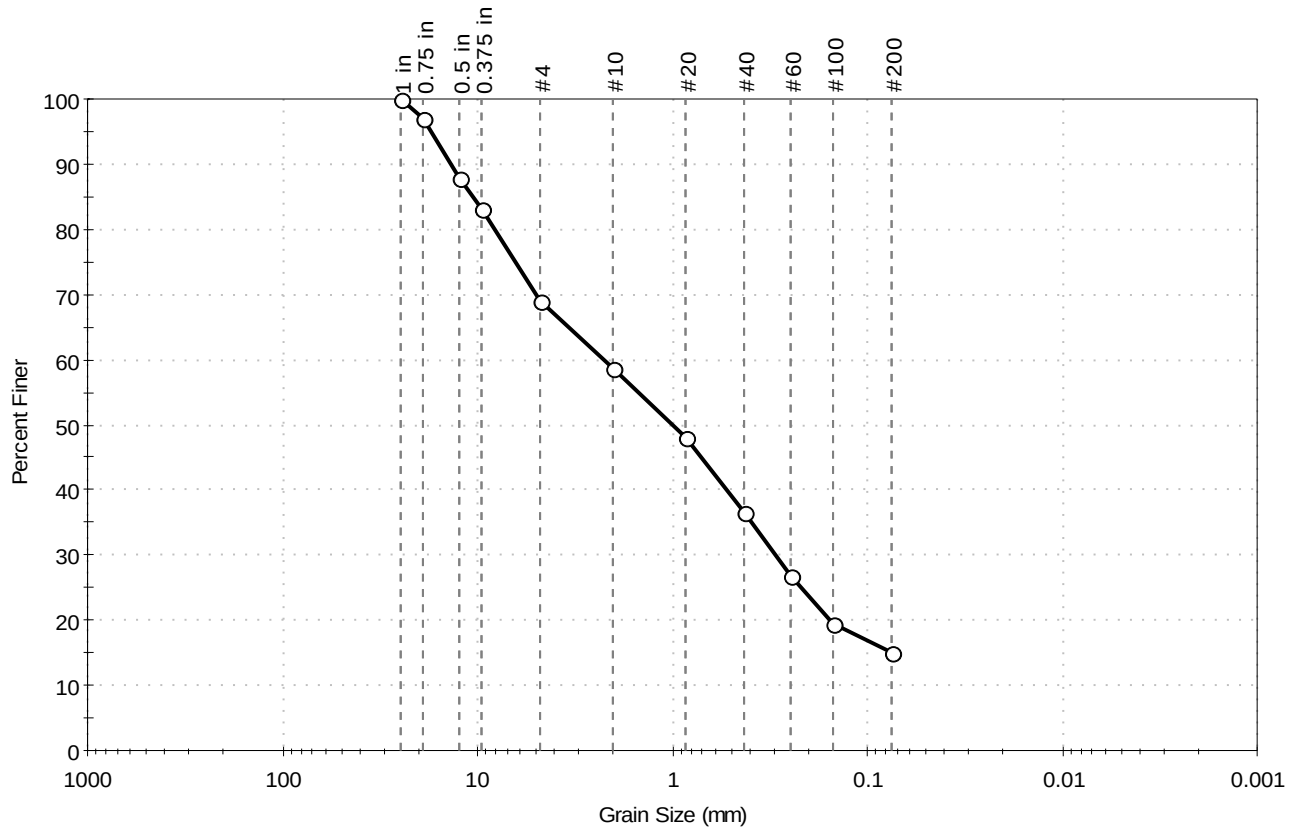
Client Project No.: 38239-000

Prepared for:

Haley & Aldrich, Inc.

Client:	Haley & Aldrich, Inc.		
Project:	U.S. Route 1 Bypass over Route 236 Bridge		
Location:	Kittery, ME	Project No:	GTX-11547
Boring ID: BB-KDH-101	Sample Type: bag	Tested By:	jbr
Sample ID:S2	Test Date: 02/20/12	Checked By:	jdt
Depth : 5-7 ft	Test Id: 230402		
Test Comment:	---		
Sample Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	31.0	53.9	15.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	88		
0.375 in	9.50	83		
#4	4.75	69		
#10	2.00	59		
#20	0.85	48		
#40	0.42	36		
#60	0.25	27		
#100	0.15	20		
#200	0.075	15		

Coefficients

D_{85} = 10.5974 mm	D_{30} = 0.2968 mm
D_{60} = 2.2318 mm	D_{15} = N/A
D_{50} = 0.9949 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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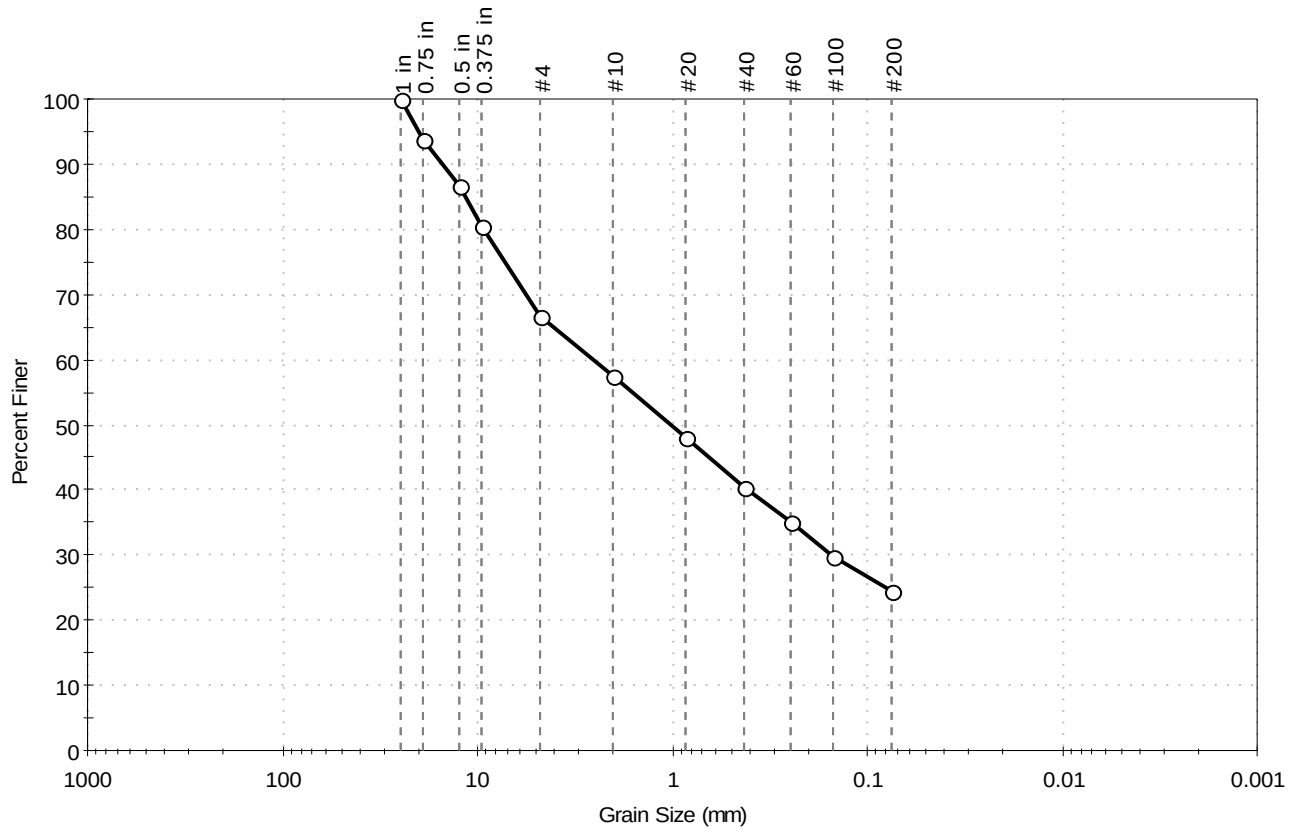
Client:	Haley & Aldrich, Inc.		
Project:	U.S. Route 1 Bypass over Route 236 Bridge		
Location:	Kittery, ME	Project No:	GTX-11547
Boring ID: BB-KDH-102	Sample Type: bag	Tested By:	jbr
Sample ID:S2	Test Date: 02/17/12	Checked By:	jdt
Depth : 2.5-4.5 ft	Test Id: 230403		
Test Comment:	---		
Sample Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	33.3	42.1	24.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	87		
0.375 in	9.50	81		
#4	4.75	67		
#10	2.00	57		
#20	0.85	48		
#40	0.42	40		
#60	0.25	35		
#100	0.15	30		
#200	0.075	25		

Coefficients

D_{85} = 11.5746 mm	D_{30} = 0.1538 mm
D_{60} = 2.5480 mm	D_{15} = N/A
D_{50} = 1.0100 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

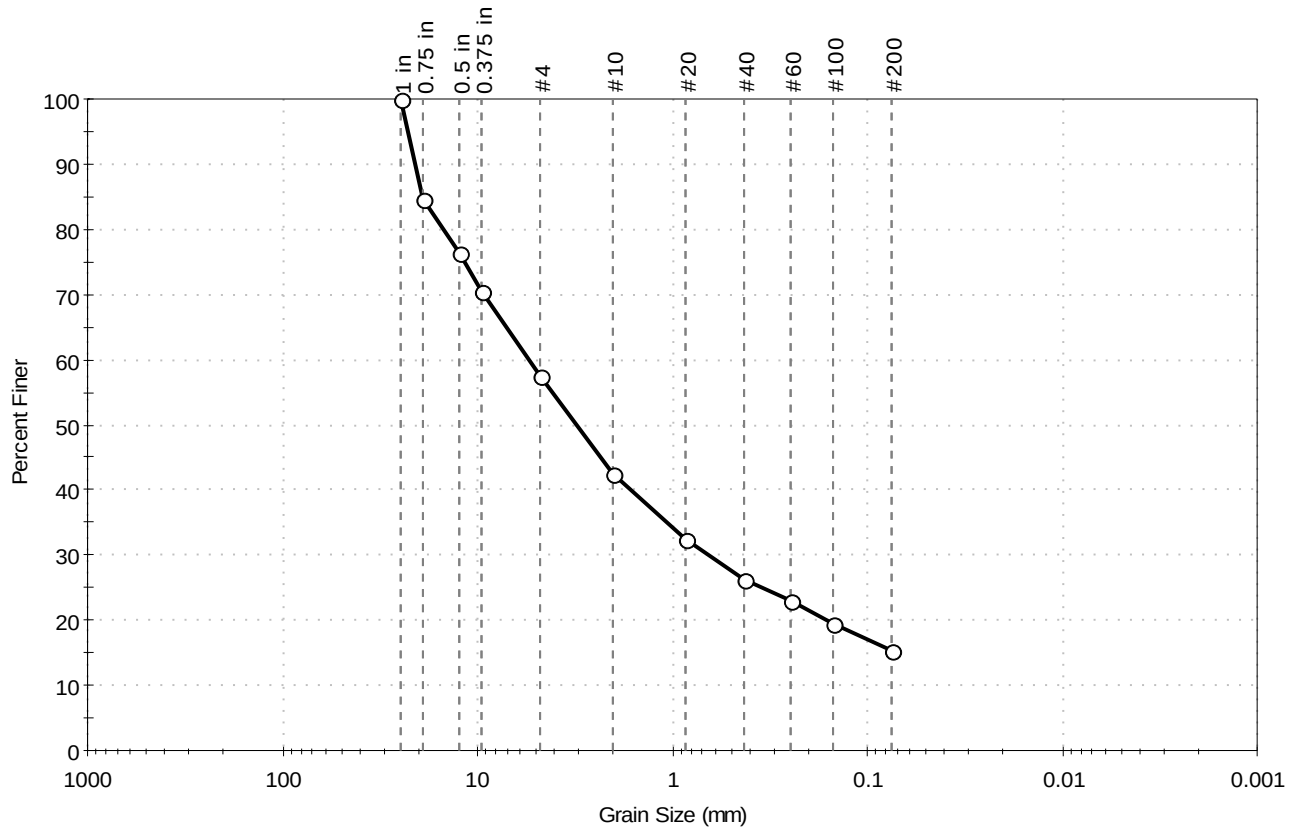
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

printed 2/23/2012 8:10:58 AM

Client:	Haley & Aldrich, Inc.		
Project:	U.S. Route 1 Bypass over Route 236 Bridge		
Location:	Kittery, ME	Project No:	GTX-11547
Boring ID: BB-KDH-105	Sample Type: bag	Tested By:	jbr
Sample ID:S2	Test Date: 02/17/12	Checked By:	jdt
Depth : 3.5-5.0 ft	Test Id: 230404		
Test Comment:	---		
Sample Description:	Moist, brown silty gravel with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	42.5	42.0	15.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	77		
0.375 in	9.50	70		
#4	4.75	57		
#10	2.00	42		
#20	0.85	32		
#40	0.42	26		
#60	0.25	23		
#100	0.15	19		
#200	0.075	15		

Coefficients

D_{85} = 19.0760 mm	D_{30} = 0.6474 mm
D_{60} = 5.4351 mm	D_{15} = N/A
D_{50} = 3.0941 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

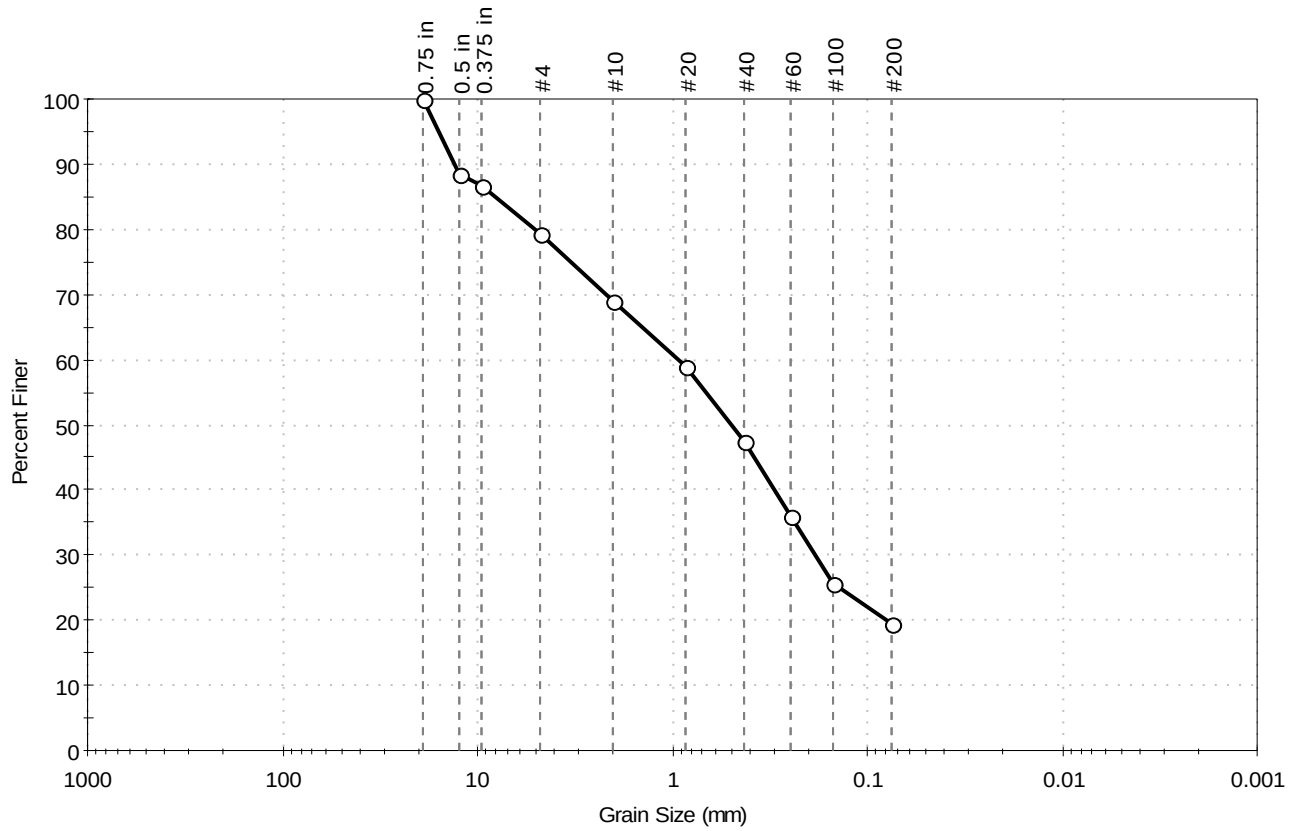
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

printed 2/23/2012 8:11:32 AM

Client:	Haley & Aldrich, Inc.		
Project:	U.S. Route 1 Bypass over Route 236 Bridge		
Location:	Kittery, ME	Project No:	GTX-11547
Boring ID: BB-KDH-106	Sample Type: bag	Tested By:	jbr
Sample ID:S2	Test Date: 02/20/12	Checked By:	jdt
Depth : 5-7 ft	Test Id: 230405		
Test Comment:	---		
Sample Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	20.8	59.7	19.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	89		
0.375 in	9.50	87		
#4	4.75	79		
#10	2.00	69		
#20	0.85	59		
#40	0.42	48		
#60	0.25	36		
#100	0.15	26		
#200	0.075	20		

Coefficients

D_{85} = 8.0402 mm D_{30} = 0.1867 mm

D_{60} = 0.9331 mm D_{15} = N/A

D_{50} = 0.4930 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

printed 2/23/2012 8:12:06 AM

GeoTesting EXPRESS

SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Avenue, Suite 203	
City, State, Zip :Portland, ME 04105	
Contact: Erin Wood	E-mail: EWood@HaleyAldrich.com
Phone: 207.482.4626	Fax: 207482.4676

PROJECT	
Project Name:U.S. Route 1 Bypass over Route 236 Bridge	
Address:	Client Project #:38239-000
City, State, Zip: Kittery, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company: Haley & Aldrich, Inc.	
Address: 70 Blanchard Rd., Ste 430	
City, State, Zip: Burlington, MA 01803-5100	
Contact: Accounts Payable	E-mail: AP@HaleyAldrich.com
Phone:617.886.7400	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #:
Shipped By: Erin Wood	Date Shipped: 2/14/12
Mode of Shipment: Fedex	Requested Turnaround: standard
Send Results To:	☒ CLIENT OFFICE ☐ PROJECT OFFICE
Send Results Via:	☒ E-MAIL ☐ FAX ☐ VERBAL ☐ HARD COPY

SOIL

*please include boring # and depth if known

	Sample ID	Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other:	Other:
1	BB-KDH-101, S2, 5 – 7 ft								x										x	
2	BB-KDH-102, S2, 2.5 – 4.5 ft (2 bags)								x										x	
3	BB-KDH-105, S2, 3.5 – 5 ft (2 bags)								x										x	
4	BB-KDH-106, S2, 5 – 7'								x										x	
5																				
6																				
7																				

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.): I have two bags of soil for several of the samples. Mix the bags together and test as one sample.

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: _____ PRINT NAME: _____ DATE: _____

Incoming Sample Inspection Performed ☑
Adverse conditions: NONE

Relinquished By:	DATE:	Received By:	DATE: 2/15/12
	TIME:		TIME: 3:30PM
Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:

GeoTesting EXPRESS

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

GeoTesting EXPRESS

125 Nagog Park
Acton, MA 01720
978 635 0424 Tel
978 635 0266 Fax

Transmittal

TO:

Erin Wood
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 4/4/2012	GTX NO: 11547
RE: U.S. Route 1 Bypass over Route 236 Bridge	

COPIES	DATE	DESCRIPTION
	4/4/2012	**March 2012 Laboratory Test Report**

REMARKS:

SIGNED: Mark Dobday, Laboratory Manager

CC:

APPROVED BY: Joe Tomei, Laboratory Manager

GeoTesting
EXPRESS

Boston
Atlanta
New York

www.geotesting.com

April 4, 2012

Erin Wood
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

RE: U.S. Route 1 Bypass over Route 236 Bridge, Kittery, ME (GTX-11547)

Dear Erin Wood:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received four samples from you on 2/15/2012. These samples were labeled as follows:

Boring Number	Sample Number	Depth
BB-KDH-101	R1	12.7-13.3 ft
BB-KDH-101	R2	15.5-16.2 ft
BB-KDH-106	R2	22.4-23.1 ft
BB-KDH-106	R5	30.1-31.3 ft

GTX performed the following tests on these samples:

4 ASTM D 7012D - Elastic Moduli of Rock in Uniaxial Compression

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Mark Dobday, P.G.
Laboratory Manager

GeoTesting
EXPRESS

125 Nagog Park
Acton, MA 01720
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

4/4/2012

GTX-11547
U.S. Route 1 Bypass over Route 236 Bridge

Kittery, ME

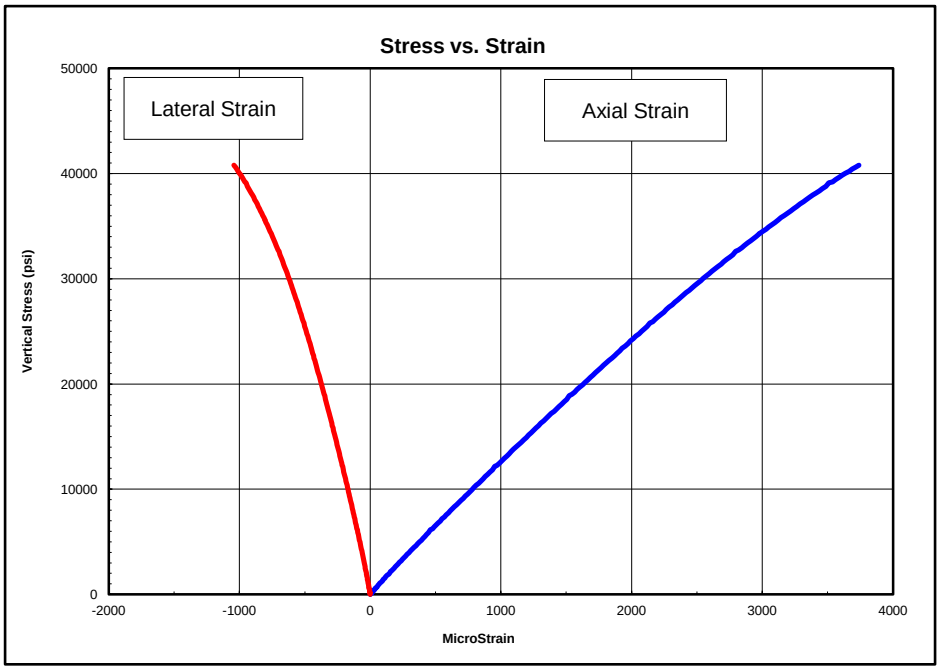
Client Project No.: 38239-000

Prepared for:

Haley & Aldrich, Inc.

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-101
Sample ID:	R1
Depth, ft:	12.77-13.14
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 41,002 psi

The graph above does not include all data up to the peak stress value. The strain gauges failed before the peak stress value was recorded.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-10000	12,400,000	0.22
10000-20000	11,800,000	0.24
20000-30000	10,900,000	0.27
30000-40000	9,200,000	0.35

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	2/22/2012
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge	Tested By:	daa
Project Location:	Kittery, ME	Checked By:	mpd
GTX #:	11547		
Boring ID:	BB-KDH-101		
Sample ID:	R1		
Depth:	12.77-13.14 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.38	4.38	4.38
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	595.19		
Bulk Density, lb/ft^3	171		
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00070	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060	0.00070
Diameter 2, in (rotated 90°)	-0.00030	-0.00040	-0.00030	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00020	0.00020

Difference between max and min readings, in:

0° = 0.00150 90° = 0.00060

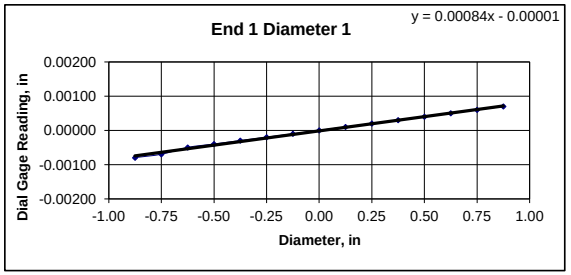
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00080	-0.00070	-0.00060	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00010	0.00020	0.00020	0.00020

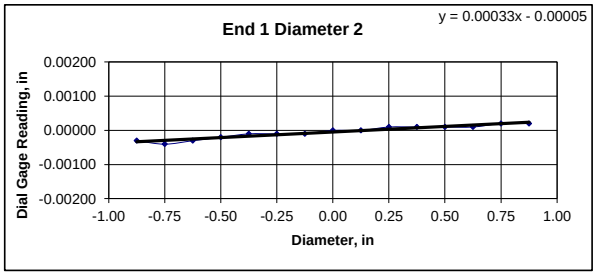
Difference between max and min readings, in:

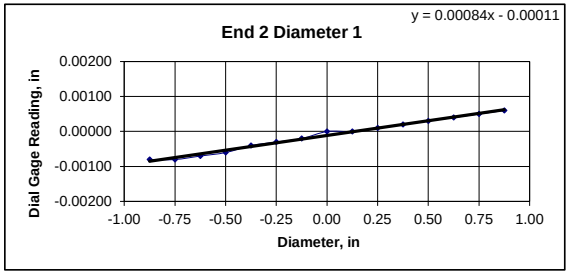
0° = 0.0014 90° = 0.0005

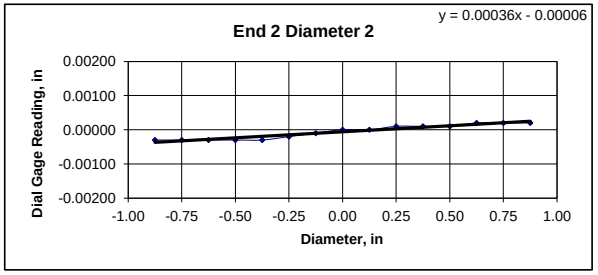
Maximum difference must be < 0.0020 in. Difference = ± 0.00075

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00084
Angle of Best Fit Line:	0.04813

End 2:

Slope of Best Fit Line	0.00084
Angle of Best Fit Line:	0.04813

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? **YES**

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00033
Angle of Best Fit Line:	0.01891

End 2:

Slope of Best Fit Line	0.00036
Angle of Best Fit Line:	0.02063

Maximum Angular Difference: 0.00172

Parallelism Tolerance Met? **YES**

Spherically Seated

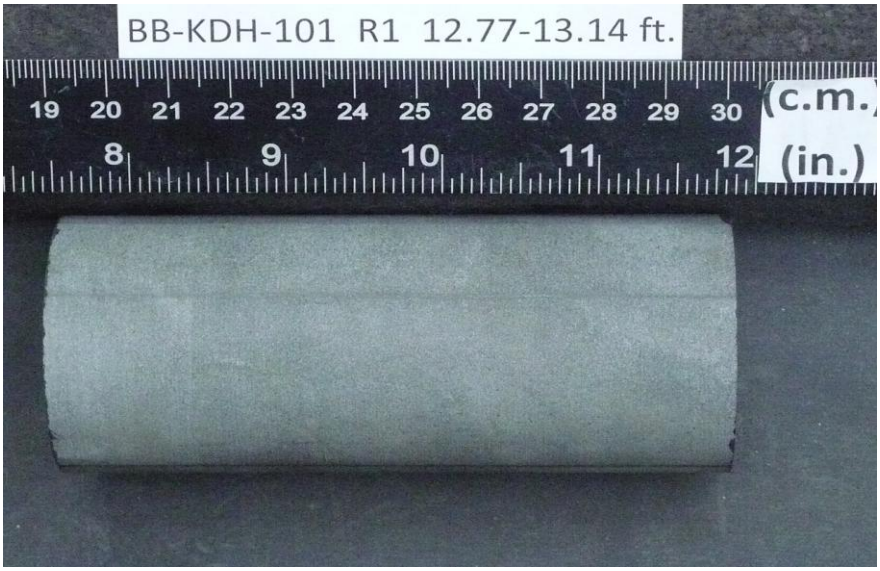
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00150	1.960	0.00077	0.044	YES
Diameter 2, in (rotated 90°)	0.00060	1.960	0.00031	0.018	YES
END 2					
Diameter 1, in	0.00140	1.960	0.00071	0.041	YES
Diameter 2, in (rotated 90°)	0.00050	1.960	0.00026	0.015	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-101
Sample ID:	R1
Depth, ft:	12.77-13.14

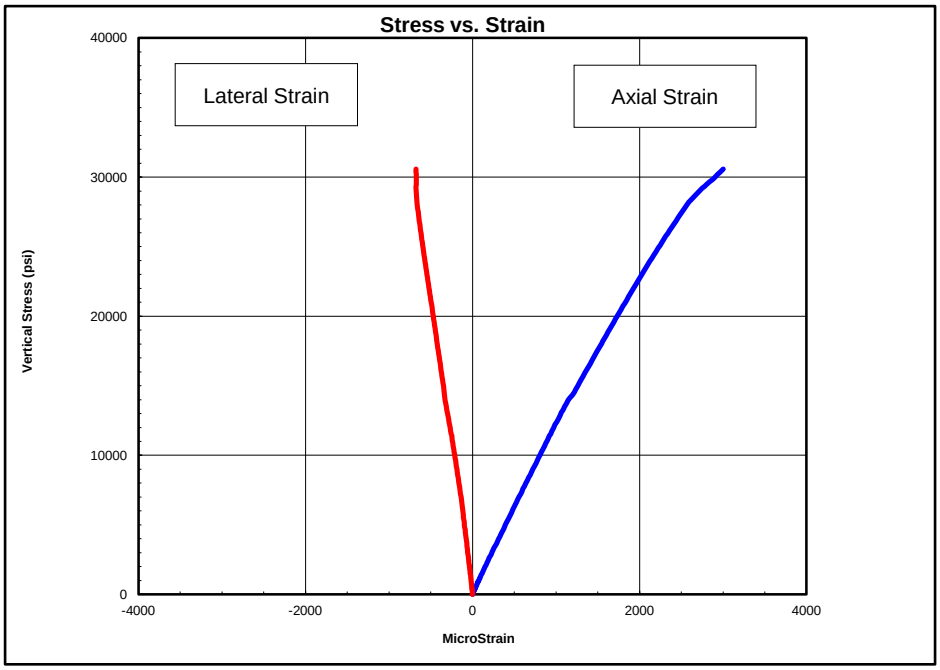


After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-101
Sample ID:	R2
Depth, ft:	15.65-16.02
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 30,720 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-10000	12,200,000	0.26
10000-20000	10,600,000	0.27
20000-30000	8,900,000	0.20

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	2/22/2012
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge	Tested By:	daa
Project Location:	Kittery, ME	Checked By:	mpd
GTX #:	11547		
Boring ID:	BB-KDH-101		
Sample ID:	R2		
Depth:	15.65-16.02 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.35	4.36	4.36
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	588.01		
Bulk Density, lb/ft^3	170		
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00090	-0.00080	-0.00070	-0.00060	-0.00040	-0.00030	-0.00010	0.00000	0.00000	0.00020	0.00030	0.00040	0.00050	0.00060	0.00070
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030	0.00030

Difference between max and min readings, in:

0° = 0.00160 90° = 0.00080

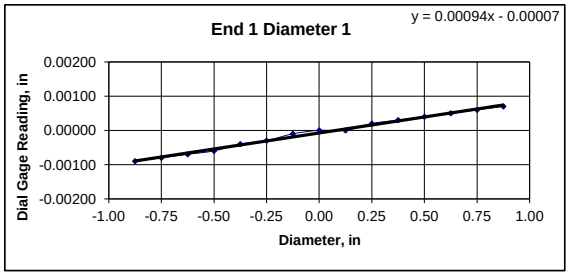
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00070	-0.00060	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00050	0.00050	0.00060	0.00080
Diameter 2, in (rotated 90°)	-0.00050	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020

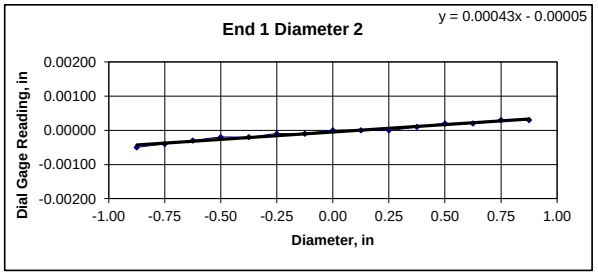
Difference between max and min readings, in:

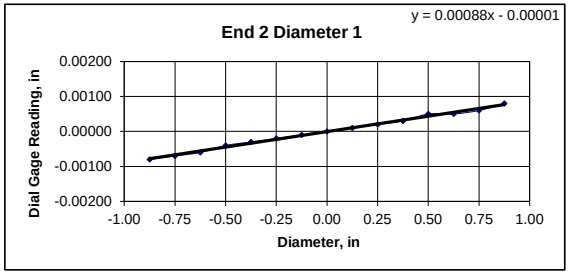
0° = 0.0016 90° = 0.0007

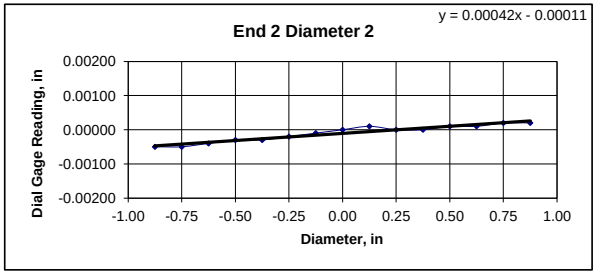
Maximum difference must be < 0.0020 in. Difference = ± 0.00080

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00094
Angle of Best Fit Line:	0.05386

End 2:

Slope of Best Fit Line	0.00088
Angle of Best Fit Line:	0.05042

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? **YES**

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00043
Angle of Best Fit Line:	0.02464

End 2:

Slope of Best Fit Line	0.00042
Angle of Best Fit Line:	0.02406

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? **YES**

Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00160	1.960	0.00082	0.047	YES
Diameter 2, in (rotated 90°)	0.00080	1.960	0.00041	0.023	YES
END 2					
Diameter 1, in	0.00160	1.960	0.00082	0.047	YES
Diameter 2, in (rotated 90°)	0.00070	1.960	0.00036	0.020	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

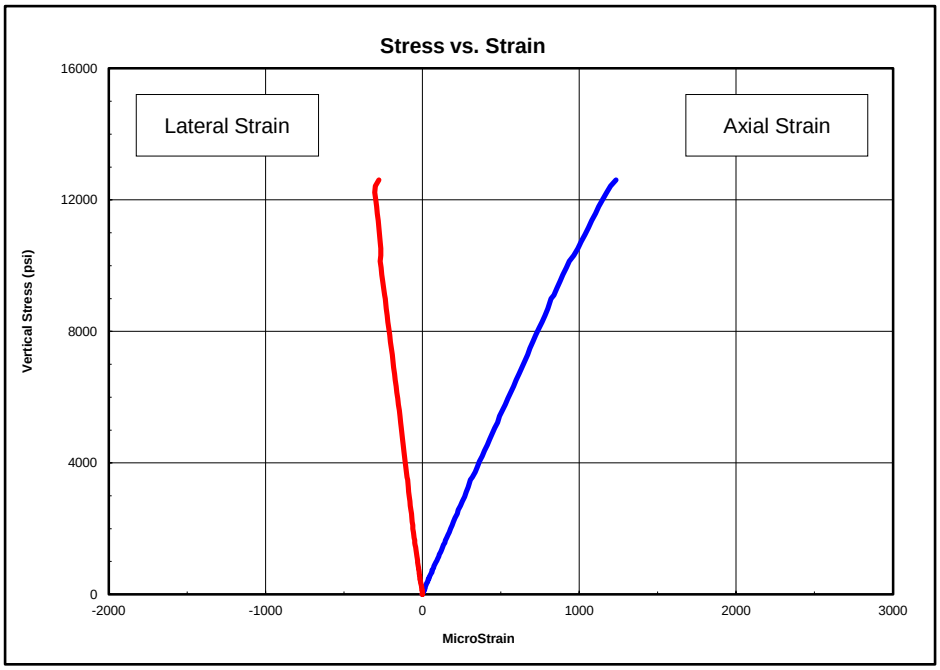
Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-101
Sample ID:	R2
Depth, ft:	15.65-16.02

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-106
Sample ID:	R2
Depth, ft:	22.54-22.91
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 12,670 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-4000	11,200,000	0.28
4000-8000	10,700,000	0.28
8000-12000	9,540,000	0.19

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	2/22/2012
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge	Tested By:	daa
Project Location:	Kittery, ME	Checked By:	mpd
GTX #:	11547		
Boring ID:	BB-KDH-106		
Sample ID:	R2		
Depth:	22.54-22.91 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.35	4.36	4.36
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	581.51		
Bulk Density, lb/ft^3	168		
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00070	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00020	0.00030	0.00040	0.00060	0.00070	0.00080
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010

Difference between max and min readings, in:

0° = 0.00160 90° = 0.00050

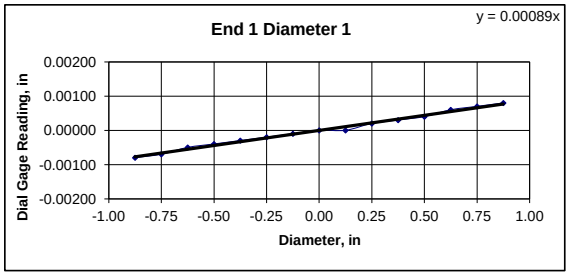
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00090	-0.00090	-0.00080	-0.00060	-0.00050	-0.00040	-0.00020	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010

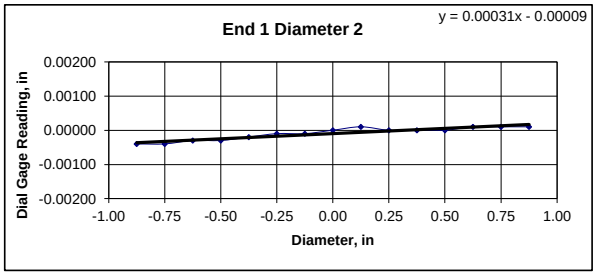
Difference between max and min readings, in:

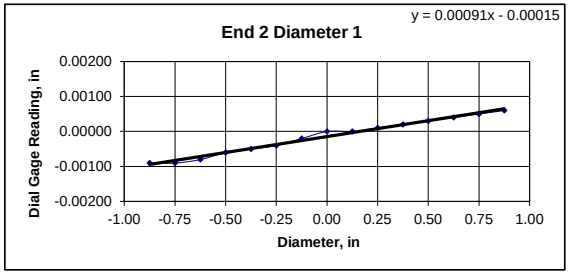
0° = 0.0015 90° = 0.0005

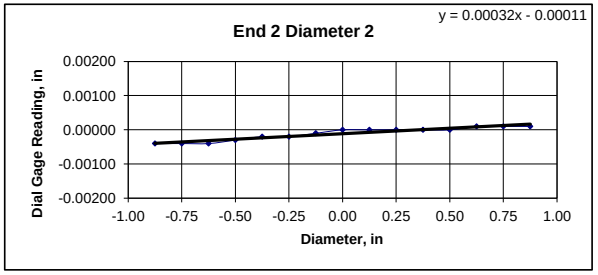
Maximum difference must be < 0.0020 in. Difference = ± 0.00080

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00089
Angle of Best Fit Line:	0.05099

End 2:

Slope of Best Fit Line	0.00091
Angle of Best Fit Line:	0.05214

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? **YES**

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00031
Angle of Best Fit Line:	0.01776

End 2:

Slope of Best Fit Line	0.00032
Angle of Best Fit Line:	0.01833

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? **YES**

Spherically Seated

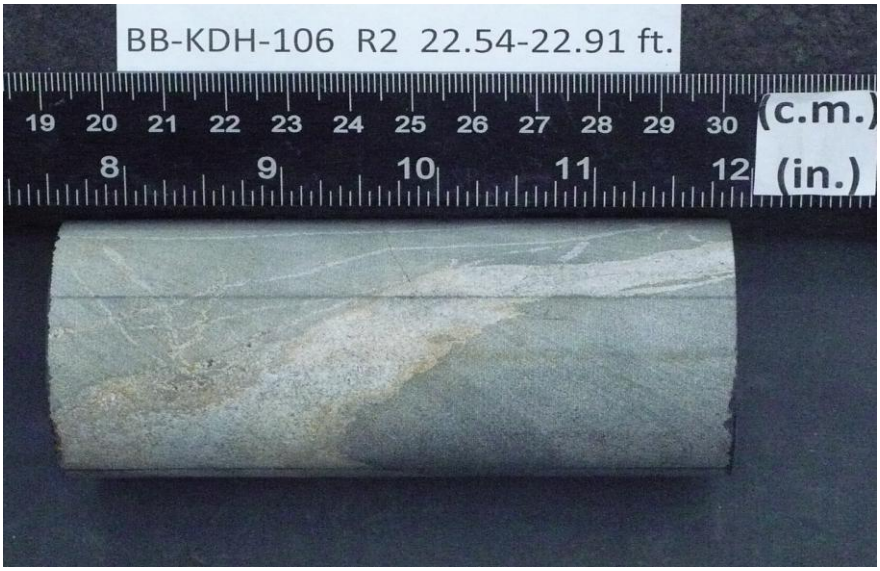
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00160	1.960	0.00082	0.047	YES
Diameter 2, in (rotated 90°)	0.00050	1.960	0.00026	0.015	YES
END 2					
Diameter 1, in	0.00150	1.960	0.00077	0.044	YES
Diameter 2, in (rotated 90°)	0.00050	1.960	0.00026	0.015	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-106
Sample ID:	R2
Depth, ft:	22.54-22.91

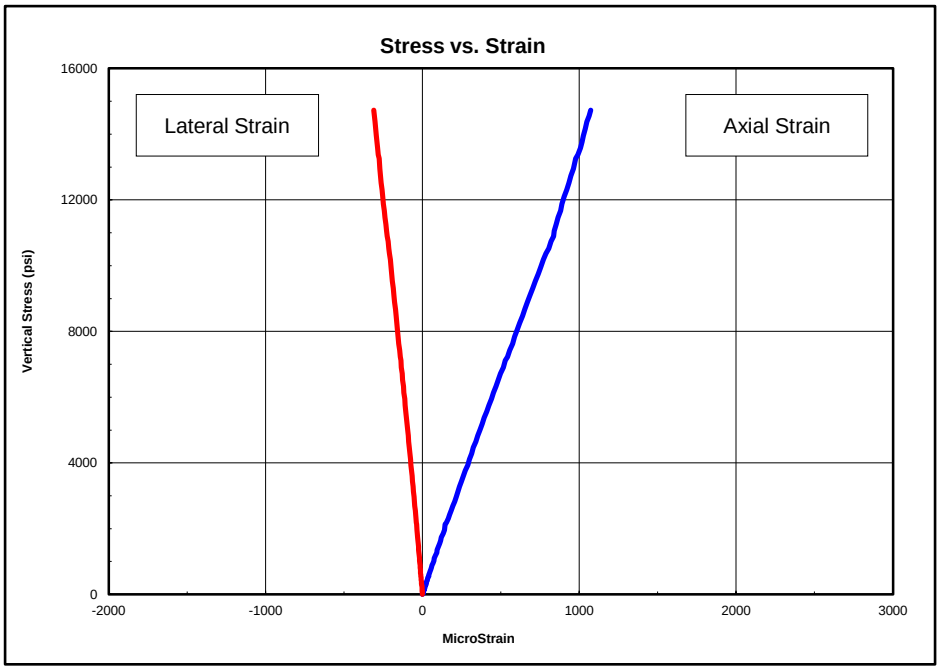


After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-106
Sample ID:	R5
Depth, ft:	30.32-30.69
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 14,797 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-5000	13,400,000	0.26
5000-10000	12,700,000	0.27
10000-14797	15,400,000	0.35

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	2/22/2012
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge	Tested By:	daa
Project Location:	Kittery, ME	Checked By:	mpd
GTX #:	11547		
Boring ID:	BB-KDH-106		
Sample ID:	R5		
Depth:	30.32-30.69 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.35	4.35	4.35
Specimen Diameter, in:	1.96	1.96	1.96
Specimen Mass, g:	585.35		
Bulk Density, lb/ft^3	170		
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00040	0.00040	0.00020	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00070
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00030	-0.00050	-0.00070

Difference between max and min readings, in:

0° = 0.00110 90° = 0.00090

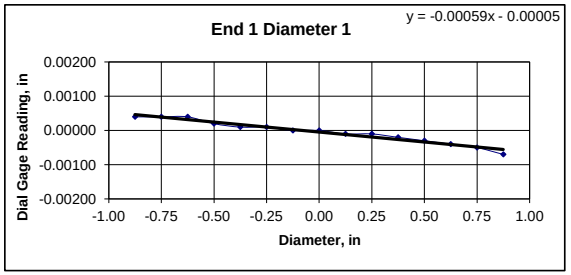
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00040	-0.00050	-0.00070	-0.00080
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060

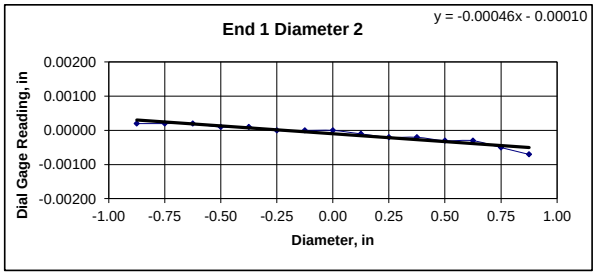
Difference between max and min readings, in:

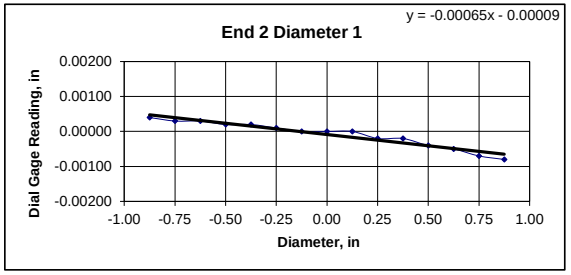
0° = 0.0012 90° = 0.0007

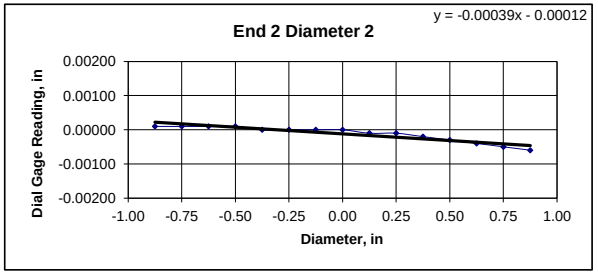
Maximum difference must be < 0.0020 in. Difference = ± 0.00060

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

Slope of Best Fit Line -0.00059

Angle of Best Fit Line: -0.03380

End 2:

Slope of Best Fit Line -0.00065

Angle of Best Fit Line: -0.03724

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? **YES**

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line -0.00046

Angle of Best Fit Line: -0.02636

End 2:

Slope of Best Fit Line -0.00039

Angle of Best Fit Line: -0.02235

Maximum Angular Difference: 0.00401

Parallelism Tolerance Met? **YES**

Spherically Seated

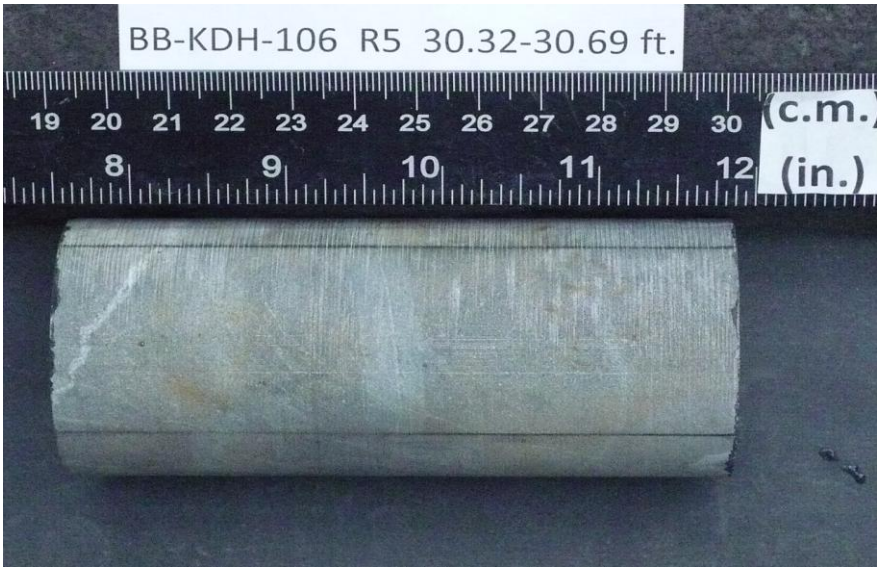
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00110	1.960	0.00056	0.032	YES
Diameter 2, in (rotated 90°)	0.00090	1.960	0.00046	0.026	YES
END 2					
Diameter 1, in	0.00120	1.960	0.00061	0.035	YES
Diameter 2, in (rotated 90°)	0.00070	1.960	0.00036	0.020	YES

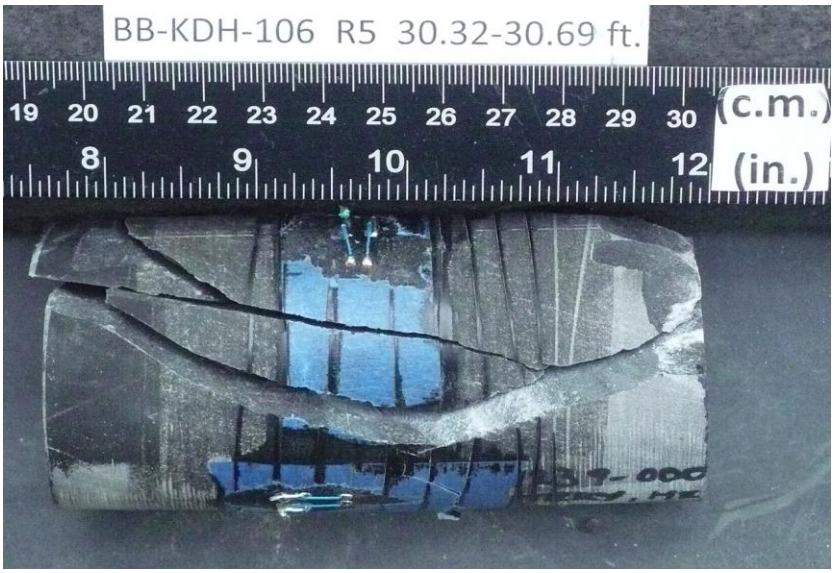
Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Bypass over Route 236 Bridge
Project Location:	Kittery, ME
GTX #:	11547
Test Date:	03/02/12
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-KDH-106
Sample ID:	R5
Depth, ft:	30.32-30.69



After cutting and grinding



After break

GeoTesting EXPRESS

SOIL CHAIN OF CUSTODY & TEST REQUEST

CLIENT	
Company: Haley & Aldrich, Inc.	
Address: 75 Washington Avenue, Suite 203	
City, State, Zip :Portland, ME 04105	
Contact: Erin Wood	E-mail: EWood@HaleyAldrich.com
Phone: 207.482.4626	Fax: 207482.4676

PROJECT	
Project Name:U.S. Route 1 Bypass over Route 236 Bridge	
Address:	Client Project #:38239-000
City, State, Zip: Kittery, Maine	
On-site Contact:	E-mail:
Phone:	Fax:

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2662 Holcomb Bridge Road, Suite 310
Alpharetta, GA 30022
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

INVOICE (complete if different from client)	
Company: Haley & Aldrich, Inc.	
Address: 70 Blanchard Rd., Ste 430	
City, State, Zip: Burlington, MA 01803-5100	
Contact: Accounts Payable	E-mail: AP@HaleyAldrich.com
Phone: 617.886.7400	Fax:

GENERAL	
Purchase Order #:	GTX Sales Order #: 0070153
Shipped By: Erin Wood	Date Shipped: 2/14/12
Mode of Shipment: Fedex	Requested Turnaround: standard
Send Results To:	☒ CLIENT OFFICE ☐ PROJECT OFFICE
Send Results Via:	☒ E-MAIL ☐ FAX ☐ VERBAL ☐ HARD COPY

SOIL

*please include boring # and depth if known

	Sample ID	Atterberg Limits (ASTM D 4318)	California Bearing Ratio (ASTM D 1883) *specify conditions below	USCS – Classification (ASTM D 2487)	Density (ASTM D 2937)	Direct Shear (ASTM D 3080) *specify conditions below	Direct Simple Shear (ASTM D 6528) *specify conditions below	Electrical Resistivity (ASTM G 57)	Grain Size (ASTM D 422) Sieve Only / Sieve & Hydrometer please circle one	Incremental Consolidation (ASTM D 2435)	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	Permeability/ Hydraulic Conductivity (Fixed Wall - ASTM D 2434) (Flexible Wall – ASTM D 5084) please circle one	Ph (ASTM D 4972)	Proctor Compaction (Standard – ASTM D 698) (Modified – ASTM D 1557) please circle one	Specific Gravity (ASTM D 854)	Triaxial Shear (UU – ASTM D 2850) (CU – ASTM D 4767) (CD – US COE EM1110) *specify conditions below please circle one	Unconfined Compression (ASTM D 2166)	Other: Unconfined Compressive Strength of Rock, with Elastic Modull	Other:
1	BB-KDH-101, R1, 12.7 – 13.3 ft																		x	
2	BB-KDH-101, R2, 15.5 – 16.2 ft																		x	
3	BB-KDH-106, R2, 22.4 – 23.2 ft																		x	
4	BB-KDH-106, R5 30.1 – 31.3																		x	
5																				
6																				
7																				

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: _____ PRINT NAME: _____ DATE: _____

Incoming Sample Inspection Performed ☑
Adverse conditions: NONE

Relinquished By:	DATE:	Received By:	DATE: 2/15/12
	TIME:		TIME: 3:37 PM
Relinquished By:	DATE:	Received By:	DATE:
	TIME:		TIME:

GeoTesting EXPRESS

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

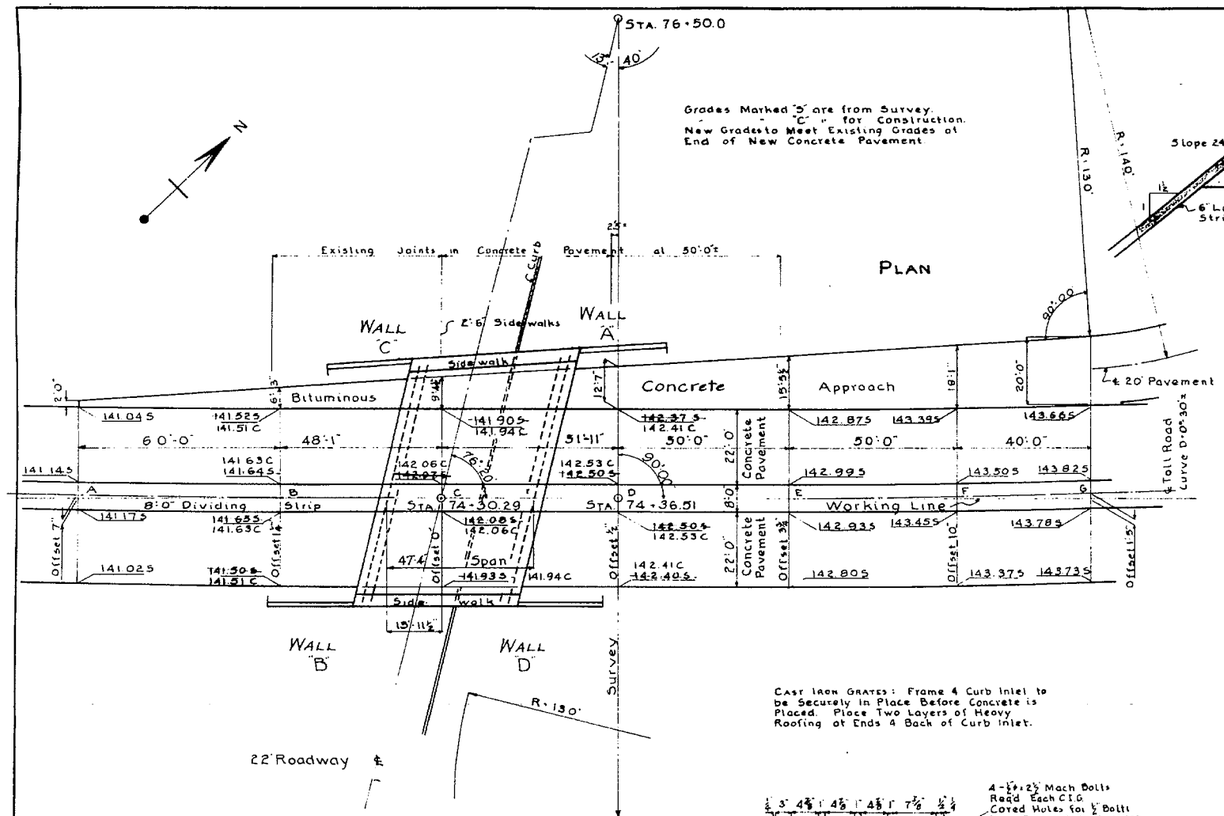
Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

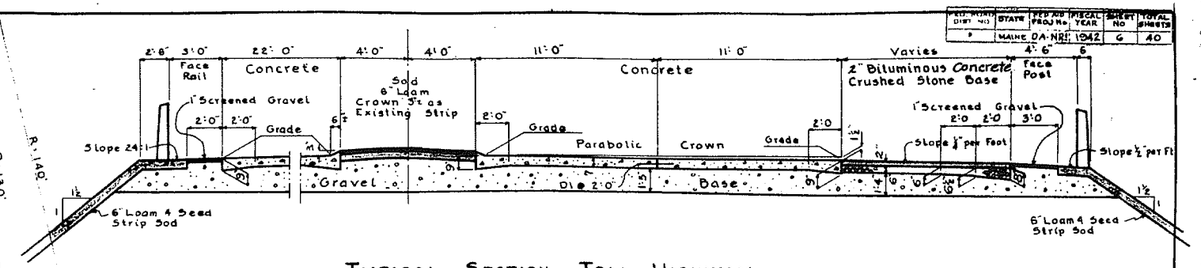
APPENDIX C

Historic Bridge Drawings

FED. PUBLIC DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	DA-NR-1	1942	6	40

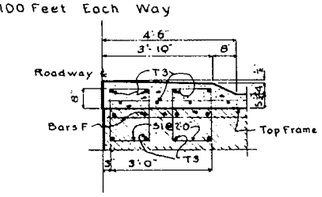


Cast Iron Grates: Frame & Curb Inlet to be Securely in Place Before Concrete is Placed. Place Two Layers of Heavy Roofing at Ends & Back of Curb Inlet.

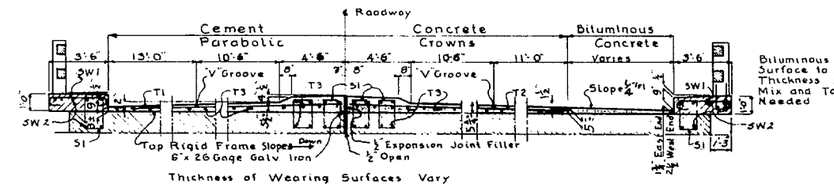


Typical Section Toll Highway

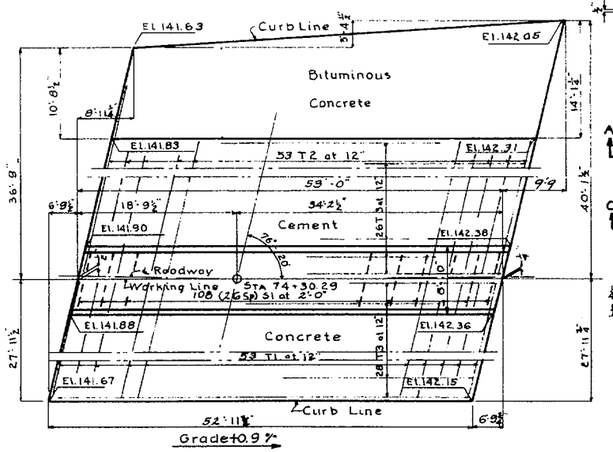
Reset Existing Guard Rail, Warp Shoulders, 100 Feet Each Way From Bridge.



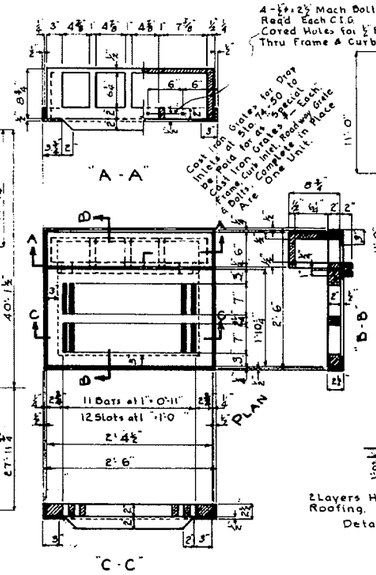
Detail Dividing Strip on Bridge



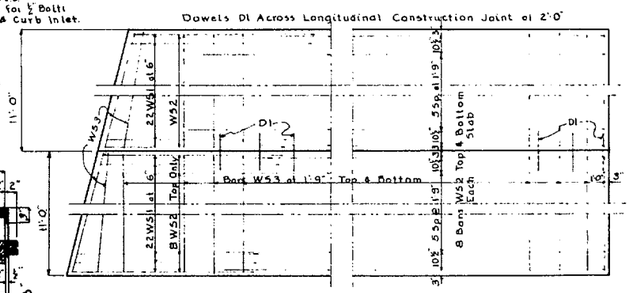
Section Roadway on Bridge



Plan Roadway Surface on Bridge



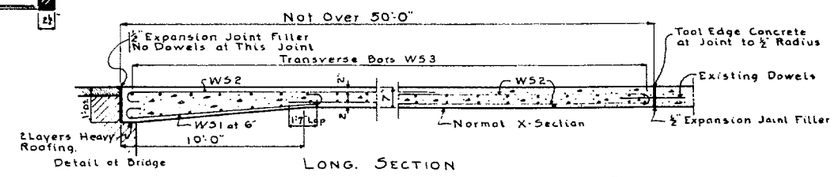
Cast Iron Grate
2 Required



Remove Existing Concrete Pavement to First Transverse Joint From Bridge. Use Care Not to Disturb Existing Dowels Across Joint.

OR Existing Slab May be Cut Between Joints. Allow Existing Steel to Extend 3'-0" From Existing Slab Clean and Wrap Extending Steel for Dowels Across New Joint.

Detail to be Determined by Engineer in Field.



Cut & Bend W52 from Random Lengths in Field. Lap 1'-7" at Splices.

Typical Detail Concrete Roadway Slab

Project No. DA-N.R. 1

Nov. 1941	Plan CDB	Town 16-12
	Check C.L.P.	Bridge

State Highway Commission
Bridge Division

Kittery Overpass over U.S. Route No. 1 in the Town of Kittery

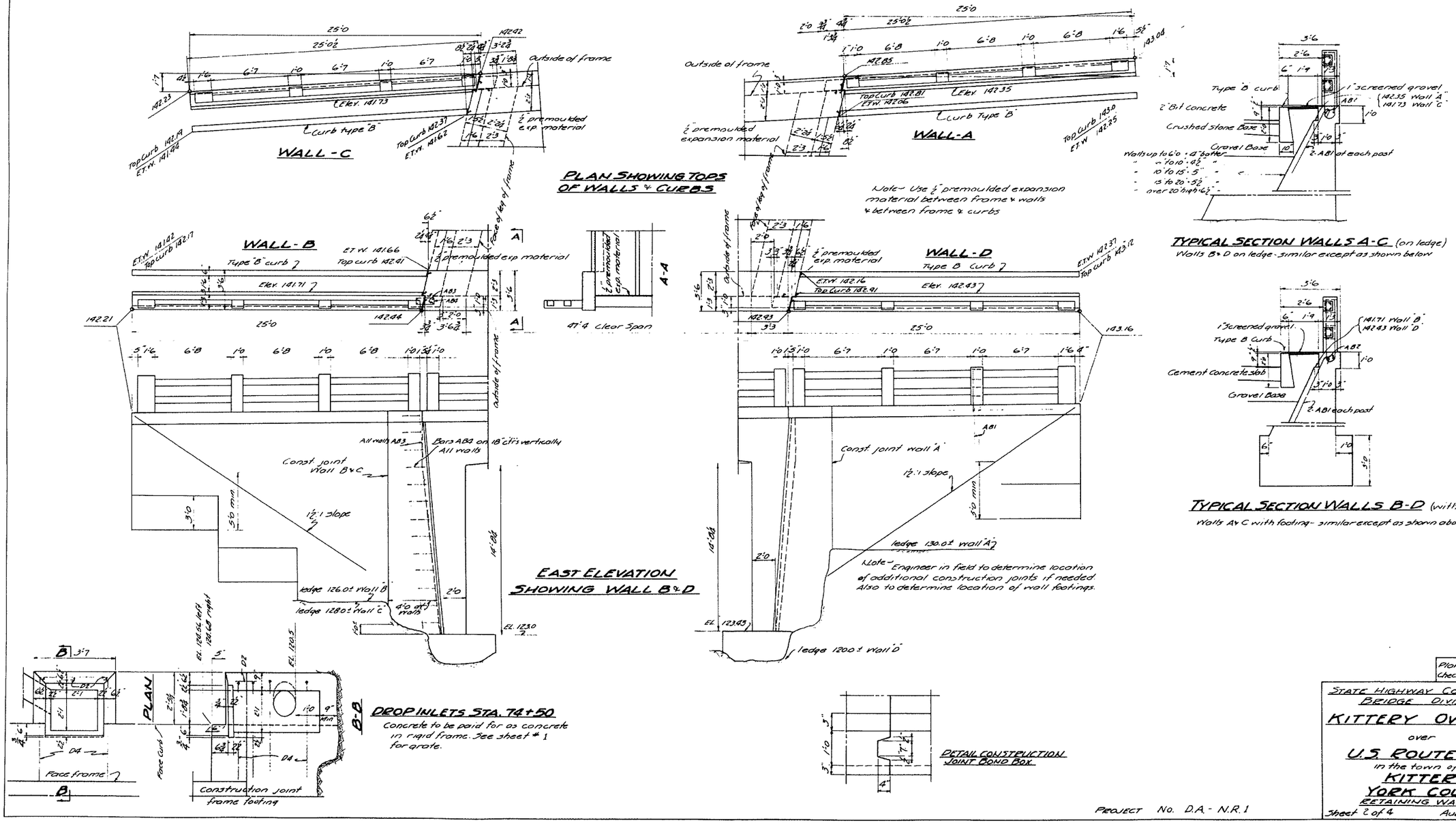
York County

Toll Road Details

Sheet 1 of 4 — Augusta, Me. — July 1941

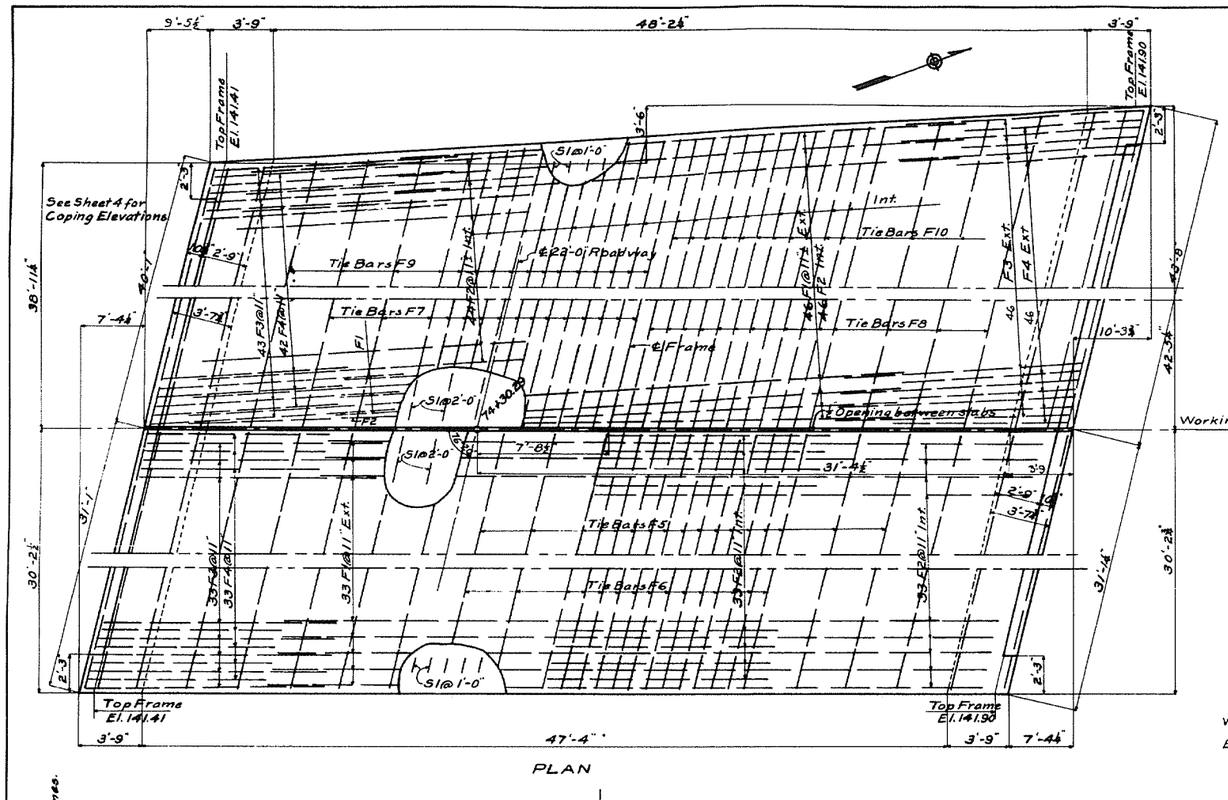
#129

FED. ROAD DIVISION	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
2	MAINE	DA-NR 1	7	40

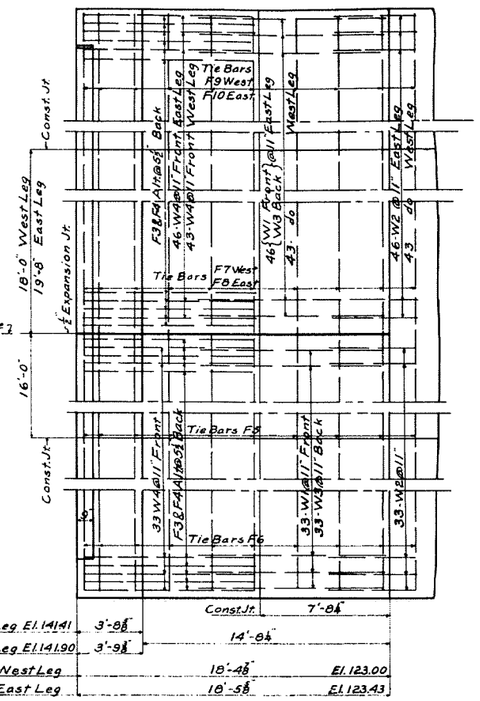


N/10 (7)

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. No.	SHEET NO	TOTAL SHEETS
9	MAINE	DA-NR 1	8	40



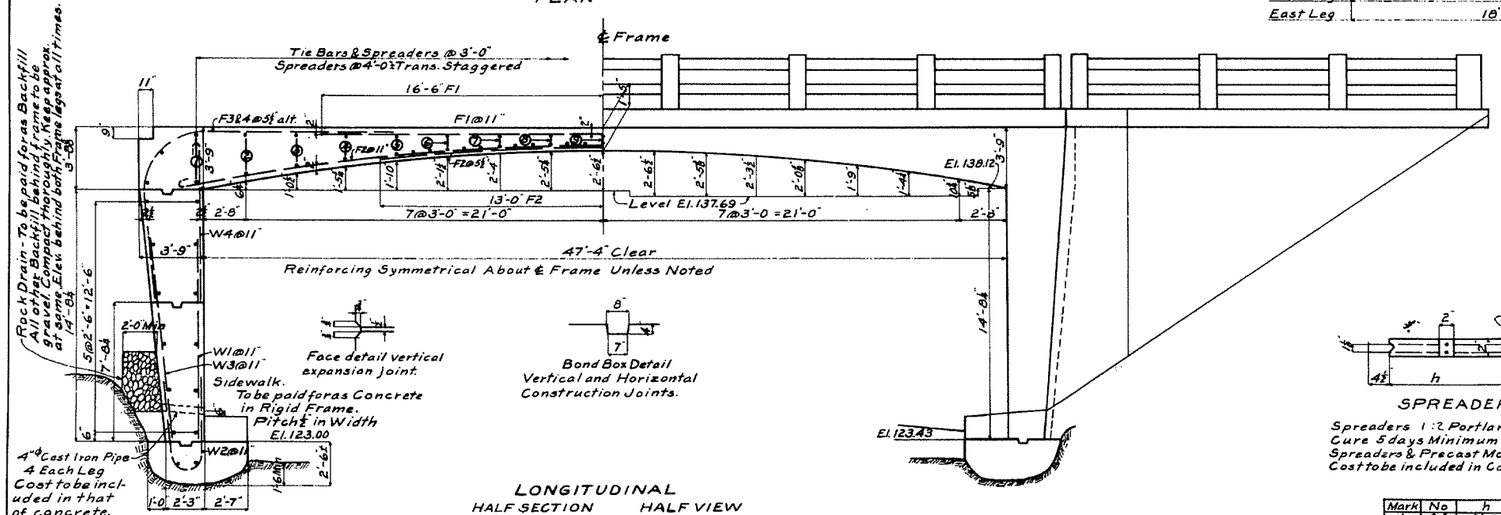
PLAN



West Leg El. 141.41	3'-8⅝"	
East Leg El. 141.90	3'-9⅝"	
West Leg	18'-4⅝"	El. 123.00
East Leg	18'-5⅝"	El. 123.43

REAR VIEW EAST LEG
REAR VIEW WEST LEG OPP. HAND

Suggested location of Vertical Construction Joints as an Aid to Maintenance of Traffic. Tie Bars F5, F7, F8 extend thru Construction Joint. Build Middle Sections First.

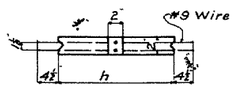


LONGITUDINAL
HALF SECTION HALF VIEW

Loading H20
fs = 18,000 fc = 1000 n = 10

Vibrate all Concrete in Rigid Frame

Cover the backside of all Expansion and Vertical Construction Joints with two layers of heavy roofing 10" wide. Paint concrete and back of each layer of roofing with hot tar or asphalt. Recess concrete ½" where roofing is to be applied. Unwatering of foundations if necessary to be included in cost of Concrete.



SPREADER

Spreaders 1:2 Portland Cement Mortar
Cure 5 days Minimum before using.
Spreaders & Precast Mortar Blocks to be staggered.
Cost to be included in Cost of Frame Concrete.

Mark	No	h
1	36	3'-2½"
2	"	2'-8½"
3	"	2'-2½"
4	"	1'-9½"
5	"	1'-3½"
6	"	1'-2½"
7	"	1'-0½"
8	"	11½"
9	18	10½"

PROJECT No. D.A.- NR 1

DESIGN B.L.A. PLAN B.L.A. TRACED L.H.W. CHECKED E.L.P. CHECKED E.G. DWN. 14-12 BRIDGE

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

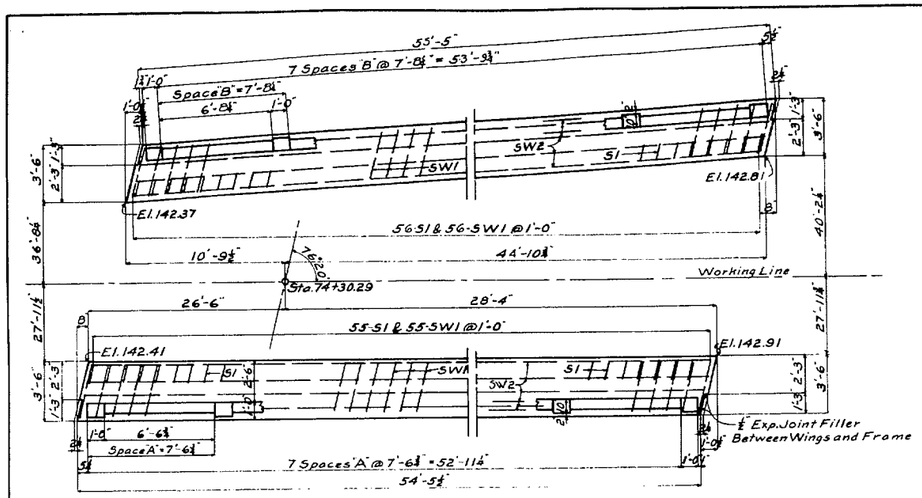
KITTERY OVERPASS
OVER
U.S. ROUTE No. 1
IN THE TOWN OF
KITTERY
YORK COUNTY

RIGID FRAME

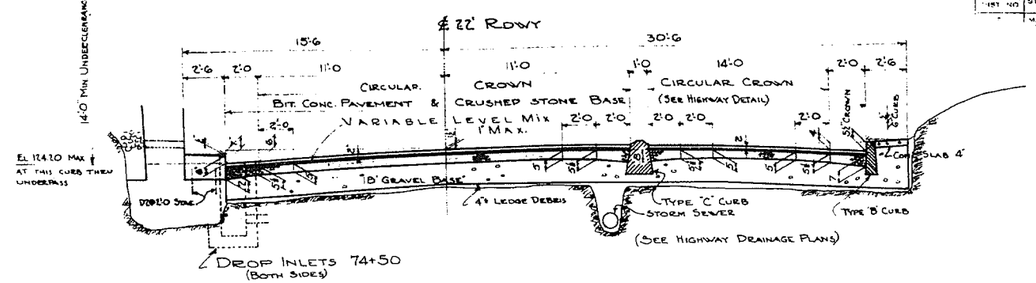
SHEET 3 OF 4 AUGUSTA, MAINE NOV. 1941

A 131 (8)

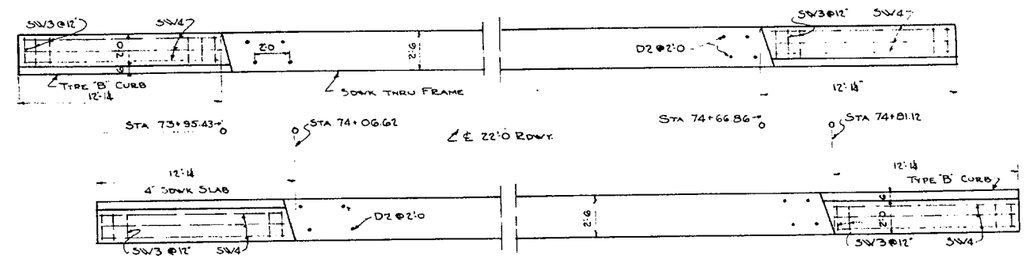
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
-	MAINE	DA-NR1	9	40



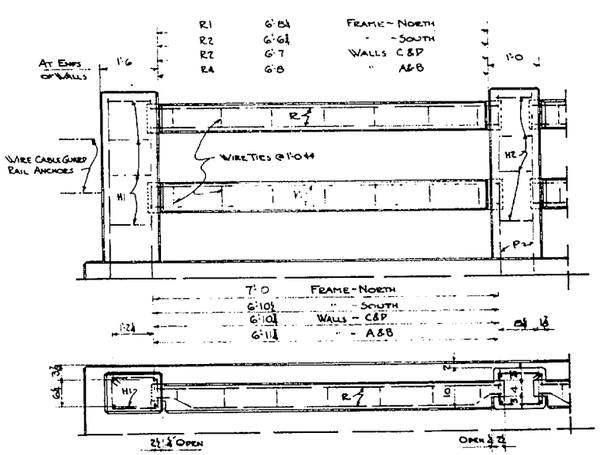
PLAN FRAME SDWK.

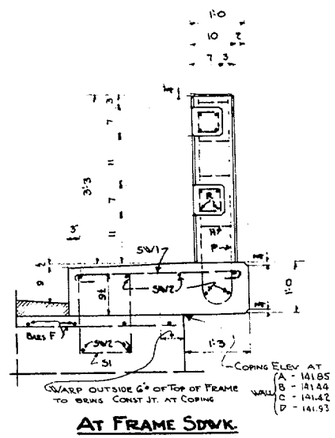


ROADWAY SECT. THRU UNDERPASS

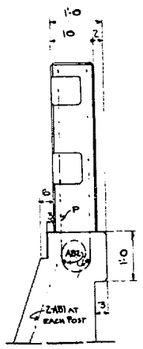


SDWK. PLAN UNDER FRAME





AT FRAME SDWK.



AT WING WALL

RAIL DETAILS

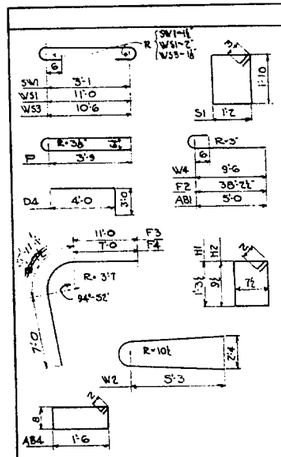
PRECAST ALL RAIL BARS. SET BARS IN POSITION SO THAT THE ENDS PROJECT 2½" INTO POST FORMS. WRAP THE TONGUE ENDS WITH 2 LAYERS OF HEAVY ROOFING.

CHAMFER EXPOSED EDGES ¾", UNLESS NOTED.

WIRE STIRRUPS FOR RAIL BARS TO BE MADE IN THE FIELD FROM A SINGLE STRAND OF #9 ANNEALED WIRE. IN FORMING STIRRUPS MAKE A COMPLETE TURN AROUND EACH BAR.

MIN. CONCRETE COVER 1".

REINFORCING STEEL



BAR	SIZE	NO	LENGTH	REMARKS
SW1	½"*	111	4'-10½	* FRAME SDWK
SW2	"	32	30'-0	" " - SPLICE
SW3	⅜"*	48	1'-10	LOWER "
SW4	"	12	11'-2	" "
P	¾"*	64	8'-3¾	* RAIL POST
H1	½"*	16	4'-2	* " "
H2	"	112	3'-2	* " "
W51	⅝"*	~~88~~ 177	13'-0	* RDWY SLAB
W52	⅝"*	—	*2600 LB	" "
W53	"	100*	12'-1	* " "
D1	½"*	100*	2'-0	" "
D2	"	160	1'-6	SDWK & D.I.
D4	½"*	16	7'-0	* D.I.
T1	⅝"*	53	28'-6	WEIR SUR.
T2	"	53	26'-6	" "
T3	"	140	27'-4	" - SPLICE

BAR	SIZE	NO	LENGTH	REMARKS
F1	¾"*	79	33'-0	FRAME - EXTRADOS
F2	1"*	156	39'-6	* " - INTRADOS
F3	1"*	155	~~24'-7½~~ 24'-11½	* - CORNER
F4	"	154	~~20'-7½~~ 20'-11½	* " "
F5	1"*	82	20'-1	" - TIE BARS
F6	"	82	15'-3	" "
F7	"	42	23'-6	" "
F8	"	40	23'-10	" "
F9	"	42	21'-6	" "
F10	"	40	24'-0	" "
R1	⅝"*	56	7'-0	RAIL BAR
R2	"	104	6'-11	"
R4	"	48	7'-0	"

BAR	SIZE	NO	LENGTH	REMARKS
S1	½"*	219	6'-6	* SDWK & CENTER STRIP
W1	1"*	155	11'-6	FRAME LEG
W2	"	155	13'-3	* "
W3	1"*	155	11'-6	"
W4	1"*	155	10'-9½	* "
AB1	¾"*	32	6'-3½	* WINGS AT RAIL POST
AB2	½"*	8	23'-0	WINGS - CUT IN FIELD
AB3	⅜"*	16	19'-0	"
AB4	"	52	4'-8	* "

#BENT BARS.
* DETERMINE EXACT FIGURE IN FIELD. DIMENSIONS TO ¼" OF BARS.

PROJECT DA-NR1

BRIDGE 16-12

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

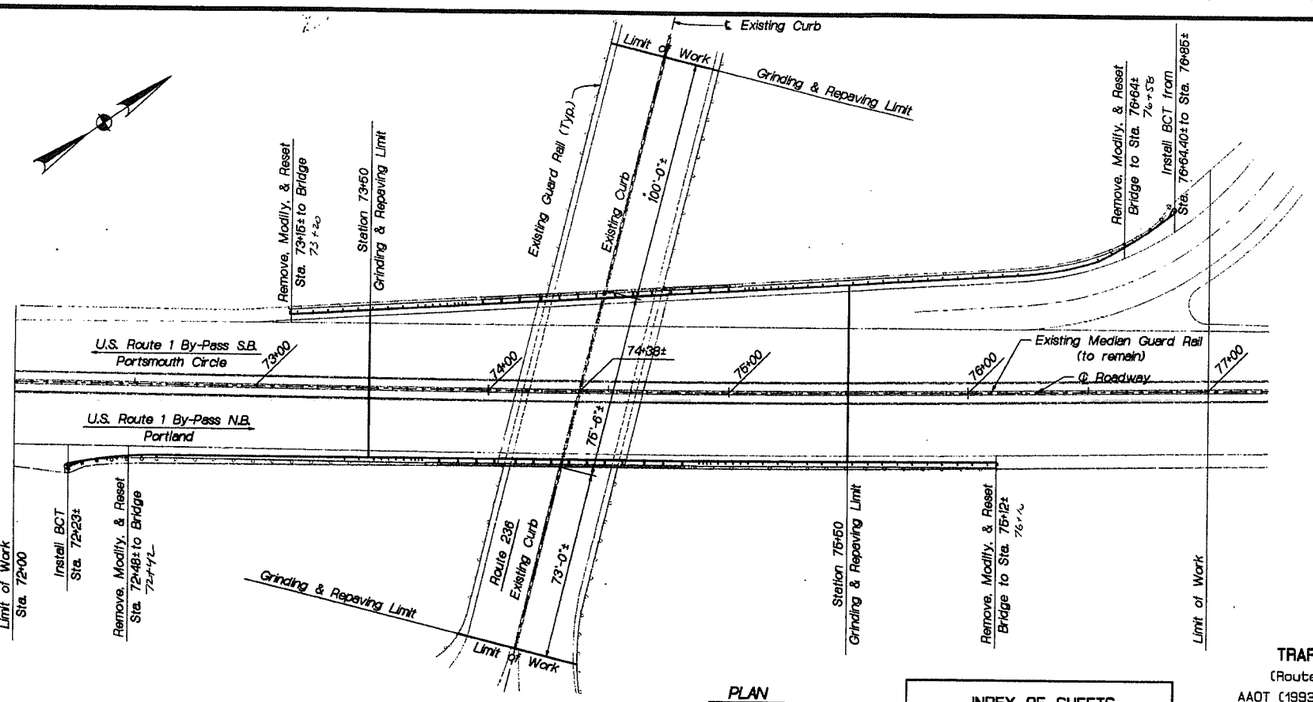
KITTERY OVERPASS
OVER
U.S. ROUTE NO. 1
IN THE TOWN OF
KITTERY, YORK COUNTY

BRIDGE DETAILS
SHEET 4 OF 4 AUGUSTA, MAINE NOV. 1941

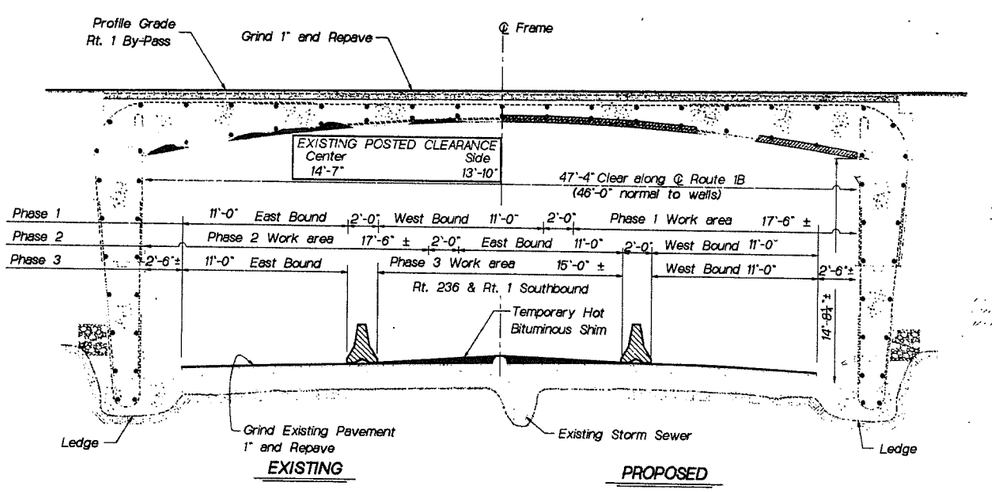
M 132 (9)

PIN 4233.00

F.H.W.A. RES. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	BH-001P(104)	1	7



PLAN



LONGITUDINAL SECTION

INDEX OF SHEETS

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HIGHWAY STANDARDS

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SPECIFICATIONS

DESIGN: Load Factor Design per AASHTO Standard Specifications for Highway Bridges 15th Edition, 1992.

CONTRACT: State of Maine, Department of Transportation, Standard Specifications, Highways and Bridges, Revision of October 1990.

DESIGN LOADING

LIVE LOAD H20 (Existing)

MATERIALS

CONCRETE: (unless otherwise specified) CLASS AAA
REINFORCING STEEL: ASTM A615 Grade 60

BASIC DESIGN STRESSES

CONCRETE: f'c = 3000 psi
SHOTCRETE: f'c = 5000 psi
REINFORCING STEEL: fy = 60,000 psi

SCOPE OF WORK

1. Rehabilitate underside of rigid frame deck while maintaining two way traffic over and under bridge, except detour Route 236 truck traffic.
2. Grind and overlay 1 inch hot bituminous pavement on Rt. 1 By-Pass (the full width of existing pavement) over the bridge and on Rt. 236 (face to face of existing guard rail) under the bridge to the limits shown on the plans.
3. Install new thrie-beam bridge rail, remove and reset existing approach rail, and install BCT'S as shown on the plans.
4. Repair spalled areas of concrete on rigid frame walls and straight back retaining walls as required.

TRAFFIC DATA

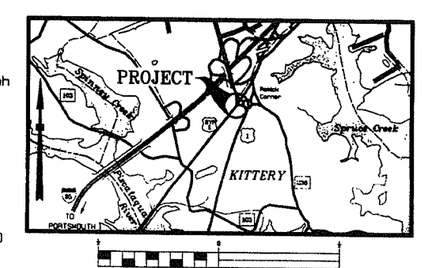
(Route 1 By-Pass)

AADT (1993)	= 6830
AADT (2013)	= 10930
DHV	= 1200
Trucks(%)	= 9
D (%)	= 51
Velocity	= 45 mph
18 Kip eq. P2.5	= 331

TRAFFIC DATA

(Route 236 & Rt. 1 S.B.)

AADT (1993)	= 21245
AADT (2013)	= 33990
DHV	= 3740
Trucks(%)	= 3
D (%)	= 61
Velocity	= 25 mph
18 Kip eq. P2.5	= 348



LOCATION MAP

321

107-361

Bridge No. 3860

Plans of existing bridge are available for the Contractor's reference at the Bridge Design Office in Augusta. These are reproductions of original drawings as prepared for the construction of the bridge and it is very unlikely that the plans will show any construction field changes or any alterations which may have been made to the bridge during its life span.

STATE OF MAINE
RICHARD A. COLEMAN 1996 REGISTERED PROFESSIONAL ENGINEER

APPROVED: State of Maine, Department of Transportation
COMMISSIONER 3-23-93 DATE
CHIEF ENGINEER 3-23-93 DATE

United States Department of Transportation, Federal Highway Administration, Region 1
APPROVED:
DIVISION ADMINISTRATOR DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

KITTERY OVERPASS (RT. 1B)
OVER
RT. 236 & U.S. RT. 1 S.B.
IN THE TOWN OF
KITTERY
YORK COUNTY

GENERAL PLAN

SHEET 1 OF 7 AUGUSTA, MAINE February 1993

PROJECT DESIGN ENGINEER
PLANS
DESIGN-DETAILED BY J. Forrest Cauffman DATE 9/93
CHECKED
REVISIONS
FIELD CHANGES

23MM93-01D130
PRELIM (GENPLAN)

PIN 4233.00

F.H.W.A. RES. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	BH-001P(104)	2	7

ESTIMATED QUANTITIES

ITEM NO.	DESCRIPTION	QUANTITY	UNIT	
202.202	REMOVING PAVEMENT SURFACE	2450	SY	
403.07	HOT BITUMINOUS PAVEMENT, GRADING B	53	TONS	
403.10	HOT BITUMINOUS PAVEMENT, GRADING D	135	TONS	
409.15	BITUMINOUS TACK COAT, APPLIED	125	GAL	
517.58	SHOTCRETE THICKNESS OF 8 INCHES OR LESS (OVERHEAD)	2700	SF	• CONTRACTORS OPTION, SEE NOTE #1
518.38	REHABILITATE STRUCTURAL CONCRETE SLAB (OVERHEAD)	2700	SF	
517.58	SHOTCRETE THICKNESS OF 8 INCHES OR LESS (VERTICAL)	330	SF	• CONTRACTORS OPTION, SEE NOTE #1
518.38	REHABILITATE STRUCTURAL CONCRETE SLAB (VERTICAL)	330	SF	
526.301	TEMPORARY CONCRETE BARRIER TYPE I (600 LF)	1	LS	
527.32	PORTABLE CRASH BARRELS	18	EACH	
606.35	GUARD RAIL DELINEATOR POST	4	EACH	
606.364	GUARD RAIL, REMOVE, MODIFY AND RESET	540	LF	
606.367	REPLACE UNUSABLE EXISTING GUARD RAIL POST	20	EACH	
606.70	TRANSITION SECTION THRIE BEAM	4	EACH	
606.73	GUARD RAIL THRIE BEAM - SINGLE RAIL BRIDGE MOUNTED	225	LF	
606.77	BREAKAWAY CABLE TERMINAL	2	EACH	
627.61	4 INCH SOLID WHITE PAVEMENT MARKING LINE	1000	LF	
627.62	4 INCH BROKEN WHITE PAVEMENT MARKING LINE	500	LF	
627.63	4 INCH SOLID YELLOW PAVEMENT MARKING LINE	1000	LF	
627.67	REMOVING PAVEMENT MARKINGS	100	SF	
627.69	TEMP. 4" PLASTIC PAVEMENT MARKING LINE, YELLOW OR WHITE	2500	LF	
629.05	HAND LABOR, STRAIGHT TIME	10	MH	
631.12	ALL PURPOSE EXCAVATOR (INCLUDING OPERATOR)	5	HOUR	
631.172	TRUCK--LARGE (INCLUDING OPERATOR)	5	HOUR	
639.19	FIELD OFFICE TYPE A	1	EACH	
639.23	TESTING FACILITIES CONCRETE	1	LS	
652.31	TYPE I BARRICADE	5	EACH	
652.33	DRUM	30	EACH	
652.34	CONE	20	EACH	
652.35	CONSTRUCTION SIGNS	1120	SF	
652.361	MAINTENANCE OF TRAFFIC CONTROL DEVICES	1	LS	
652.38	FLAGGER	1600	MH	
659.10	MOBILIZATION	1	LS	

GENERAL CONSTRUCTION NOTES

1. The Contractor will be allowed to choose either Item 517.58 or 518.38 for vertical wall and overhead concrete repairs. The Contractor must select one alternate for vertical work and one alternate for overhead work.

2. Traffic shall be protected at all times from falling concrete by using falsework or other methods approved by the Engineer. Payment for any labor or materials will be considered incidental to Items 517 and/or 518.

3. A minimum of 12'-0" underclearance under the bridge shall be maintained at all times except within the work areas.

4. Shim with bituminous pavement as required to maintain traffic over the existing median curb under the bridge. Payment shall be made under Item 403.07 Hot Bituminous Pavement, Grading B. The shim shall be removed in its entirety prior to the placement of the new Hot Bituminous Pavement. Payment for this work shall be considered incidental to related contract items.

5. Each of the three Work Areas shall be divided into four panels. Route 1 By-Pass traffic over the bridge shall be aligned such that traffic is not directly over the panels concurrently being repaired.

6. Concrete shall not be removed from the underside of the deck until such time as the individual panel is to be repaired. The concrete shall not be removed from more than 2 panels at a time.

7. The Contractor shall not remove existing concrete within 72 hours following the completion of a concrete or shotcrete placement.

8. The perimeter of the underside of the deck panels shall have square edged joints. Any saw cutting required to achieve a minimum ½" square edge will be incidental to the related contract items.

9. Breakaway cable terminals shall be installed concurrent with the placement of each section of beam guard rail, unless other approved temporary protection has been authorized. There will be no additional payment for other approved protection.

10. One guard rail delineator post and one terminal end shall be installed at each guard rail end.

107-362 ~~322~~

PROJECT DESIGN ENGINEER	BY	DATE
DESIGN-DETAILED	J. Foltz	3/93
CHECKED		
REVISIONS		
FIELD CHANGES		

PLANS

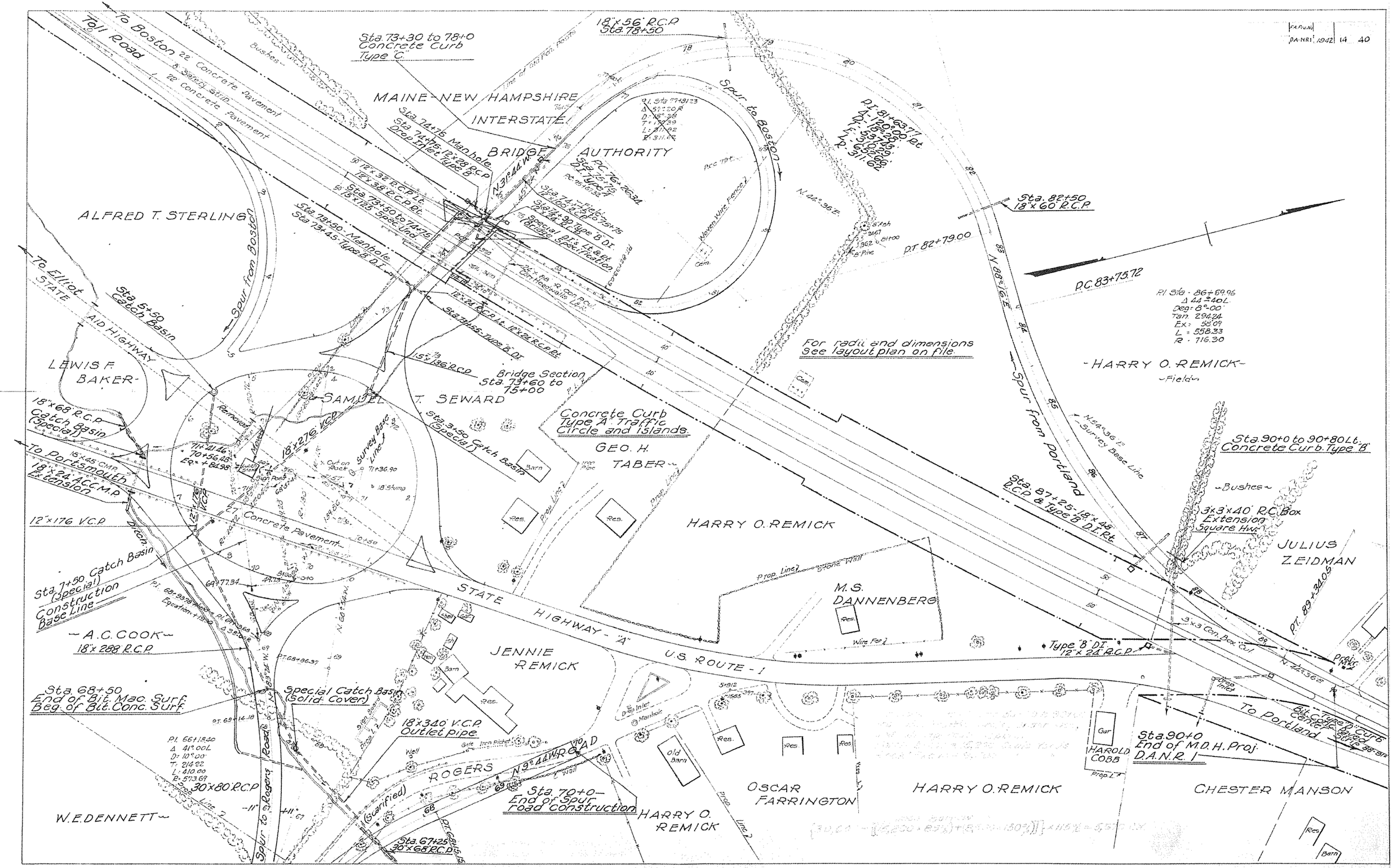
23MA193-01.00.30
ESTIMATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

KITTERY OVERPASS (RT. 1B)
OVER
RT. 236 & U.S. RT. 1 S.B.
IN THE TOWN OF
KITTERY
YORK COUNTY

QUANTITIES & NOTES

SHEET 2 OF 7 AUGUSTA, MAINE February 1993



KITTERY - No. D.A. NR1

APPENDIX D

Calculations

Approach Embankment Settlement

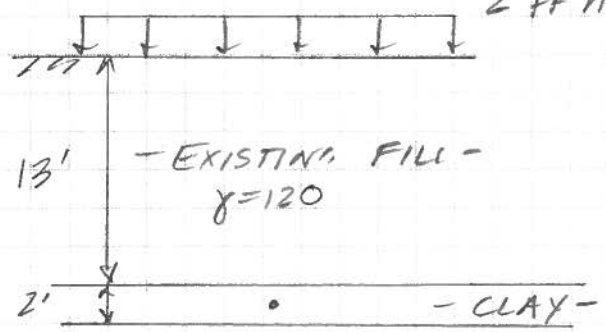
HALEY & ALDRICH

Calculations

Client	McFarland-Johnson	File No.	38239-100
Project	U.S. Route 1 Bypass	Sheet	1 of 1
Subject	Embankment Settlement	Date	11/22/13
		Computed By	EFW
		Checked By	WAC

At boring BB-KDH-105

2 ft new fill = (2)(125) = 250 psf



γ = 110 pcf CR = 0.2
OCR = 1.0

$\sigma'_{vo} = (13\ ft)(120\ pcf) + (1\ ft)(110\ pcf) = 1670\ psf$

$\Delta\sigma_v = 250\ psf$

$\sigma'_{vf} = 1670 + 250 = 1920\ psf$

$$\delta = (2\ ft)(0.2)\log\left(\frac{1920\ psf}{1670\ psf}\right)\cdot\frac{12\ in}{1\ ft} = \underline{0.3\ in}$$

MAXIMUM CONSOLIDATION SETTLEMENT ASSUMING NORMALLY CONSOLIDATED CLAY DEPOSIT; BASED ON SPT'S WE ANTICIPATE THAT CR < 0.2

- IF OVER CONSOLIDATED, (CR ~ 0.02) THE ESTIMATE SETTLEMENT ≈ 0.03"
- POST-CONSTRUCTION SETTLEMENT LIKELY BETWEEN 0 & 1/4", SAY MAX 1/4".

Seismic Design

HALEY & ALDRICH	CALCULATIONS	File No.	38239-100
		Sheet	1 of 3
Client	Maine Department of Transportation	Date	9/27/2013
Project	U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine	Computed By	EFW
Subject	Seismic Site Class & Design Parameters	Checked By	EK

OBJECTIVE:

Assess the seismic hazard at the site in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition 2011 and the MaineDOT Bridge Design Guide 2003 with interim revisions through August 2008.

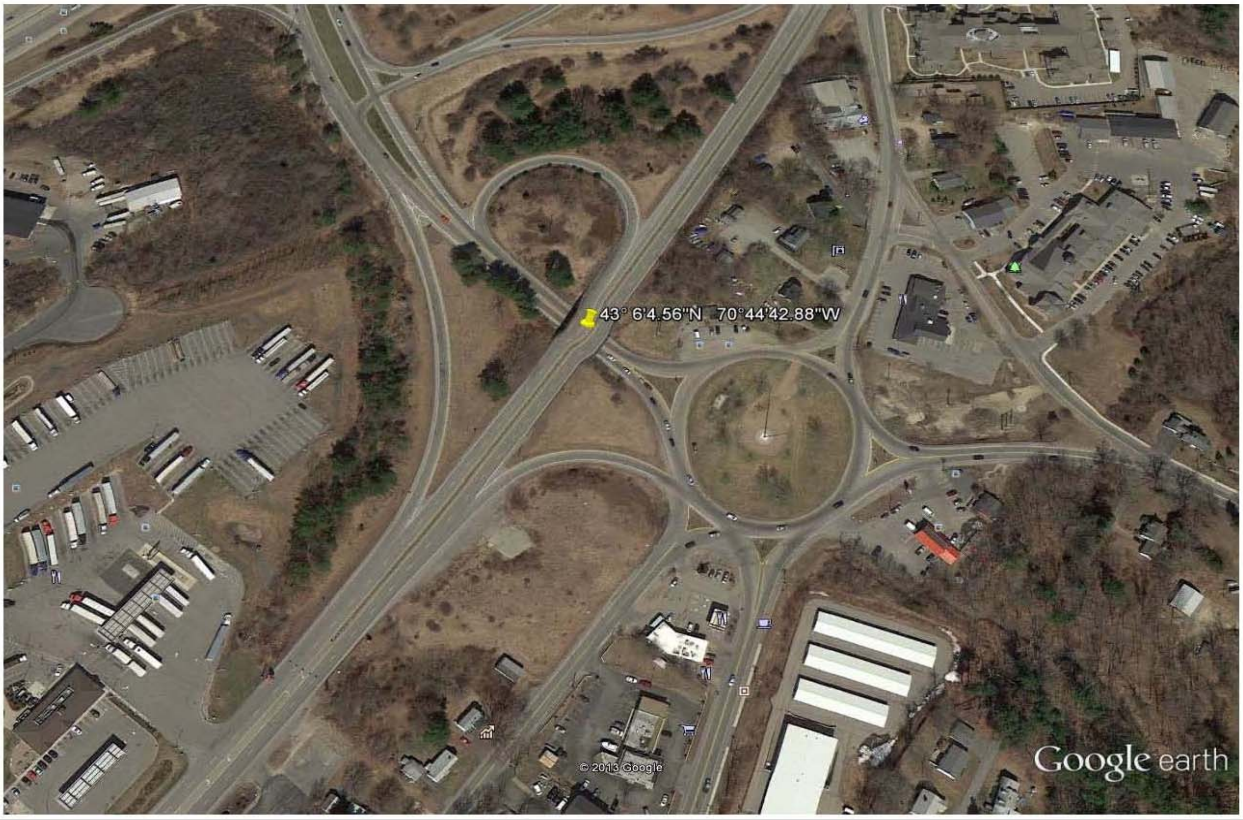
METHODOLOGY:

1 Determine geographic location of the site (latitude-longitude coordinates).
2 Determine Site Class in accordance with Section 3.4.2.2 (N method for cohesionless soils) considering only test borings drilled at proposed substructure locations. These test borings were drilled into bedrock.
For sublayers with an SPT N-value equal to 0 bpf or in excess of 100 bpf, default values of 1 and 100 bpf were used, respectively.
3 Determine Site Factors and Elastic Seismic Response Coeffecients based on (1) and (2) in accordance with Section 3.4.1. Factors are determined using the AASHTO earthquake ground motion parameters application v. 2.10 for a seismic event having a 7 percent probability of exceedence in 75 years.
4 Determine Seismic Design Category based on (3) in accordance with Section 3.5.

1. GEOGRAPHIC LOCATION:

Determine latitude-longitude site coordinates using Google Earth.

Latitude =	43° 6' 5" North	Convert to decimal degrees.	Latitude =	43.1014° North
Longitude =	70° 44' 43" West		Longitude =	70.7453° West



HALEY & ALDRICH

CALCULATIONS

File No.	38239-100
Sheet	2 of 3
Date	9/27/2013
Computed By	EFW
Checked By	EK

Client Maine Department of Transportation

Project U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine

Subject Seismic Site Class & Design Parameters

2. SITE CLASS:

BB-KDH-101 45.3 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
42.3	3.0	21	4.5	53
39.3	6.0	9	4.0	
34.3	11.0	10	3.3	
-10.2	55.5	100	88.2	
		Total =	100.0	

BB-KDH-103 27.6 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
26.1	1.5	34	2.0	95
24.6	3.0	65	3.0	
-15.3	52.5	100	95.0	
		Total =	100.0	

BB-KDH-104 27.7 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
26.7	1.0	44	2.4	97
-23.5	51.2	100	97.6	
		Total =	100.0	

BB-KDH-106 45.7 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
43.7	2.0	37	4.0	43
39.7	6.0	11	4.5	
34.7	11.0	9	5.0	
29.7	16.0	13	5.4	
-13.8	59.5	100	81.1	
		Total =	100.0	

BB-KDH-201 37.2 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
36.2	1.0	8	3.5	62
31.2	6.0	23	5.0	
26.2	11.0	56	3.9	
-19.0	56.2	100	87.6	
		Total =	100.0	

BB-KDH-202 45.6 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
21.6	6.0	15	7.4	42
16.6	11.0	4	4.1	
-20.1	57.3	100	88.5	
		Total =	100.0	

BB-KDH-203 46.2 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
35.0	2.25	100	4.5	46
31.2	6.0	5	4.0	
26.2	11.0	14	5.0	
21.2	16.0	28	3.5	
-21.3	58.5	100	83.0	
		Total =	100.0	

BB-KDH-204 32.1 (existing ground surface elevation)

Elevation (ft, NAVD 88)	Depth (ft, BGS)	N (bpf)	Thickness (ft)	Weighted SPT N-Value (bpf)
26.6	1.0	12	3.5	78
21.6	6.0	58	2.9	
-16.0	53.2	100	93.6	
		Total =	100.0	

HALEY & ALDRICH

CALCULATIONS

File No.	38239-100		
Sheet	3 of 3		
Date	9/27/2013		
Computed By	EFW		
Checked By	EK		

Client Maine Department of Transportation

Project U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine

Subject Seismic Site Class & Design Parameters

2. SITE CLASS: (CONT.)

Notes:
1. For test boring locations where bedrock was encountered within 100 ft of ground surface/mudline, the thickness of the rock layer was adjusted so that the total thickness of material encountered at each test boring location is equal to 100 ft.
2. For sublayers where the average SPT N60 value was in excess of 100 bpf a default value of 100 bpf was used in accordance with AASHTO LRFD. Applicable cells are highlighted in orange.
3. For sublayers where the SPT N60 value was equal to 0 bpf a default value of 1 bpf was used. Applicable cells are highlighted in green.

Summarize weighted SPT N60 values for each test boring….

Test Boring No.	Weighted SPT N-Value (bpf)
BB-KDH-101	53
BB-KDH-103	95
BB-KDH-104	97
BB-KDH-106	43
BB-KDH-201	62
BB-KDH-202	42
BB-KDH-203	46
BB-KDH-204	78
Average =	65

In accordance with Table 1613.5.2 for SPT N > 50 bpf the site should be considered Site Class C.

Determine site factors and elastic seismic response coefficients based on geographic location and Site Class C.

3. SITE FACTORS & ELASTIC SEISMIC RESPONSE COEFFICIENTS:

Determine parameters using the Seismic Design Parameters v. 2.0 application developed by USGS and included in the AASHTO LRFD Bridge Design Specifications. The values summarized below are based on the AASHTO response spectra for an event having a 7 percent probability of exceedance in 75 years.

Design Parameter	Design Value
Site Factor for short-period range of acceleration response spectrum, $\mathbf{F_a}$ =	1.20
Site Factor for long-period range of acceleration response spectrum, $\mathbf{F_v}$ =	1.70
Horizontal response spectral acceleration coefficient at 0.2-s period on rock, $\mathbf{S_S}$ =	0.189
Horizontal response spectral acceleration coefficient at 1.0-s period on rock, $\mathbf{S_1}$ =	0.044
Peak seismic ground acceleration coefficient on rock, **PGA =**	0.099
Site factor at zero-period on acceleration response spectrum, $\mathbf{F_{pga}}$ **=**	1.2
Horizontal response spectral acceleration coefficient at 0.2-s period modified by F_a, $\mathbf{S_{DS}}$ =	0.227
Horizontal response sepctral acceleration coefficient at 1.0-s period modified by F_v, $\mathbf{SD_1}$ **=**	0.075

Bearing Resistance

HALEY & ALDRICH	CALCULATIONS	File No.	38239-100
		Sheet	1 of 3
Client	Maine Department of Transportation	Date	9/9/2013
Project	U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine	Computed By	EFW
Subject	Abutment Footing Bearing Resistance Evaluation	Checked By	BCS

OBJECTIVE:

Evaluate bearing resistance for proposed bridge abutments bearing on bedrock in accordance with AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG).

REFERENCES:

1 AASHTO LRFD Bridge Design Specifications 6th Edition 2012.
2 MaineDOT Bridge Design Guide 2003, with Interim Revisions through August 2008.

SUMMARY OF BEDROCK DATA:

BEDROCK CORE DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Rock Core Length (ft)	Recovery		RQD	
					in.	%	in.	%
BB-KDH-101	R1	12.0	15.0	3.0	36	100%	22	61%
BB-KDH-101	R2	15.0	17.0	2.0	24	100%	19	79%
BB-KDH-101	R3	17.0	19.9	2.9	35	101%	8	23%
BB-KDH-101	R4	19.9	22.0	2.1	25	99%	8	32%
BB-KDH-101	R5	22.0	26.0	4.0	46	96%	31	65%
BB-KDH-103	R1	5.2	6.2	1.0	12	100%	0	0%
BB-KDH-103	R2	6.2	7.2	1.0	12	100%	0	0%
BB-KDH-103	R3	7.2	10.0	2.8	34	101%	8	24%
BB-KDH-106	R1	19.0	21.0	2.0	24	100%	0	0%
BB-KDH-106	R2	21.0	24.4	3.4	34	83%	16	39%
BB-KDH-106	R3	24.4	26.5	2.1	25	99%	8	32%
BB-KDH-106	R4	26.5	29.1	2.6	27	87%	11	36%
BB-KDH-106	R5	29.1	31.7	2.6	28	90%	24	77%
BB-KDH-201	R1	12.4	17.1	4.7	56	99%	31	55%
BB-KDH-202	R1	14.5	19.5	5.0	60	100%	51	85%
BB-KDH-203	R1	17.0	22.0	5.0	60	100%	30	50%
BB-KDH-204	R1	6.4	9.9	3.5	42	100%	6	14%
BB-KDH-204	R2	9.9	10.9	1.0	12	100%	0	0%

LABORATORY TEST DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Peak Compressive Stress (psi)	Bulk Density (pcf)
BB-KDH-101	R1	12.8	13.1	41,002	171
BB-KDH-101	R2	15.7	16.0	30,720	170
BB-KDH-106	R2	22.5	22.9	12,670	168
BB-KDH-106	R5	30.3	30.7	14,797	170

BEDROCK DESCRIPTION:

Aphanitic SLATE. Hard, fresh to slightly weathered. Joints dipping at low to steep angles, very close to close, tight to open, planar to stepped, smooth to rough.

Fine-grained to aphanitic PHYLLITE. Hard, fresh to moderately weathered. Joints dipping at low to steep angles, very close to moderately close, planar to stepped, smooth to rough, tight to open.

For additional details refer to test boring logs.

BEARING RESISTANCE EVALUATION:

SERVICE LIMIT STATE:

Determine service limit state bearing resistance based on presumptive values provided in Table C10.6.2.6.1-1 (attached).

For foliated metamorphic rock (slate) values range from 60 to 80 kips per square foot (ksf).
For weathered or broken bedrock values range from 16 to 24 ksf.

For service limit state design use the following: 30 ksf (abutments)

STRENGTH LIMIT STATE:

Calculate upper and lower bound nominal bearing resistance for spread footings bearing on bedrock in accordance with Section 10.8.3.5.4c.

Intact/Tightly Jointed Rock: upper bound value

qp = 2.5 x qu (Eqn. 10.8.3.5.4c-1)

Where: qu = unconfined compressive strengh of rock (ksf)

From previous page, 12.67 ksi < qu < 41.002 ksi…… average = 25 ksi
3571 ksf

qu = 3571 ksf

qp = 8,927 ksf

Jointed Rock: lower bound solution

$qp = ((s)^{1/2} + (m(s)^{1/2} + s)^{1/2}) \times qu$ (Eqn. 10.8.3.5.4c-2)

Where: s,m = fractured rock mass parameters specified in Table 10.4.6.4-4.

parameters s and m are a function of rock quality as determined use the Rock Mass Rating (RMR) system.
Determine rock quality in accordance with Tables 10.4.6.4-1 and 10.4.6.4-2 (see attached spreadsheet summary).

For slate and phyllite, assume Rock Type B. Calculate m and s directly based on Hoek & Brown, 1988.

m/mi = exp ((RMR-100)/14)

where mi = m for intact rock

from Table 10.4.6.4-4, Rock Type B, m for intact rock = 10

therefore, m = 10 exp ((RMR-100)/14)

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

HALEY & ALDRICH	CALCULATIONS	File No.	38239-100
		Sheet	3 of 3
Client	Maine Department of Transportation	Date	9/9/2013
Project	U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine	Computed By	EFW
Subject	Abutment Footing Bearing Resistance Evaluation	Checked By	BCS

BEARING RESISTANCE EVALUATION: (CONT.)

STRENGTH LIMIT STATE: (CONT.)

$qp = ((s)^{1/2} + (m(s)^{1/2} + s)^{1/2}) \times qu$ (Eqn. 10.8.3.5.4c-2)

Test Boring No.	Sample No.	RMR	m	s	qp	
BB-KDH-101	R1	50.0	0.281	2.404E-04	298	abutment
BB-KDH-101	R2	54.0	0.374	4.682E-04	408	abutment
BB-KDH-101	R3	40.0	0.138	4.540E-05	135	abutment
BB-KDH-101	R4	45.0	0.197	1.045E-04	201	abutment
BB-KDH-101	R5	50.0	0.281	2.404E-04	298	abutment
BB-KDH-103	R1	40.0	0.138	4.540E-05	135	wingwall
BB-KDH-103	R2	40.0	0.138	4.540E-05	135	wingwall
BB-KDH-103	R3	40.0	0.138	4.540E-05	135	wingwall
BB-KDH-106	R1	32.0	0.078	1.197E-05	72	abutment
BB-KDH-106	R2	37.0	0.111	2.754E-05	107	abutment
BB-KDH-106	R3	37.0	0.111	2.754E-05	107	abutment
BB-KDH-106	R4	37.0	0.111	2.754E-05	107	abutment
BB-KDH-106	R5	56.0	0.432	6.534E-04	477	abutment
BB-KDH-201	R1	42.0	0.159	6.336E-05	159	wingwall
BB-KDH-202	R1	46.0	0.211	1.234E-04	217	abutment
BB-KDH-203	R1	42.0	0.159	6.336E-05	159	abutment
BB-KDH-204	R1	21.0	0.035	1.913E-06	30	wingwall
BB-KDH-204	R2	21.0	0.035	1.913E-06	30	wingwall

At abutments:

qp (average): 215 ksf
qp (minimum): 72 ksf
qp (maximum): 477 ksf

Select qp: 100 ksf (abutments)

Resistance factor for bearing resistance of footings on rock, ϕ = 0.45

Calculate factored bearing resistance = 45 ksf (abutments)

EXTREME EVENT LIMIT STATE:

Based on above calculation, qp = 100 ksf (abutments)

Resistance factor for the extreme event, ϕ = 1.0

Calculate factored bearing resistance = 100 ksf (abutments)

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. 45.3 | X G/S | ☐ Mudline | Core Barrel Type NQ | Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name U.S. Route 1 Bypass Bridge over Route 236

Drilling Contractor Maine Test Borings

Project Location Kittery, Maine

Drill Rig Type Mobile Drill B-53 Truck-Mounted Drill Rig

File Number 38239-100

Client Maine Department of Transportation

Haley & Aldrich Rep. Marleigh Snow

Date 9/9/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-KDH-101	R1	12.0	15.0	33.3	30.3	slate	41,002 psi	15	36	100%	22	61%	13	<2 - 12		10	.004 - .1		12	-7	7	**50**
BB-KDH-101	R2	15.0	17.0	30.3	28.3	slate	30,720 psi	15	24	100%	19	79%	17	<2 - 12		10	.004 - .1		12	-7	7	**54**
BB-KDH-101	R3	17.0	19.9	28.3	25.4	slate	no test	15	35	101%	8	23%	3	<2 - 12		10	.004 - .1		12	-7	7	**40**
BB-KDH-101	R4	19.9	22.0	25.4	23.3	slate	no test	15	25	99%	8	32%	8	<2 - 12		10	.004 - .1		12	-7	7	**45**
BB-KDH-101	R5	22.0	26.0	23.3	19.3	slate	no test	15	46	96%	31	65%	13	<2 - 12		10	.004 - .1		12	-7	7	**50**
																					ave:	**48**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Condition of Joints

	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions

	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: BB-KDH-101

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. 27.6 | X G/S | ☐ Mudline | Core Barrel Type NQ | Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name U.S. Route 1 Bypass Bridge over Route 236

Drilling Contractor Maine Test Borings

Project Location Kittery, Maine

Drill Rig Type Mobile Drill B-53 Truck-Mounted Drill Rig

File Number 38239-100

Client Maine Department of Transportation

Haley & Aldrich Rep. Marleigh Snow

Date 9/9/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-KDH-103	R1	5.2	6.2	22.4	21.4	slate	no test	15	12	100%	0	0%	3	<2 - 4		10	.02 - .1		12	-7	7	**40**
BB-KDH-103	R2	6.2	7.2	21.4	20.4	slate	no test	15	12	100%	0	0%	3	<2 - 4		10	.02 - .1		12	-7	7	**40**
BB-KDH-103	R3	7.2	10.0	20.4	17.6	slate	no test	15	34	101%	8	24%	3	<2 - 8		10	.004 - .1		12	-7	7	**40**
																					ave:	**40**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Condition of Joints

	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions

	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: **BB-KDH-103**

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. 45.7 | X G/S | ☐ Mudline | Core Barrel Type NQ | Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name U.S. Route 1 Bypass Bridge over Route 236

Drilling Contractor Maine Test Borings

Project Location Kittery, Maine

Drill Rig Type Mobile Drill B-53 Truck-Mounted Drill Rig

File Number 38239-100

Client Maine Department of Transportation

Haley & Aldrich Rep. Marleigh Snow

Date 9/9/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-KDH-106	R1	19.0	21.0	26.7	24.7	phyllite	no test	7	24	100%	0	0%	3	<2 - 12		10	.004 - .1		12	-7	7	**32**
BB-KDH-106	R2	21.0	24.4	24.7	21.3	phyllite	12,670 psi	7	34	83%	16	39%	8	<2 - 12		10	.004 - .1		12	-7	7	**37**
BB-KDH-106	R3	24.4	26.5	21.3	19.2	phyllite	no test	7	25	99%	8	32%	8	<2 - 12		10	.004 - .1		12	-7	7	**37**
BB-KDH-106	R4	26.5	29.1	19.2	16.6	phyllite	no test	7	27	87%	11	36%	8	<2 - 12		10	.004 - .1		12	-7	7	**37**
BB-KDH-106	R5	29.1	31.7	16.6	14.0	phyllite	14,797 psi	7	28	90%	24	77%	17	12 - 40		20	.02 - .1		12	-7	7	**56**
																					ave:	**40**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Condition of Joints	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: BB-KDH-106

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. _____ | X G/S | ☐ Mudline | Core Barrel Type NQ | Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name U.S. Route 1 Bypass Bridge over Route 236

Drilling Contractor Maine Test Borings

Project Location Kittery, Maine

Drill Rig Type Mobile Drill B-53 Bombardier-Mounted Drill Rig

File Number 38239-100

Client Maine Department of Transportation

Haley & Aldrich Rep. Marleigh Snow

Date 9/9/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-KDH-201	R1	12.4	17.1	24.8	20.1	phyllite	no test	7	56	99%	31	55%	13	<2 - 12		10	.004 - .1		12	-7	7	**42**
BB-KDH-202	R1	14.5	19.5	31.1	26.1	phyllite	no test	7	60	100%	51	85%	17	<2 - 40		10	.004 - .1		12	-7	7	**46**
BB-KDH-203	R1	17.0	22.0	29.2	24.2	phyllite	no test	7	60	100%	30	50%	13	<2 - 12		10	.004 - .1		12	-7	7	**42**
BB-KDH-204	R1	6.4	9.9	25.7	22.2	phyllite	no test	7	42	100%	6	14%	3	<2 - 12		5	.02 - .1		6	-7	7	**21**
BB-KDH-204	R2	9.9	10.9	22.2	21.2	phyllite	no test	7	12	100%	0	0%	3	<2		5	.02 - .1		6	-7	7	**21**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Condition of Joints	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: **BB-KDH-201, 202, 203, 204**

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR, as specified in Article 10.4.6.4. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3 f'_c$.

10.8.3.5.4c—Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0*B* is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5*B* (O'Neill and Reese, 1999):

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

- If the rock below the base of the shaft to a depth of 2.0*B* is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$q_p = \left[\sqrt{s} + \sqrt{(m\sqrt{s} + s)}\right] q_u \qquad (10.8.3.5.4c\text{-}2)$$

where:

s, m = fractured rock mass parameters and are specified in Table 10.4.6.4-4

q_u = unconfined compressive strength of rock (ksf)

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

For further information see O'Neill and Reese (1999).

Eq. 10.8.3.5.4c-2 is a lower bound solution for bearing resistance for a drilled shaft bearing on or socketed in a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate higher tip resistance than that calculated using Eq. 10.8.3.5.4c-2. Resistance factors for this method have not been developed and must therefore be estimated by the designer.

10.8.3.5.4d—Combined Side and Tip Resistance

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing, such as the methodology provided by O'Neill and Reese (1999), shall be used to estimate axial compressive resistance of shafts embedded in rock.

C10.8.3.5.4d

Typically, the axial compression load on a shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 0.4 in. occurs.

Designs which consider combined effects of side friction and end-bearing of a drilled shaft in rock require that side friction resistance and end bearing resistance be evaluated at a common value of axial displacement, since maximum values of side friction and end-bearing are not generally mobilized at the same displacement.

Where combined side friction and end-bearing in rock is considered, the designer needs to evaluate whether a significant reduction in side resistance will occur after the peak side resistance is mobilized. As indicated in Figure C10.8.3.5.4d-1, when the rock is brittle in shear, much shaft resistance will be lost as vertical movement increases to the value required to develop the full value of q_p. If the rock is ductile in shear, i.e., deflection softening does not occur, then the side resistance and end-bearing resistance can be added together directly. If the rock is brittle, however, adding them directly may be unconservative. Load testing or laboratory shear strength testing, e.g., direct shear testing, may be used to evaluate whether the rock is brittle or ductile in shear.

Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polyminerallic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite* E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	*m* *s*	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	*m* *s*	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

HALEY & ALDRICH

CALCULATIONS

		File No.	38239-100
		Sheet	1 of 1
Client	Maine Department of Transportation	**Date**	11/22/2013
Project	U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine	**Computed By**	EFW
Subject	Wingwall Footing Bearing Resistance Evaluation	**Checked By**	WAC

BEARING RESISTANCE EVALUATION:

SERVICE LIMIT STATE:

Determine service limit state bearing resistance based on presumptive values provided in Table C10.6.2.6.1-1 (attached).

For gravel, gravel-sand mixture, medium dense to dense values range from 8 to 14 kips per square foot (ksf).
For coarse to medium sand, medium dense to dense values range from 4 to 8 kips per square foot (ksf).

For service limit state design use the following: 8 ksf

STRENGTH LIMIT STATE:

Refer to attached table for calculation of strength limit state.

For walls with B up to 8 ft, qn = 18 to 20 ksf use: 18 ksf
For walls with B up to 14 ft, qn = 22 to 25 ksf use: 22 ksf

Resistance factor for bearing resistance of footings on rock, ϕ = 0.45

Calculate factored bearing resistance = 8 ksf for walls with B up to 8 ft
10 ksf for walls with B up to 14 ft

EXTREME EVENT LIMIT STATE:

Resistance factor for the extreme event, ϕ = 1

Calculate factored bearing resistance = 18 ksf for walls with B up to 8 ft
22 ksf for walls with B up to 14 ft

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR, as specified in Article 10.4.6.4. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

Project: U.S. Route 1 Bypass Bridge over Route 236, Kittery, Maine
Client: McFarland-Johnson
Subject: Bearing Resistance of Wingwalls
File Number: 38239-100
Date: 11/22/13
Prepared by: EFW
Checked by: WAC

Case	ϕ	γ_t (pcf)	c (psf)	D_f (ft)	D_w, (ft)	B	Eccentricity (Applies to B only) =	B'	L	L'	$N_\phi = f(\phi)$	$N_c = f_1(\phi)$	$N_q = f_2(\phi)$	$N\gamma = f_3(\phi)$	s_c	s_q	s_γ	Depth Correction, Y or N?	d_q	C_{wq}	$C_{w\gamma}$	N_{cm}	N_{qm}	$N_{\gamma m}$	q_n (psf)	q_n (ksf)	RF	RF×q_n (ksf)
1	34	125	0	2	10	6	1.0	4.0	25	25	3.54	42.1	29.4	41.0	1.11	1.11	0.94	N	1.00	1.00	1.00	46.80	32.57	38.38	17,737	17.7	0.45	8.0
2	34	125	0	2	10	8	1.3	5.3	25	25	3.54	42.1	29.4	41.0	1.15	1.14	0.91	N	1.00	1.00	1.00	48.37	33.63	37.50	20,908	20.9	0.45	9.4
2	34	125	0	2	10	10	1.7	6.7	25	25	3.54	42.1	29.4	41.0	1.19	1.18	0.89	N	1.00	1.00	0.90	49.94	34.69	36.63	22,407	22.4	0.45	10.1
3	34	125	0	2	10	12	2.0	8.0	25	25	3.54	42.1	29.4	41.0	1.22	1.22	0.87	N	1.00	1.00	0.83	51.51	35.75	35.75	23,833	23.8	0.45	10.7
4	34	125	0	2	10	14	2.3	9.3	25	25	3.54	42.1	29.4	41.0	1.26	1.25	0.85	N	1.00	1.00	0.79	53.08	36.80	34.88	25,186	25.2	0.45	11.3

Notes:
1. Assumes maximum eccentricity less than B/6.

G:\PROJECTS\38239 - Kittery Overpass Bridge\100\Calculations\[2013-1122-HAI-Single Layer Bearing Capacity.xls]Original Calculations

Lateral Earth Pressures

HALEY & ALDRICH	CALCULATIONS	File No.	38239-100
		Sheet	1 of 3
Client	Maine Department of Transportation	Date	9/27/2013
Project	U.S. Route 1 Bypass Bridge over Route 236 - Kittery, Maine	Computed By	EFW
Subject	Lateral Earth Pressures on Abutments and Wingwalls	Checked By	EK

OBJECTIVE:

Caculate static and dynamic lateral earth pressures for bridge abutments and wingwalls in accordance with AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 with 2012 Interim Revisions.

EVALUATION:

Static Lateral Earth Pressure:

Calculate static lateral earth pressure in accordance with Sections 3.11.5.1 and 3.11.5.3.

3.11.5—Earth Pressure: *EH*

3.11.5.1—Lateral Earth Pressure

Lateral earth pressure shall be assumed to be linearly proportional to the depth of earth and taken as:

$$p = k\gamma_s z \qquad (3.11.5.1\text{-}1)$$

where:

p = lateral earth pressure (ksf)

k = coefficient of lateral earth pressure taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions, or k_p, specified in Article 3.11.5.4, for walls that deflect or move sufficiently to reach a passive condition

γ_s = unit weight of soil (kcf)

z = depth below the surface of earth (ft)

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma[\sin^2\theta \sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

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

β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)

θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)

ϕ'_f = effective angle of internal friction (degrees)

Summarize input parameters….

δ =	26	deg.	0.45	radians	
β =	0	deg.	0.00	radians	
θ =	90	deg.	1.57	radians	
$\phi`_f$ =	32	deg.	0.56	radians	(value is assumed for granular borrow used to construct approach embankments)

Calculate coefficient Γ…. Γ = 2.91 Calculate coefficient of active lateral earth pressure…. k_a = 0.275

Assume total unit weight of abutment backfill…. γ_s = 0.125 kips per cubic foot (kcf)

Summarize abutment heights from 50 percent design drawings…. $z_{abut\ 1}$ = 23 ft $z_{abut\ 2}$ = 24 ft

Calculate static lateral earth pressure at base of abutment wall…. $p_{abut\ 1}$ = 789 psf $p_{abut\ 2}$ = 824 psf

presented in terms of force per unit width of wall = 9,078 lb/lf 9,885 lb/lf

The static lateral earth pressure should be applied as a triangular distribution, linearly increasing from zero at the top of the wall to the bottom. The resultant load acts at a height of H/3 above the base of the wall, where H is the total wall height.

EVALUATION: (CONT.)

Dynamic Lateral Earth Pressure:

Calculate dynamic lateral earth pressure in accordance with Sections A11.3.1 and A11.5.1.

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - B)}{\cos\theta_{MO}\cos^2\beta\cos(\delta+\beta+\theta_{MO})} \times \left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_{MO}-i)}{\cos(\delta+\beta+\theta_{MO})\cos(i-\beta)}}\right]^{-2}$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1 - k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

$$k_h = 0.74\,A_S\left(\frac{A_S}{d}\right)^{0.25}$$

where:

A_S = earthquake ground acceleration coefficient as specified in Eq. 3.10.4.2-2 (dim.)
k_h = horizontal seismic acceleration coefficient (dim.)
d = lateral wall displacement (in.)

From "Seismic Site Class & Design Parameters" calculation package the site is categorized as Site Class E.
Summarize relevant design parameters and calculate earthquake ground motion acceleration coeff. and horizontal seismic acceleration coeff….

F_{pga} = 1.20 Calculate A_s as the product of F_{pga} and PGA…. A_s = 0.119
PGA = 0.099

Assume a lateral wall displacement as follows…. d = 1.0 in.

Calculate horizontal seismic acceleration coefficient, k_h = 0.052

Summarize remaining input parameters….

k_v = 0 (assumed, typically equal to zero)
i = 0 deg. 0 radians

Calculate parameter θ_{MO}…. 2.955 degrees 0.052 radians

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Client	Maine Department of Transportation	Date	9/27/2013
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Subject	Lateral Earth Pressures on Abutments and Wingwalls	Checked By	EK

EVALUATION: (CONT.)

Dynamic Lateral Earth Pressure: (cont.)

Summarize all input parameters for calculation of seismic active earth pressure coefficient, K_{AE}….

δ =	0.454	radians	$\cos(\phi - \theta_{MO} - \beta)^2$ =	0.764
β =	0.000	radians	$\cos(\theta_{MO})$ =	0.999
$\phi`_f$ =	0.559	radians	$\cos(\beta)^2$ =	1.000
i =	0.000	radians	$\cos(\delta + \beta + \theta_{MO})$ =	0.875
θ_{MO} =	0.052	radians	$\sin(\phi + \delta)$ =	0.848
			$\sin(\phi - \theta_{MO} - i)$ =	0.486
			$\cos(i - \beta)$ =	1.000

Calculate K_{AE} = 0.308

Calculate the dynamic lateral earth pressure at the base of the abutment walls….

$$P_{AE} = 0.5\,\gamma\,h^2\,K_{AE} \qquad (11.6.5.3\text{-}2)$$

Assume total unit weight of abutment backfill…. γ_s = 0.125 kips per cubic foot (kcf)

Summarize abutment heights from 50 percent design drawings…. $z_{abut\,1}$ = 23 ft $z_{abut\,2}$ = 24 ft

k_v = 0 (assumed, typically equal to zero)

$P_{AE(1)}$ = 10,173 lb/lf $P_{AE(2)}$ = 11,077 lb/lf

442 psf 462 psf

Note: P_{AE} represents the total static and seismic active earth load per unit width of wall. This can be applied as a uniform pressure.

Live Load Surcharge:

Calculate the live load surcharge on abutments and wingwalls in accordance with Section 3.11.6.4.

$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Coefficient of active lateral earth pressure…. k_a = 0.275

Assume total unit weight of abutment backfill…. γ_s = 0.125

Since both abutments are greater than 20 ft, h_{eq} = 2 ft

Calculate Δ_p = 0.0688 ksf

69 psf

The pressure should be applied uniformly distributed over the entire height of the abutments/wingwalls.