



PRELIMINARY GEOTECHNICAL INVESTIGATION

Clark Bridge Replacement
Route 102
Tremont, Maine
Maine DOT WIN 19305.00

REPORT

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December 12, 2012

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Timothy S. Bryant, P.E.
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**RE: PRELIMINARY GEOTECHNICAL INVESTIGATION
CLARK BRIDGE REPLACEMENT
MAINE ROUTE 102
TREMONT, MAINE
MAINE DOT WIN 19305.00**

Dear Mr. Bryant:

Golder Associates Inc. (Golder) is pleased to submit this Preliminary Geotechnical Investigation report to Vanasse Hangen Brustlin, Inc. (VHB) for the proposed Clark Bridge replacement carrying Maine Route 102 over Marshall Brook and abutting approach causeway improvements in Tremont, Maine. The report presents the findings from our field investigation and laboratory testing programs, and summarizes our preliminary geotechnical considerations for design in support of VHB's Preliminary Design Report to be submitted to the Maine Department of Transportation. Our work was completed in accordance with our Project Contract with VHB executed April 25, 2012, and our proposal dated March 8, 2012.

It has been a pleasure working with VHB on this project and we are looking forward to providing our services to VHB through final design. Please contact us if you have any questions concerning our report or require additional geotechnical information.

Sincerely,

GOLDER ASSOCIATES INC.

A handwritten signature in blue ink, appearing to read "Jay R. Smerekanicz".

Jay R. Smerekanicz, P.G.
Senior Consultant and Associate

A handwritten signature in blue ink, appearing to read "Mark S. Peterson".

Mark S. Peterson, P.E.
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Table of Contents

1.0	INTRODUCTION.....	1
1.1	Background.....	1
1.2	Site and Project Description	2
2.0	geologic setting	3
3.0	preliminary SUBSURFACE investigation	4
3.1	Subsurface Explorations.....	4
3.2	Bedrock Mapping.....	5
3.3	Geophysical Investigation.....	6
4.0	LABORATORY TESTING	7
5.0	GENERAL SUBSURFACE CONDITIONS.....	8
5.1	Regional Geology	8
5.2	Soil Conditions.....	8
5.3	Bedrock.....	10
5.4	Groundwater	11
5.5	Geophysical Survey Results/Interpretation	11
6.0	REPLACEMENT BRIDGE FOUNDATION CONSIDERATIONS.....	13
7.0	CAUSEWAY WIDENING CONSIDERATIONS.....	14
8.0	CLOSURE	15

List of Tables

Table 1	Subsurface Exploration Locations
Table 2	Summary of Summary of Rock Core Quality
Table 3	Summary of Laboratory Soil Index and Classification Testing Results
Table 4	Summary of Rock Core Testing Results

List of Figures

Figure 1	Site Location Map
Figure 2	Subsurface Investigations Location Plan
Figure 3	Interpreted Subsurface Profile A-A'

List of Appendices

Appendix A	1924 Bridge Drawings
Appendix B	Boring Logs
Appendix C	Rock Core Photographs
Appendix D	Laboratory Test Results of Soil Samples
Appendix E	Laboratory Test Results of Rock Core Samples
Appendix F	Geophysical Survey Report
Appendix G	RMR and GSI Estimates



1.0 INTRODUCTION

This report summarizes the results of Golder Associates Inc.'s (Golder's) preliminary geotechnical investigation for replacement of the Clark Bridge carrying Maine Route 102 over Marshall Brook in Tremont, Maine (see Figure 1). The purpose of this report is to present soils and bedrock information at the bridge and causeway approach embankments obtained from subsurface investigations, geophysical investigations, and laboratory tests; discuss geotechnical considerations for the bridge foundations and causeway embankments; and present recommended geotechnical parameters for design and construction. Our work was completed in accordance with our original Project Contract executed April 25, 2012.

1.1 Background

The Maine Department of Transportation (MaineDOT) is planning to replace the existing Clark Bridge carrying Maine Route 102 over Marshall Brook in Tremont, Maine, and widen the existing approach causeways to accommodate a pedestrian walkway. Clark Bridge was constructed in 1924 to replace two single span bridges, and is a 15-foot (ft) clear span cast-in-place concrete superstructure supported on dry-laid stone masonry abutments. The bridge has a clear roadway width of 21 ft. Two causeways form the west and east bridge approaches, and are about 20 and 100 ft long, respectively, as shown on Figure 2. According to the 1924 design drawings (see Appendix A), the causeways were to be constructed of timber cribbing and stone fill, and the abutments and wingwalls were to be constructed of dry laid stone blocks. The 1924 design drawings also show a timber span bridge about 18 ft long within the southeast causeway. Overhead utilities exist on both sides of the roadway at the bridge approaches, and a permanent United States Geological Survey (USGS) gauging station is located at the northwest corner of the bridge. The bridge is subject to rapid, supercritical flows during ebb and flood tides, and a hydraulic evaluation is required to assess the existing flow constrictions and hydraulic performance of replacement alternatives with larger or multiple openings. VHB has subcontracted Northstar Hydro, Inc. of Winthrop, Maine to conduct the hydrologic, hydraulic and scour analyses for the project.

According to MaineDOT¹ the concrete superstructure is in fair condition with extensive map cracking and efflorescence visible on the underside. Map cracking, also called pattern cracking, is often caused by surface drying shrinkage, restrained by underlying (or overlying in the case of a bridge deck) concrete that shrinks less; expansion due to alkali-silica reaction; and restrained thermal contraction. MaineDOT also concluded the stone masonry abutments are in poor condition, with evidence of scour undermining portions of the abutments, resulting in movement of the stones in some courses of the abutment masonry.

¹ Maine Department of Transportation, June 1, 2011. Structure Inventory and Appraisal Sheet (English Units), Bridge No. 3663, State Route 102, Tremont, Maine. Maintenance and Operations, Bridge Maintenance, 1 p.



1.2 Site and Project Description

The site lies within the town of Tremont, located in the southwest portion of Mount Desert Island, Maine. The roadway across the causeway and bridge is narrow, about 21 (on bridge) to 24 ft (on causeway) inclusive of both lanes and thin shoulders, and the total length inclusive of both causeways and the bridge is about 220 ft. The inboard and outboard slopes of the causeways are graded at about 1.75 horizontal to 1 vertical (1.75H:1V) grades, and are armored with rip rap stone typically varying from about 6 to 12 inches in size. The causeway crest and bridge grade lie about 6 to 7 ft above mean water elevation as surveyed by VHB, and have an elevation of about 11 to 12 ft, NAVD88 datum. Right-of-way is limited to about 33 ft on both sides of the roadway centerline (i.e., 66 ft in total width). Adjacent landowners include Acadia National Park on both sides of the causeway, and the Town of Tremont elementary school northwest of the causeway. Marshall Brook flows to the southwest, draining Bass Harbor Marsh into Bass Harbor. The brook is tidally influenced, and the mean water elevation is approximately +5.5 ft mean sea level (msl). The tidal variation is about 10 to 12 ft². We observed that during low tide, the water level at the bridge and causeway is governed by the water level draining from Marshall Brook under the bridge. During high tide, the water level at the bridge is governed by the tide level, as the flow reverses and is directed upstream.

The replacement bridge is currently planned to be either a 30-ft simple span bridge or a 50- to 60-ft span bridge. Bridge replacement alternatives planned to be evaluated by VHB include: concrete rigid frame; semi-integral or integral, precast-prestressed beam bridge; and single or multiple cell concrete box culverts. We understand the new construction is planned to include 2 ft shoulders and a 5 ft wide pedestrian walkway to the southwest side of the causeway and bridge. Accordingly, the replacement bridge is planned to be about 31 to 32 ft wide and the causeway will be widened about 7 to 8 ft to the southwest. To stay within the existing right-of-way the causeway fill slope on the sidewalk side (southwest) needs to be steeply sloped at about 1.25H:1V or steeper. The current project schedule includes completion of the Preliminary Design Report (PDR) in November 2012, PS&E completion in January 2014, construction award in March 2014, and end of construction by September 2014.

² <http://freetidetables.com/sid/3320c7e4/tf/ttp/fmt/ttb>, accessed July 17, 2012.



2.0 GEOLOGIC SETTING

Regional surficial geologic mapping indicates the surficial soils are Pleistocene-aged emerged marine sediments deposited in environment when relative sea level was higher than present³. The surficial soils mapped on the west side of the brook consist of fine-grained sediments consisting of silt, clay and sand laid down in quite-water environments on the sea floor. The surficial soils mapped on the east side of the brook consist of undifferentiated sediments, comprised of marine sediments that have been mixed by wave action and thus are difficult to distinguish as either coarse- or fine-grained.

Regional bedrock geologic mapping indicates the bedrock consists of light-colored volcanic tuffs and metamorphosed siltstones and slates of the Silurian aged Cranberry Island Volcanic Series^{4,5}. The bedrock beneath the site is mapped as the upper member of the series, consisting of brown felsites and gray to brown dacite flows. The felsites have been mapped as a uniform, pink, very fine- to fine-grained granite.⁶

³ Lowel, T.V. and Borns, H.W.Jr., 1988. Surficial Geology of Mount Desert Island – A Visitor's Guide to the Geology of Acadia National Park, Maine Geological Survey, Bulletin 38, map scale 1:50,000.

⁴ Gilman, R.A., Chapman, C.A., Lowel, T.V. and Borns, H.W.Jr., 1988. The Geology of Mount Desert Island – A Visitor's Guide to the Geology of Acadia National Park. Maine Geological Survey, Bulletin 38, 50 p.

⁵ Graham, J., 2010. Acadia National Park: Geologic Resources Inventory Report. Natural Resource Report NPS/NRPC/GRD/NRR-2010/232, National Park Service, Ft. Collins, Colorado, 63 p.

⁶ Lubick, N., 1994. The Silurian (?) Cranberry Island Volcanic Series on Mt. Desert Island, Maine. In Woodward, H.H., ed., The Keck Geology Consortium, 7th Keck Symposium Volume, Beloit College, p. 25-28.



3.0 PRELIMINARY SUBSURFACE INVESTIGATION

Golder's field investigation comprised a three-phased investigation, consisting of a subsurface geotechnical borehole investigation program (including laboratory testing); a geophysical survey investigation program; and a bedrock mapping program. Prior to conducting the field work, we compiled our site-specific "Health and Safety Plan" (HASP) for the field program based on our understanding of the potential risks to which field personnel may be exposed while performing work in the project area. During the borehole investigation program, a Golder field geotechnical engineer conducted surface reconnaissance, monitored the drilling operations on a full-time basis, selected sample locations, and logged the subsurface conditions encountered. During the geophysical investigation, a Golder senior geologist observed and assisted the geophysical subconsultant during the surveys. For both the borehole and geophysical surveys, the drilling subcontractor provided temporary lane closures to facilitate room to conduct the work. Golder mapped bedrock outcrops visible in the vicinity of the bridge. The field program was completed in accordance with "Level D" personal protection. The remainder of this section provides a summary of the field investigations for these programs.

3.1 Subsurface Explorations

The subsurface exploration program included drilling six test borings and making a series of shallow hand rod probes at the locations shown on Figure 2. The borings were drilled between May 14 and 16, 2012 by Maine Test Borings, Inc. (MTB), of Hermon, Maine using a Mobile B-53 truck-mounted rotary drill rig. BB-TMB-101, BB-TMB-102 and BB-TMB-103 were advanced with solid stem augers to a depth of approximately 3 feet before switching to HW spun casing to the bedrock surface and then rock coring with a double-tube core barrel. Total boring depths ranged from 21.2 to 30.0 ft below ground surface (bgs) at these three borings. BB-TMB-104, BB-TMB-105 and BB-TMB-106 were advanced to depths of 6.6 to 12.1 ft bgs with solid stem augers to approximately 5 ft and hollow stem augers to refusal with no rock coring. At the completion of each boring, MTB removed the drilling tools and allowed the borehole to partially collapse before backfilling the remaining open borehole with drill cuttings. The top 12 inches of each borehole was filled with cold patch asphalt.

Standard Penetration Test (SPT) sampling was conducted in all six borings at intervals varying from continuous to 5 ft. Three inch inner diameter (ID) split spoons were used to increase the soil recovery and attempt to sample rock fill encountered underneath the roadway. Standard 1-3/8-inch ID split spoons were driven 24 inches by a 140 pound hammer dropped 30 inches using a rope and cathead. Three inch split spoons were driven 24 inches by a 300 pound hammer dropped 16 inches using a hydraulic safety driver. Golder recorded the number of hammer blows required to advance the sampler at 6-inch increments, lithologic descriptions, sample recovery lengths, and groundwater conditions. Drill behavior during casing and auger advancement and cuttings observed during drilling were also recorded. Golder collected soil samples from each SPT split spoon for visual identification and laboratory testing. After



reaching bedrock refusal in BB-TMB-101, -102 and -103, MTB obtained 9.6 to 10 feet of NQ sized bedrock core from these three borings. Groundwater measurements were taken within the borehole casing at the time of drilling.

In June 2012 VHB completed a survey of the as-drilled locations and ground surface elevations for the borings. The locations are reported as northing and easting coordinates (in feet) relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 East Zone. Ground surface elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88). Table 1 includes a summary of the boring information including as-drilled elevations, station, offsets, coordinates, and boring depth. Station and offsets provided in Table 1 are from the centerline of Route 102.

Details of the sampling methods used, field data obtained, and soil, bedrock and groundwater conditions encountered are presented on the boring logs included in Appendix B. A hammer efficiency factor of 0.60 was used for the rope and cathead method in determining the N_{60} corrections⁷ shown on the logs. Soils were field classified in general accordance with ASTM D 2488 and Modified Burmister descriptions. A description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs presented in Appendix B. Photographs of the rock core are included in Appendix C. Table 2 summarizes rock core descriptions and quality assessments.

3.2 Bedrock Mapping

Golder performed bedrock mapping on May 17, 2012 to provide additional information concerning bedrock structure and quality. Three exposures of bedrock located on the southeast side of Marshall Brook were mapped for the program (one downstream of the bridge and two upstream). At each outcrop we described the lithology, evaluated the discontinuities, and collected representative rock samples. The discontinuities consisted of joints, foliation planes and mineral veins. Data gathered at each outcrop station are summarized in Appendix G.

In general, for a specific discontinuity to be noted and measured, it had to have a persistence greater than three feet. This criteria produced a bias toward open or potentially unstable joint conditions and was incorporated in the field measurements to avoid measuring insignificant discontinuities that would have little influence on kinematic stability calculations (e.g., tight foliation planes). The following notes and measurements were made for each discontinuity:

- type (i.e., joint, foliation or vein)
- orientation (dip and dip direction)
- strength
- surface roughness

⁷ Standard Penetration Test N-values (blows per foot) corrected for hammer energy to determine an equivalent N value for a hammer with 60% efficiency are designated as N_{60} values. N values discussed in the report generally refer to corrected N_{60} values.



- persistence (feet)
- aperture (inches)
- infilling material
- fracture shape
- water
- fracture spacing (feet)

Orientation measurements were obtained with a Brunton geologic compass, with a magnetic declination correction of 17 degrees west of true north, as indicated on regional United States Geological Survey topographic maps. A total of 35 discontinuities were measured at the outcrops.

3.3 Geophysical Investigation

Golder subcontracted Hager-Richter Geoscience Inc. (HR) to conduct geophysical surveys at the site. HR conducted the surveys on May 17, 2012 along roughly the 235 foot long section of Route 102 shown on Figure 2 including the bridge and adjacent causeway approaches. The survey consisted of seismic refraction to estimate subsurface geophysical properties and estimate depth to bedrock; and ground penetrating radar (GPR) to investigate potential voids and other buried structures within the causeways. Due to traffic concerns, the need for the seismic survey line to be straight, and the curve of the roadway, the seismic survey could only be conducted in the northeast side of the roadway, i.e., the west bound lane. As the GPR survey lines do not need to be exactly straight/perpendicular, HR conducted the GPR survey over the whole roadway width at a longitudinal spacing of 5-ft, and a perpendicular spacing of about 50-ft. Appendix F contains HR's geophysical survey report including a description of the survey methods performed, a presentation of findings, and an interpretation of results.



4.0 LABORATORY TESTING

Golder subcontracted GeoTesting Express (GTX) of Acton, Massachusetts to perform the laboratory soil and rock testing. Golder selected eight soil samples from the geotechnical borings for mechanical sieve gradation and moisture content analyses for all samples according to ASTM methods. The gradation and moisture content test results are summarized in Table 3 and laboratory data are presented in Appendix D. Golder also selected four rock core samples for point load testing and two rock core samples for unconfined compression testing in accordance with ASTM methods. The results of the rock core testing are presented in Table 4 and Appendix E. Golder used the results of gradation tests to classify the soils and estimate soil engineering properties. Golder used the results of the rock core tests to estimate bedrock properties (in conjunction with the results of the bedrock mapping program).



5.0 GENERAL SUBSURFACE CONDITIONS

5.1 Regional Geology

The site is located in the New Brunswick Highlands physiographic province of downeast Maine. Surficial materials of the area consist of unconsolidated sediments of glacial and nonglacial origin, including glacial till, and glaciofluvial, glaciolacustrine and marine clay deposits. Other deposits include marine sediments, beach, talus and wetland deposits. Some of these sediments occur above current sea level due to crustal rebound, the effect of the earth's crust rising slowly after deglaciation.

The northwest portion of the site area is underlain by emerged marine sediments mapped as Quaternary-age fine-grained sediments (blanket deposits of silt, clay and sand laid down in quiet water sea floor environments). The southeast portion of the site area is underlain by undifferentiated emerged marine sediments mapped as Quaternary-age marine sediments mixed by wave action, consisting of both coarse- and fine-grained sediments. The underlying bedrock consists of the upper member Silurian-Devonian Cranberry Island Series, consisting of light grained fine grained granitic rocks and dark gray lava flows. These two distinct lithologies exist at the site, as described below.

5.2 Soil Conditions

Soils encountered at the borings were found to generally include asphalt and fill materials placed during construction of the causeway and bridge abutments, and naturally occurring emerged marine sediments. The underlying bedrock surface is interpreted to range from about 5 to 20 ft below the road grade as shown on Figure 3 subsurface profile based on the test boring and geophysical data.

In general, our subsurface investigations determined that causeway fill materials are consistent with those shown in the 1924 design drawings. These drawings show distinct areas where timber cribbing piers separated with stone fill were intended to be constructed. Both stone fill and wood were obtained in samples collected during the drilling program, and linear features interpreted as wooden structures were found during the geophysical investigation. However the data generated from our investigation are not detailed enough to corroborate or confirm the extent of the materials depicted and thus intended in the 1924 design drawings. But our investigation does suggest that the bedrock surface beneath the bridge and causeways does not agree with that shown in the 1924 drawings, particularly a dip in the bedrock beneath the northwest abutment.

The following sections describe the encountered soil and bedrock layers in more detail.

Asphalt Pavement

Asphalt pavement ranging from 6 to 10 inches (0.5 to 0.8 ft) thick was encountered in the six borings. The asphalt pavement consists of two layers, and upper wearing coarse ranging from 4 to 7 inches (0.3 to 0.6 ft) thick; and an underlying base course ranging from 1 to 4 inches (0.1 to 0.3 ft) thick.



Causeway Subgrade Fill

Sand and gravel subgrade fill materials ranging in thickness from 1.7 to 4.8 ft were encountered directly beneath the asphalt pavement at the boring locations. The fill materials consist of light to dark brown, red-brown and light brown-gray, medium dense to dense, damp, gravelly, fine to coarse sand; sandy coarse gravel; and silty fine sand. The subgrade fill is thinner closer to the abutments (about 1.8 ft thick), and is thicker within the causeways (up to about 4.8 ft thick). Corrected SPT N-values⁸ in the fill materials ranged from 12 to 50+, with an average of 25, indicating a medium dense consistency; however, the high 50+ N-value is likely influenced by underlying rock fill and is therefore likely artificially high. Three grain size analyses of this layer indicated AASHTO classifications of A-4, A-1-a and A-1-b; and GM, SM and GW-GM under the Unified Soil Classification System (Unified). The measured water content of the tested samples ranged from about 5 to 22 percent.

Rock Fill

Rock fill material was encountered in five of the six borings (BB-TMB-106 being the exception). Rock fill ranged in thickness from 1.6 to 8.6 ft. The rock fill consists of boulders, cobbles, gray to dark green fine to coarse gravels, with a trace of silt and trace to some fine to coarse sand. Sample recovery, even with the 3-inch diameter split spoons, was poor. Cobbles up to 3 inches in diameter, and boulders up to 1 ft thick were encountered during drilling. N-values ranged from 18 to 76, with an average of 37, indicating a dense consistency; however, the high N-values are likely influenced by boulders and are therefore likely artificially high. From visual identification the material has a GM Unified classification. The borehole log description of the rock fill in boring BB-TMB-104 indicates the sand content of the auger cuttings generally decreased with depth. Wood, interpreted to be from the timber cribbing used to construct the causeway and bridge abutments, was encountered within the lower portion of the rock fill layer in boring BB-TMB-103.

Wood

Wood materials interpreted to be from timber cribbing were encountered in boring BB-TMB-103 from 9.0 to 10.4 ft bgs. The wood materials were returned to the surface while washing out the drill casing with a roller bit. The wood did not appear to be rotten. Interpreted wood structures detected during the geophysical survey are discussed in Section 5.5.

Emerged Marine Sediments

Emerged marine sediments were encountered in borings BB-TMB-103 and -106, beneath the rock fill and subgrade fill materials, respectively. These sediments range in thickness from 1.8 to 7.3 ft at BB-TMB-106 and 103, respectively, and consist of gray, blue-gray and dark gray, loose to very dense, fine to coarse gravel, fine sandy silt, and silt. N-values ranged from 7 to 110, with an average of 56, indicating a

⁸ All subsequent references to SPT N-values in this report pertain to the energy corrected N_{60} values shown on the boring logs in Appendix A.



very dense consistency. Three grain size analyses of this layer indicated AASHTO classifications of A-1-a, A-1-b and A-4; and GM, SM and ML Unified classifications. The measured water content of the tested samples ranged from about 11 to 47 percent. The emerged marine sediments beneath the current bridge consist of coarser denser materials (fine to coarse gravel), and the marine sediments beneath the southeast causeway consist of silts.

5.3 Bedrock

Bedrock was encountered in three borings as determined from rock core (BB-TMB-101, -102 and -103), and suspected from drill rig/solid stem auger refusal and extrapolated exposed bedrock in the other three borings. Bedrock surface depths are interpreted to be about 18.3 ft and 10.8 ft bgs at the west and east abutments of the existing bridge, respectively. Along the east causeway approach the bedrock surface rises to the east and varies from about 10.8 ft to 6.6 ft bgs. At the west causeway approach the bedrock surface appears to rise from the west bridge abutment from about 18.3 to about 12.1 ft bgs at boring BB-TMB-104 and to about 6.9 ft bgs at Sta. 15+00 based on the geophysical survey findings discussed in Section 5.5. Based on bedrock outcrops on the east side of Marshall Brook, and in the rock core, two lithologies are present beneath the site. Within boring BB-TMB-101 and in the bedrock exposures on the east side of Marshall Brook, a dark pinkish gray, fine to medium grained granite gneiss is present. This rock is strong (R4), moderately to slightly weathered and contains discontinuities close to closely spaced. Thin basalt dikes and quartz veins cut through the rock. Laboratory testing indicates this rock type has a compressive strength ranging from 26,800 to 37,000 pounds per square inch (psi).

Within borings BB-TMB-102 and -103, a dark gray to greenish black, very fine grained metavolcanic unit was encountered. This rock is medium weak to strong (R3 to R4), moderately to highly weathered, with close to very closely spaced discontinuities, with thin weak mineral coatings (e.g., chlorite and talc). Laboratory testing indicates this rock type has a compressive strength ranging from 11,900 to 18,800 psi.

The nature of the contact between these lithologies is unknown at this time, but may either be stratigraphic, dipping at a steep angle between borings BB-TMB-101 and -102, or due to a high angle fault, also located between these two borings. An unmapped fault may be the plausible explanation, providing a path of weakness in the bedrock for Marshall Brook to follow.

Golder used bedrock data collected from outcrop and rock core, and from laboratory testing to estimate the rock mass rating (RMR) and geologic strength index (GSI) for use in assessing bedrock bearing resistance for both bedrock lithologies encountered. The RMR for individual core runs is presented in Table 2. The rock core based RMR within the granite gneiss lithology, i.e., bedrock southeast of the southeast abutment ranges from 46 to 49 (average of 47.5). The rock core based RMR for individual core runs within the metavolcanics lithology, i.e., bedrock beneath both abutments, ranges from 13 to 31 (average of 23.6). The rock core/outcrop based RMR and GSI for the granite gneiss lithology are 50 and



45; the rock core/outcrop based RMR and GSI for the metavolcanics is 28 and 31. Appendix E contains the calculations for estimating these ratings.

5.4 Groundwater

Groundwater levels measured in open boreholes in the borings during drilling ranged from Elevation +1.6 to +3.0 ft, which generally corresponds to the surface water level in Marshall Brook. However, as Marshall Brook is tidally influenced, these groundwater levels likely fluctuate with a lag relative to the tides. Groundwater levels fluctuate due to other natural variations, such as seasonal/climatic influences, precipitation and temperature, and to other variations such as construction and groundwater pumping.

5.5 Geophysical Survey Results/Interpretation

A detailed discussion of the findings and interpretation of HR's geophysical survey is presented in their report included in Appendix F. The results of the seismic survey indicate undifferentiated seismic velocities of the causeway fill and rock fill materials and emerged marine sediments overlying bedrock range from 1,600 to 2,000 feet/second (ft/sec). Bedrock seismic velocities ranged from 12,500 ft/sec to 15,800 ft/sec. The seismic survey indicates bedrock depths range between 6.9 ft bgs beneath the northwest causeway, to 15.8 ft bgs beneath the northwest abutment, to 7.3 ft bgs beneath the southeast causeway. These results are generally consistent with the bedrock surface interpreted from the borehole investigation.

The results of the GPR survey indicate a "possible shallow buried structure" exists between Station (Sta) 16+37 and 16+55.5 within the southeast causeway, approximately where a former timber span bridge is shown on the 1924 design drawings. The GPR survey also detected three linear features near Sta 15+60, 16+22 and 16+63, interpreted to be edges of timber cribbing structures within or near the base of the rock fill. Comparison with the 1924 drawings indicates the linear features at Sta 15+60 and 16+22 correlate with a timber crib area and an edge of timber cribbing, respectively. The feature at Sta 16+65 occurs in an area where no timber cribbing is shown on the 1924 design drawings.

The interpretation of the possible buried structure is consistent with an apparent opening that may have been subsequently filled as shown in the 1924 drawing (see Appendix A). The drawing indicates a log pier crib was constructed northwest of the northwest bridge abutment to support the northwest causeway. The drawing also shows two crib piers separated by stone fill (about 45 ft long total) were placed southeast of the southeast abutment to support the northwest side of the southeast causeway. An open area about 18 ft long and supported by horizontal timber elements, presumably a previous inlet/outlet structure for the brook, was constructed southeast of the two crib pier structures. Southeast of the open area the drawing shows a field stone abutment and retaining walls. These structures appear to have been placed on bedrock and constructed to elevations above high tide.



The GPR also detected buried surfaces within the northwest and southeast ends of the causeways, possibly representing the road base materials overlying rock fill. HR interpreted reflections in the GPR data as surfaces between sand and gravel (i.e., causeway subgrade fill) overlying boulder and cobble fill (i.e., rock fill). The depth of these surfaces (where detected) ranged from about 3 to 4 ft bgs.



6.0 REPLACEMENT BRIDGE FOUNDATION CONSIDERATIONS

As indicated on the Figure 3 subsurface profile the bedrock surface is interpreted to be at, or in close proximity to, the base of new abutments for a 30 foot span or 60 ft span replacement bridge centered across the existing bridge location. Accordingly, technically feasible foundation systems for the replacement bridge are considered to be limited to full height cantilever-type abutments and wingwalls on spread footings bearing directly on bedrock or on a concrete seal, if required, bearing on bedrock. Conventional stub abutments on spread footings constructed on mechanically stabilized earth (MSE) wall-wrapped approach embankments are not considered feasible due to erosion risks of the MSE wall backfill materials. Pile supported abutments are not considered feasible due to scour, fixity and lateral resistance considerations.

The bedrock surface for the new east abutment is estimated to be at about Elev. 0 ft which is close to the streambed grade of the adjacent Marshall Book. If the new west abutment is located in the vicinity of boring BB-TMB-103 the bedrock surface is estimated to be at about Elev. -5 ft. Excavations in causeway fill materials at both abutments are expected to encounter sand and gravel subgrade fills, gravel and cobble rock fills and timber cribbing as described in Section 5.2. At the east abutment the bedrock surface is estimated to be at the base of the timber crib rock fill. At the west abutment about 7 ft of medium dense very dense fine to coarse gravel marine sediments will need to be removed below the base of the timber crib rock fill to expose the bedrock surface.

The results of the seismic survey indicate undifferentiated bedrock seismic velocities range from 12,500 ft/sec to 15,800 ft/sec. These relatively high velocities, coupled with the high compressive strengths determined from rock core laboratory testing indicate the bedrock is not rippable, i.e., cannot be excavated with construction equipment such as an excavator. If rock needs to be removed for foundations or other structures, close-in blasting or specialized non-blasting rock removal methods must be used.



7.0 CAUSEWAY WIDENING CONSIDERATIONS

The proposed widening of travel lanes and shoulders plus an addition of a 5 ft wide pedestrian walkway on the south side of the bridge and southern crest of the causeway on both sides of the bridge will require about a 7 to 8 ft widening of the causeway. The widening is currently planned to be made on the south side of the causeway; however, right-of-way limits require the widened fill slope to be very steep, i.e., 1.25H:1V or steeper. Feasible alternatives for widening the causeway crest are considered to include: a moment slab supported on stable subgrade; a geo-grid (or other material) reinforced soil slope; and a retaining wall structure with or without a sloping backfill surfaced with riprap extending up to finished road grade. A number of different wall types could be considered including precast wall systems (e.g., T-Wall and Doublewall), a cast-in-place cantilever wall, a geocell wall, and a gabion wall. The wall foundation would need to bear directly on the bedrock surface and the wall backfill materials would need to be designed to be free draining and stable from erosion under fluctuating tide cycles. Excavation requirements into the existing causeway fill materials for reinforced slope and retaining wall alternatives will need to be evaluated considering maintenance of traffic requirements. Design life requirements in the salt water environment would also need to be considered.



8.0 CLOSURE

The geotechnical information, test results and foundation considerations included in this report are preliminary and provided for the exclusive use of VHB for development of the Preliminary Design Report (PDR) for the replacement of the Clark Bridge in Tremont, Maine. The report was prepared in accordance with generally accepted soil and foundation engineering practices. In the event that any changes in the nature, design, or location of the proposed project are planned, Golder should be notified to review the appropriateness of our conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, our analyses, and recommendations are based in part on the subsurface explorations completed. Golder should be notified if actual conditions encountered vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report. Upon completion of the PDR and selection of a final bridge alignment, profile and span arrangement, geotechnical evaluations should be conducted to provide final geotechnical design recommendations.

The professional services provided by Golder for this project included only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

TABLES

Table 1: Subsurface Exploration Locations
Preliminary Geotechnical Data Report
Clark Bridge
Tremont, Maine
MaineDOT WIN: 19305.00

Test Boring Designation	Existing Ground Surface Elevation ¹ (ft-msl)	Boring Depth ² (ft)	As-Drilled Locations ¹		Comments
			Northing (ft)	Easting (ft)	
BB-TMB-101	11.2	21.2	153,296.02	2,172,345.66	Bedrock at Elev. 0.4 ft-msl
BB-TMB-102	11.4	21.5	153,303.70	2,172,331.97	Bedrock at Elev. 0.3 ft-msl
BB-TMB-103	11.9	30.0	153,317.75	2,172,305.90	Bedrock at Elev. -6.4 ft-msl
BB-TMB-104	12.3	12.1	153,318.89	2,172,279.54	Auger Refusal at Elev. 0.2 ft-msl
BB-TMB-105	10.5	6.6	153,261.04	2,172,384.42	Auger Refusal at Elev. 3.9 ft-msl
BB-TMB-106	10.5	7.5	153,247.51	2,172,407.71	Auger Refusal at Elev. 3.0 ft-msl

Notes:

- As-drilled locations and elevations surveyed by VHB. Coordinates reference the NAD83 (98) ME2000 West Zone coordinate system. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
- Depth below roadway surface.
- Boring logs presented in Appendix A.
- ft=feet; msl=mean sea level.

Prepared By: MRHChecked By: JRSReviewed By: MSP

**Table 2: Summary of Rock Core Quality
Preliminary Geotechnical Data Report
Clark Bridge
Tremont, Maine
MaineDOT WIN: 19305.00**

Test Boring Designation	Existing Ground Surface Elevation ¹ (ft-msl)	Run			TCR ²		RQD ³			Intact Rock				
		No.	Depth Below Ground Surface (ft)		Length (ft)	Length (ft)	Percent	Length (ft)	Percent	Designation	Weathering	Strength	Rock Mass Rating ⁴	Description
			Start	End										
BB-TMB-101	11.2	R1	12.0	17.0	5.0	5.0	100%	2.1	42%	Poor	Moderately Weathered to Slightly Weathered	R4	46	12.0 to 12.9 ft-bgs: Dark gray to greenish-black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular. 12.9 to 17.0 ft-bgs: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide.
		R2	17.0	21.2	4.2	4.2	100%	2.1	50%	Poor	Moderately Weathered to Slightly Weathered	R4	49	Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft-bgs., approximately 1 inch thick and subhorizontal.
BB-TMB-102	11.4	R1	11.5	16.5	5.0	4.2	83%	2.3	45%	Poor	Moderately Weathered to Slightly Weathered	R4	24	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
		R2	16.5	19.8	3.3	3.2	98%	1.1	33%	Poor	Moderately Weathered to Slightly Weathered	R4	22	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Calcite veins up to 1/4 inch thick common. Highly fractured zone 17.5 to 18 ft-bgs.
		R3	19.8	21.5	1.7	1.4	85%	1.1	65%	Fair	Moderately Weathered to Slightly Weathered	R4	28	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
BB-TMB-103	11.9	R1	20.0	25.0	5.0	3.0	60%	1.6	32%	Poor	Moderately Weathered to Slightly Weathered	R3	31	Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered, highly fractured, discontinuities close to very closely spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating.
		R2	25.0	30.0	5.0	2.3	45%	0.3	7%	Very Poor	Moderately Weathered to Slightly Weathered	R3	13	Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very closely spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs.

Notes:

- As-drilled elevations surveyed by VHB.
- TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
- RQD = rock quality designation. RQD is the total length of core pieces recovered with a length greater than or equal to twice the core diameter (4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length.
- Rock Mass Rating System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. For the rating adjustment for joint orientation, values of Fair (-7) were typically assigned for gently dipping, Fair to Unfavorable (-11) for moderately to steeply dipping, and Unfavorable (-15) for steeply dipping discontinuity angles observed in the core samples. Correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table. Reference Appendix C for the RMR calculations.
- ft=feet; msl=mean sea level

Prepared By: MRH
Checked By: JRS
Reviewed By: MSP

Table 3: Summary of Laboratory Soil Index and Classification Testing Results**Preliminary Geotechnical Data Report****Clark Bridge****Tremont, Maine****MaineDOT WIN: 19305.00**

Test Boring Designation ¹	Existing Ground Surface Elevation ² (ft-msl)	Sample Number	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation (ft-msl)	Sieve Minus No. 200 (%)	Natural Moisture Content (%)	USCS Soil Classification ³	AASHTO Soil Classification ³
BB-TMB-101	11.2	1D	0.5 to 2.5	10.7 to 8.7	16.9	6.7	GM	A-1-b
BB-TMB-103	11.9	3D	11 to 13	0.9 to -1.1	10.2	10.7	GM	A-1-a
		5D	15 to 15.5	-3.2 to -3.6	27.8	10.7	SM	A-2-4
BB-TMB-104	12.3	2D	3 to 5	9.3 to 7.3	36.0	21.9	SM	A-4
BB-TMB-105	10.5	1D	1 to 3	9.5 to 7.5	10.3	4.7	GW-GM	A-1-a
BB-TMB-106	10.5	3D	5 to 7	5.5 to 3.5	52.9	46.6	ML	A-4

Notes:

1. Test boring locations are shown on Figure 2, Boring Location Plan.
2. As-drilled locations and elevations surveyed by VHB.
Elevations reference the North American Vertical Datum of 1988 (NAVD88).
3. AASHTO and USCS symbols assigned based on field identification and interpretation of laboratory test results.
4. Laboratory test results for soil testing are provided in Appendix C.
5. Laboratory testing was performed by GeoTesting Express, Inc. of Acton, MA.
6. ft=feet; msl=mean sea level.

Prepared By: MRHChecked By: JRSReviewed By: MSP

Table 4: Summary of Laboratory Rock Core Testing Results
Preliminary Geotechnical Investigation
Clark Bridge
Tremont, Maine
MaineDOT Win: 19305.00

Test Boring Designation ¹	Existing Ground Surface Elevation ² (ft-msl)	Top of Bedrock Elevation ^{2,3} (ft-msl)	Sample Number	Approximate Sample Depth Below Top of Bedrock (ft)	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation (ft-msl)	Uniaxial Compressive Strength ⁴ (psi)	Point Load Index Test ⁵ (psi)
BB-TMB-101	11.2	0.4	R1	2.5 to 3.1	13.3 to 13.9	-2.1 to -2.7	-	26,800
			R2	7.4 to 7.8	18.2 to 18.6	-7.0 to -7.4	31,567	-
			R2	8.0 to 8.4	18.8 to 19.2	-7.6 to -8.0	-	37,000
BB-TMB-102	11.4	0.3	R2	8.1 to 8.5	19.2 to 19.6	-7.8 to -8.2	13,189	-
BB-TMB-103	11.9	-6.4	R1	2.5 to 2.9	20.8 to 21.2	-8.9 to -9.3	-	11,900
			R2	2.9 to 3.4	21.2 to 21.7	-9.3 to -9.8	-	18,800

Notes:

1. Test boring locations are shown on Figure 2, Boring Location Plan.
2. As-drilled locations and elevations surveyed by VHB. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
3. Top of bedrock refers to rock based on depth to casing refusal or observed rock surface in core barrel. Notes concerning weathered rock, if present, are provided on the boring logs.
4. Test performed based on ASTM D7012 - *Compressive Strength and Elastic Moduli on Intact Rock Core Specimens Under Varying States of Stress and Temperatures*.
5. Test performed based on ASTM D5731 - *Point Load Index*. Diametral Test Type. Value listed is the estimated compressive strength based on the point load index determined from the test.
6. Laboratory testing performed by GeoTesting Express, Inc.
7. Laboratory test results for rock core testing are provided in Appendix D.
8. ft = feet; psi = pounds per square inch; msl=mean sea level.

Prepared By: MRH
Checked By: JRS
Reviewed By: MSP

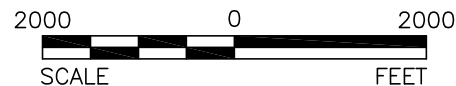
FIGURES

Drawing file: 12387440A001.dwg Dec 10, 2012 - 3:29pm



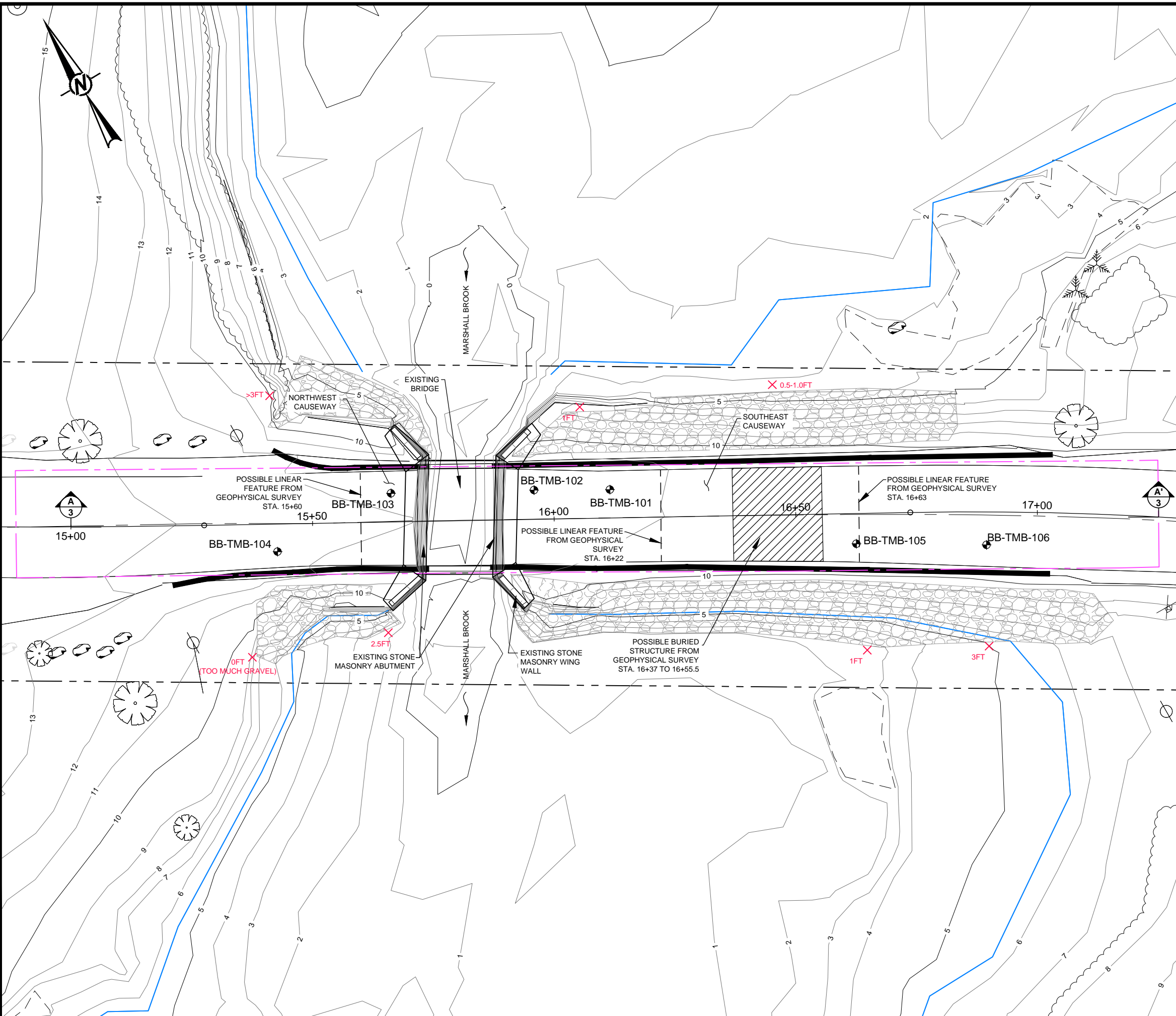
REFERENCES

BASEMAP TAKEN FROM USGS 7.5 MINUTE SERIES (TOPOGRAPHIC) QUADRANGLE MAP TITLED, "SOUTHWEST HARBOR, MAINE" AND "BASS HARBOR, MAINE" DATED 1983.



	SCALE	AS SHOWN	TITLE	<h2>SITE LOCATION MAP</h2>	
	DATE	12/10/12			
	DESIGN	JRS			
	CADD	MPB			
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PROJECT No.	123-87440	REV. 0	REVIEW	MSP	VHB CLARK BRIDGE REPLACEMENT TREMONT, MAINE
					FIGURE 1

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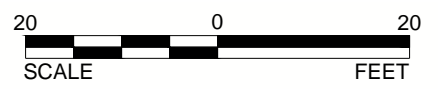


LEGEND

- 5 FT INDEX CONTOUR (FT MSL)
- 1 FT INDEX CONTOUR (FT MSL)
- EDGE OF DRAINAGE WATERWAY
- GUARDRAIL BEAM
- LEDGE OUTCROP
- RIGHT OF WAY
- STATIONING ALONG CENTERLINE
- GEOPHYSICAL SURVEY LOCATION
- TREE LINE
- BOULDER
- DECIDUOUS TREES
- BORING LOCATION AND DESIGNATION
- RIP RAP ARMOR
- CROSS SECTION LOCATION AND DIRECTION
- ROCK PROBE LOCATION (APPROXIMATE) AND REFUSAL DEPTH

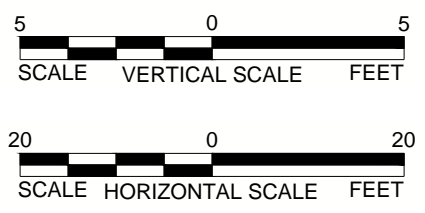
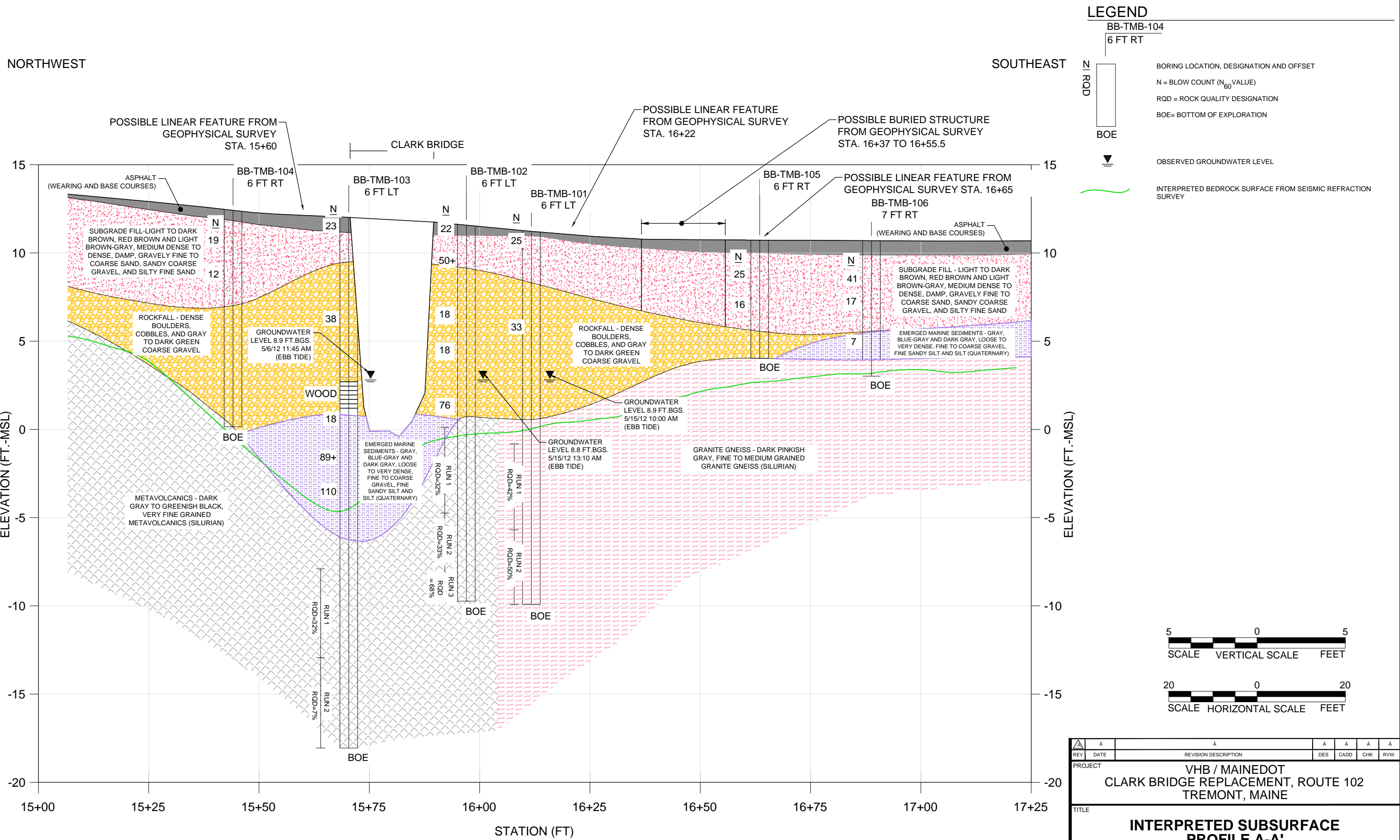
- ### REFERENCES & NOTES
- 1.) BASEMAP ELEMENTS FROM VHB, INC. DRAWING TITLED 3DPTOP_13MAR12.dwg. DATED MAY 10, 2012.
 - 2.) BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY MAINE TEST BORINGS, INC.
 - 3.) BASIS OF BEARING IS MAINE GRID NORTH. HORIZONTAL DATUM IS MAINE 2000 EAST ZONE NAD83 (1996).
 - 4.) ELEVATIONS ARE IN FEET AND ARE REFERENCED TO NAVD88 (GEOID03).
 - 5.) BRIDGE ABUTMENT AND WINGWALL EXTENTS FROM 1924 BRIDGE DRAWINGS (SEE APPENDIX A).

BORING ID	NORTHING (FT)	EASTING (FT)	ELEVATION (FT-MSL)
BB-TMB-101	153296.02	2172345.66	11.19
BB-TMB-102	153303.70	2172331.97	11.42
BB-TMB-103	153317.75	2172305.90	11.89
BB-TMB-104	153318.89	2172279.54	12.27
BB-TMB-105	153261.04	2172384.42	10.45
BB-TMB-106	153247.51	2172407.71	10.46



REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW
PROJECT						
VHB / MAINE DOT CLARK BRIDGE REPLACEMENT, ROUTE 102 TREMONT, MAINE						
TITLE						
SUBSURFACE EXPLORATION LOCATIONS						
PROJECT No.		123-87440	FILE No.		12387440A002	
DESIGN	JRS	12/07/12	SCALE		AS SHOWN	
CADD	MPB	12/07/12				
CHECK	JRS	12/07/12				
REVIEW	MSP	12/07/12				
FIGURE 2						

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REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT: **VHB / MAINEDOT
CLARK BRIDGE REPLACEMENT, ROUTE 102
TREMONT, MAINE**

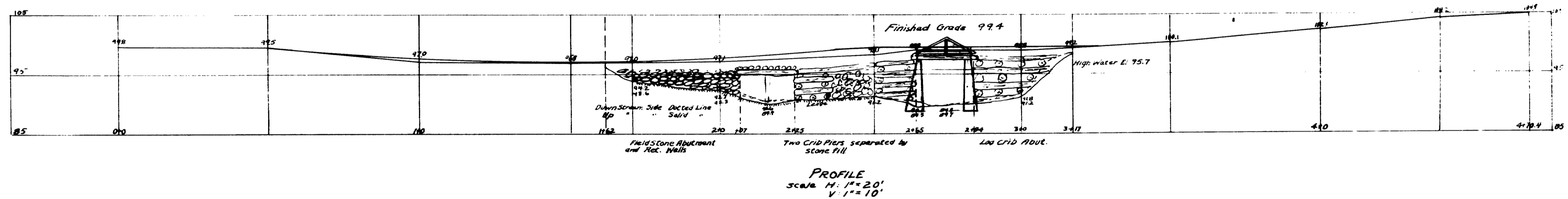
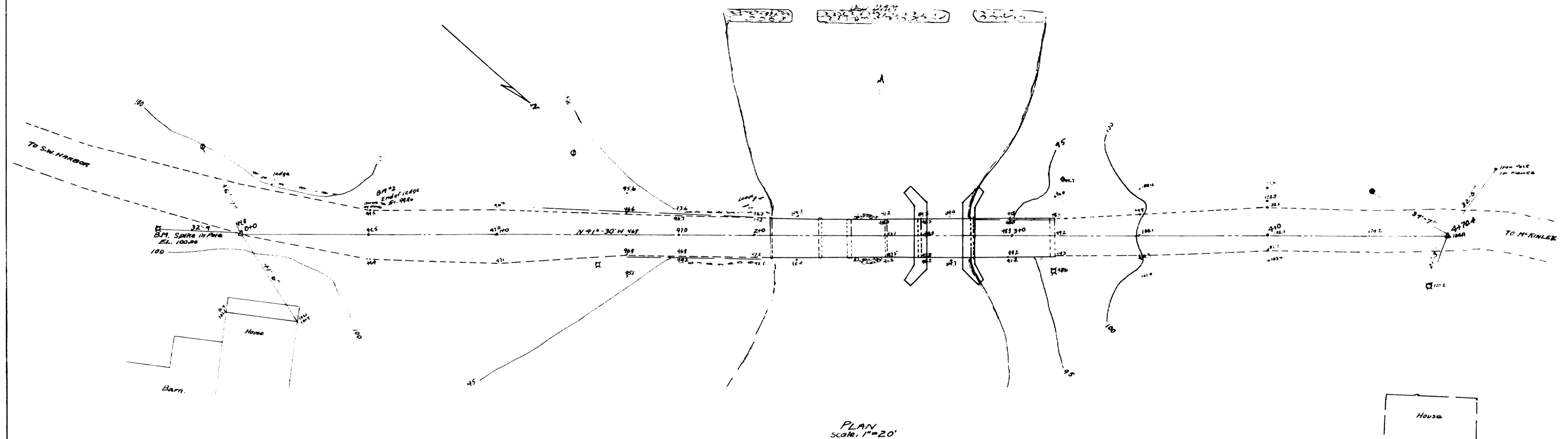
TITLE: **INTERPRETED SUBSURFACE
PROFILE A-A'**

PROJECT No.	123-87440	FILE No.	12387440A003
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CHECK	JRS 12/07/12		
REVIEW	MSP 12/07/12		

FIGURE 3

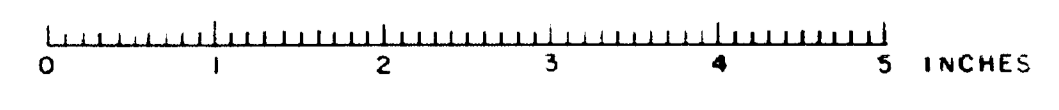


APPENDIX A
1924 BRIDGE DRAWINGS

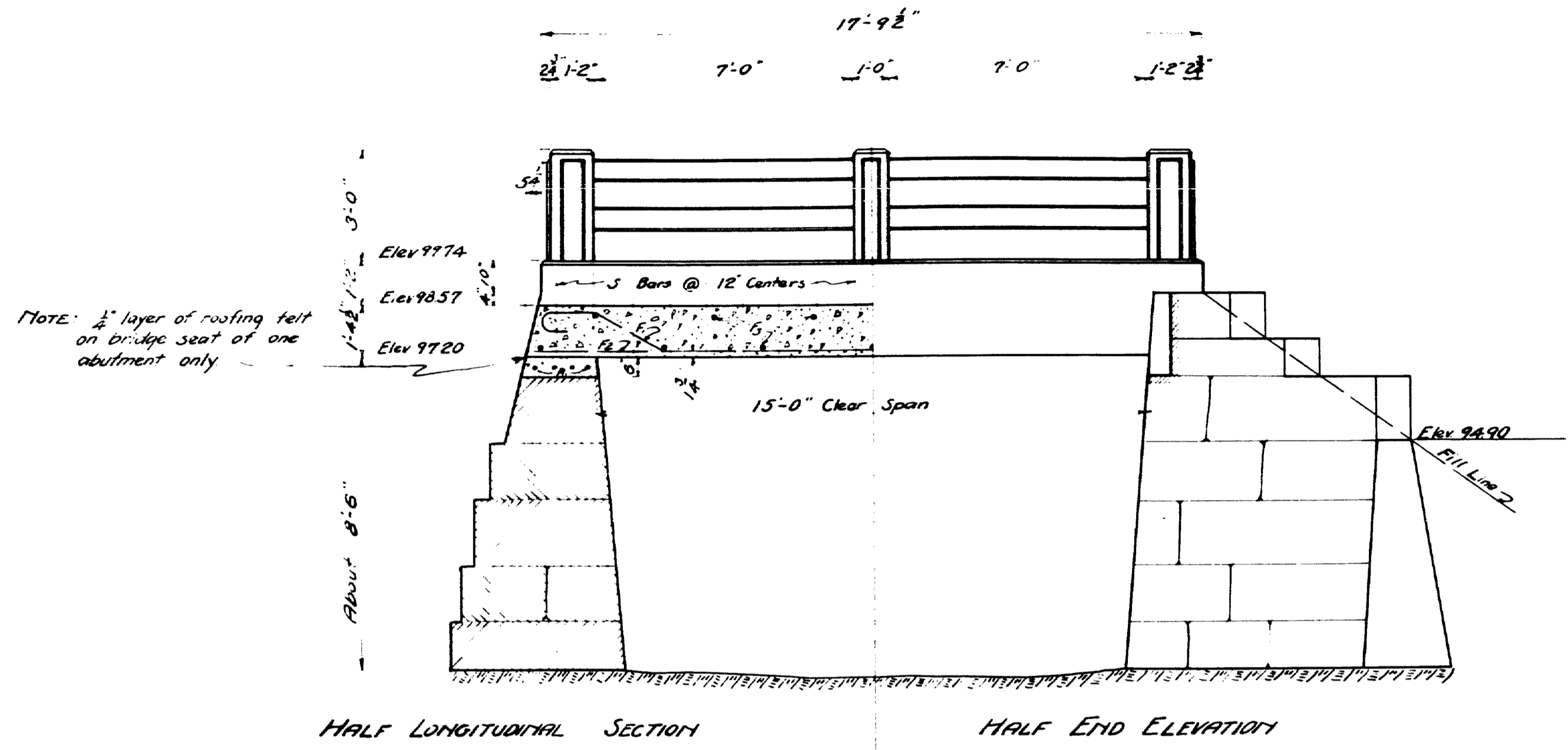


MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
CLARK BRIDGE
IN THE TOWN OF
TREMONT, HANCOCK Co.
SURVEY PLAN
SHEET 1
AUGUSTA ME. JUNE 4, 1924.

5-57

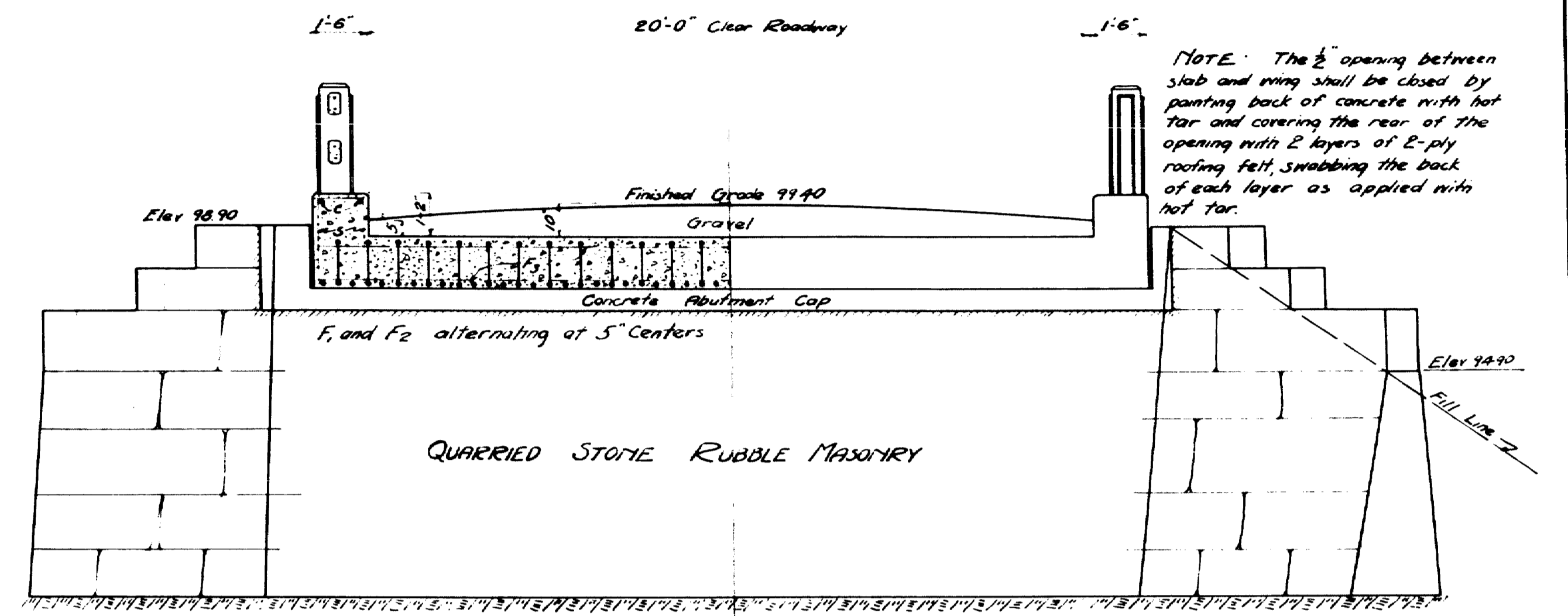


Surveyed by H. H. H. 5-1924
Designed by H. H. H.



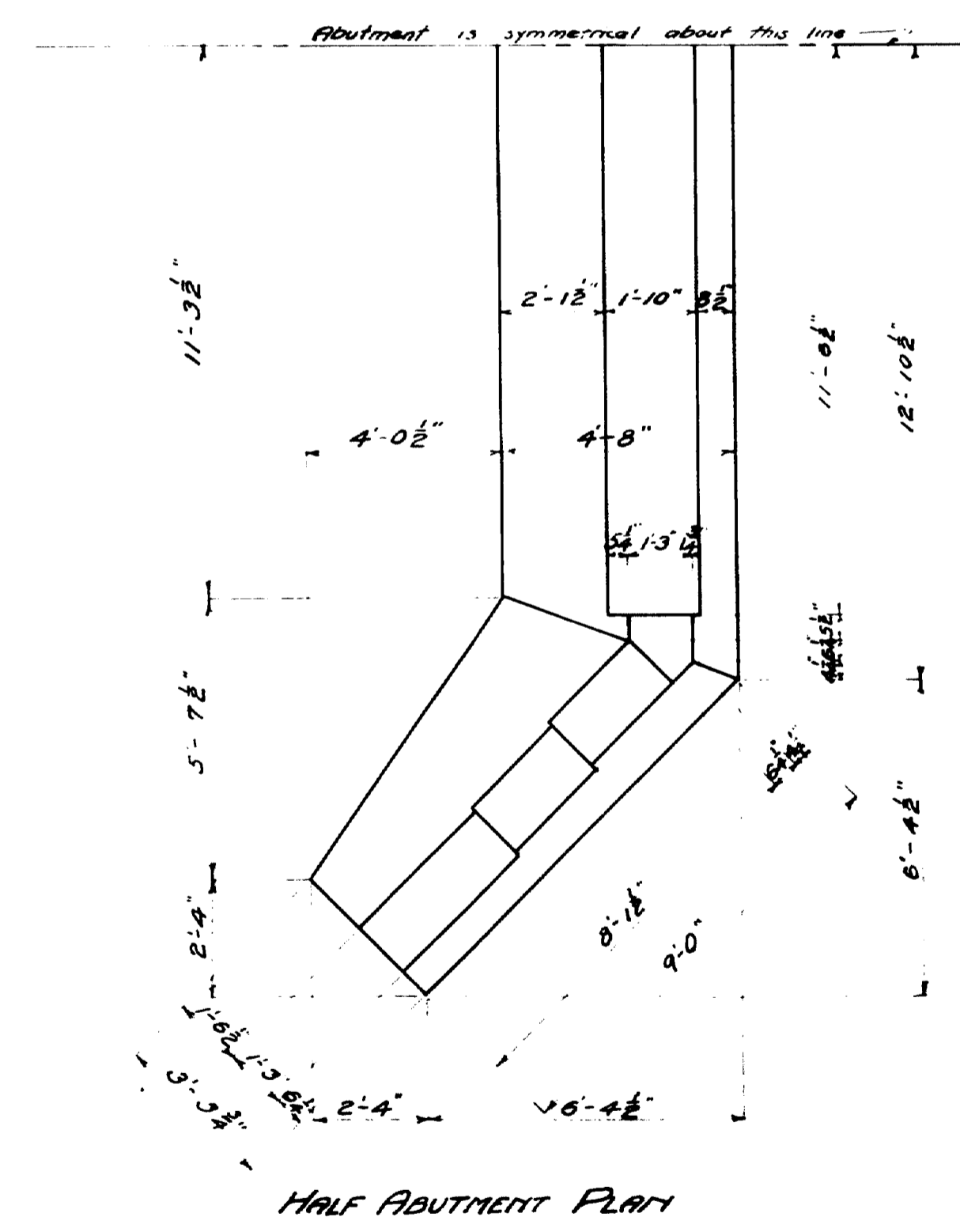
HALF LONGITUDINAL SECTION

HALF END ELEVATION



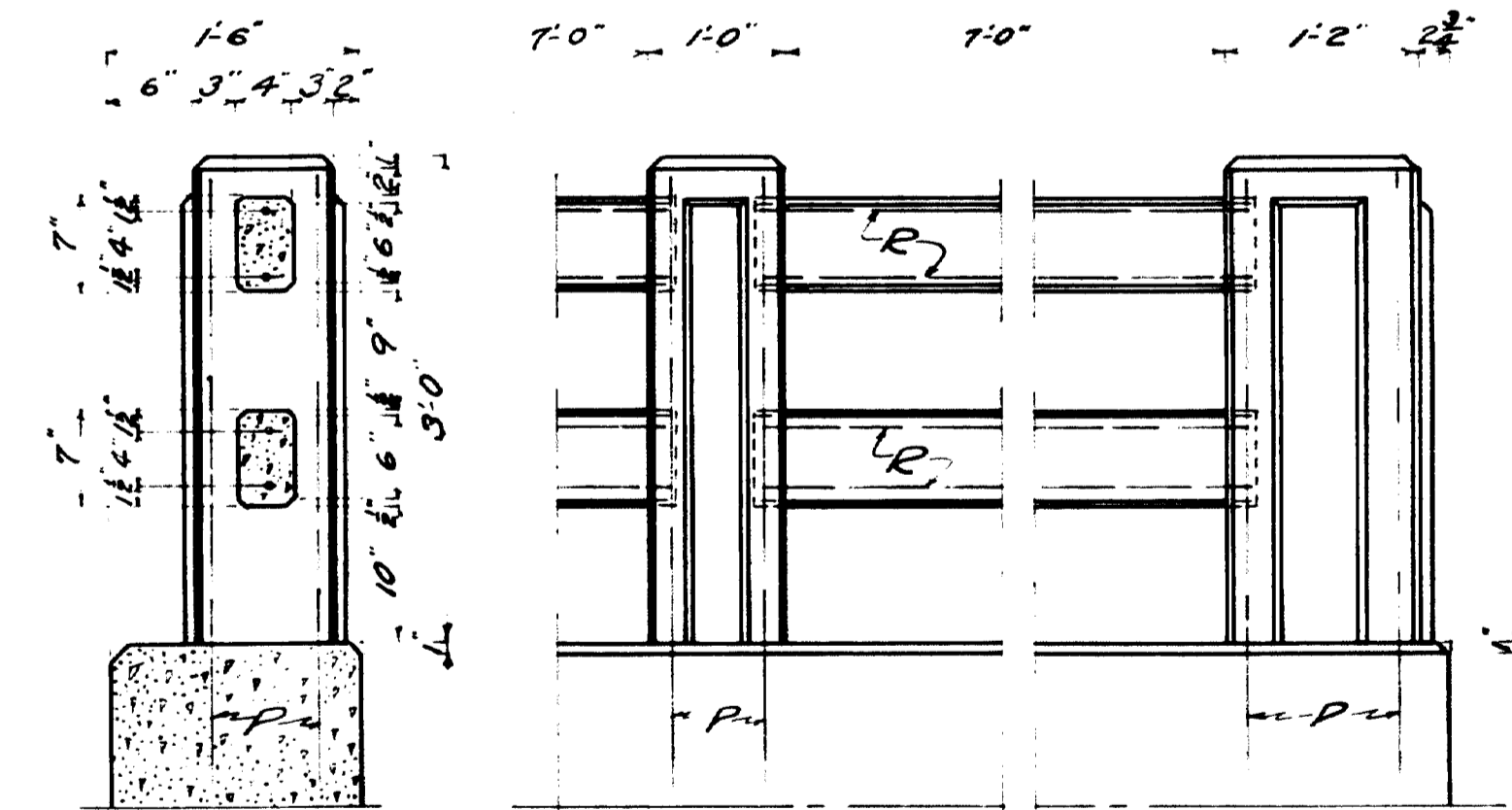
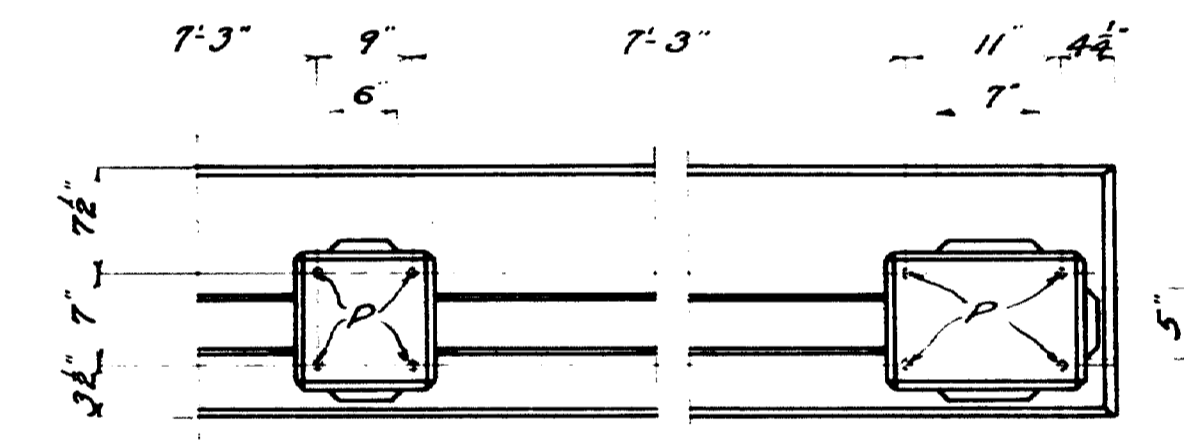
HALF TRANSVERSE SECTION

HALF REAR ELEVATION



HALF ABUTMENT PLAN

NOTE:
Vertical joints in each course shall overlap those in the course below at least 12". No shall any vertical joint occur directly above or below a header.
Each step forming the top of a wing wall shall be composed of a single stone and shall overlap the stone forming the step below it at least 12".



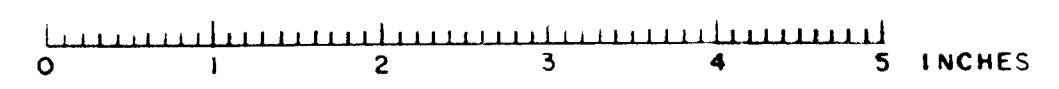
HANDRAIL DETAILS

Curb to be cast with slab. Place vertical rods for posts in curb. Precast rail bars 7'-5" long. Forms for posts to be built with rail bars in position, extending 2 1/2" into post forms. Wrap and 6" of bar with 2 layers of 2-ply roofing felt. Fold in ends. After post forms are removed, cut away all exposed felt. Panels on posts 3/4" thick. Chamfer all exposed edges of concrete 1/2" unless otherwise shown.

STEEL SCHEDULE				
BENT BARS				
Mark	Size	No Bars	Length	Location
F1	1/2"	27	20'-4 1/2"	Slab
STRAIGHT BARS				
Mark	Size	No Bars	Length	Location
S1	1/2"	36	5'-10 1/2"	Slab and curb
F2	3/4"	28	18'-3 1/2"	Slab
F3	5/8"	13	22'-9"	Slab
C	3/4"	4	17'-7"	Curb
P	3/4"	24	4'-0"	Handrail posts
B	3/4"	16	7'-4"	Handrail
A1	3/4"	8	24'-4"	Abutment caps

All steel dimensions are given on E of bar. Steel to be plain rounds, structural grade.

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
CLARK BRIDGE
IN THE TOWN OF
TREMONT-HANCOCK Co.
SURVEY PLAN
SHEET 2 AUGUSTA, ME. OCT 15, 1924



APPENDIX B
BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																												
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES																													
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p>Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.</p> <p>Modified Burmister System</p> <table border="1"> <thead> <tr> <th>Descriptive Term</th> <th>Portion of Total</th> </tr> </thead> <tbody> <tr> <td>trace</td> <td>0% - 10%</td> </tr> <tr> <td>little</td> <td>11% - 20%</td> </tr> <tr> <td>some</td> <td>21% - 35%</td> </tr> <tr> <td>adjective (e.g. sandy, clayey)</td> <td>36% - 50%</td> </tr> </tbody> </table> <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>> 50</td> </tr> </tbody> </table>	Descriptive Term	Portion of Total	trace	0% - 10%	little	11% - 20%	some	21% - 35%	adjective (e.g. sandy, clayey)	36% - 50%	Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50					
		Descriptive Term	Portion of Total																													
		trace	0% - 10%																													
	little	11% - 20%																														
	some	21% - 35%																														
	adjective (e.g. sandy, clayey)	36% - 50%																														
Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)																															
Very loose	0 - 4																															
Loose	5 - 10																															
Medium Dense	11 - 30																															
Dense	31 - 50																															
Very Dense	> 50																															
GRAVEL WITH FINES (Appreciable amount of fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines																														
SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines																													
	SANDS WITH FINES (Appreciable amount of fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.																													
	SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures																													
SC		Clayey sands, sand-clay mixtures.																														
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	<p>Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.</p> <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value blows per foot</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr> <td>Very Soft</td> <td>WOH, WOR, WOP, <2</td> <td>0 - 250</td> <td>Fist easily Penetrates</td> </tr> <tr> <td>Soft</td> <td>2 - 4</td> <td>250 - 500</td> <td>Thumb easily penetrates</td> </tr> <tr> <td>Medium Stiff</td> <td>5 - 8</td> <td>500 - 1000</td> <td>Thumb penetrates with moderate effort</td> </tr> <tr> <td>Stiff</td> <td>9 - 15</td> <td>1000 - 2000</td> <td>Indented by thumb with great effort</td> </tr> <tr> <td>Very Stiff</td> <td>16 - 30</td> <td>2000 - 4000</td> <td>Indented by thumb nail</td> </tr> <tr> <td>Hard</td> <td>>30</td> <td>over 4000</td> <td>Indented by thumb nail with difficulty</td> </tr> </tbody> </table>	Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumb nail	Hard	>30	over 4000	Indented by thumb nail with difficulty
		Consistency of Cohesive soils	SPT N-Value blows per foot		Approximate Undrained Shear Strength (psf)	Field Guidelines																										
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	Medium Stiff	5 - 8	500 - 1000		Thumb penetrates with moderate effort																											
	Stiff	9 - 15	1000 - 2000		Indented by thumb with great effort																											
Very Stiff	16 - 30	2000 - 4000	Indented by thumb nail																													
Hard	>30	over 4000	Indented by thumb nail with difficulty																													
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																															
OL	Organic silts and organic silty clays of low plasticity.																															
SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.																														
	CH	Inorganic clays of high plasticity, fat clays.																														
	OH	Organic clays of medium to high plasticity, organic silts																														
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.																														
<p>Desired Soil Observations: (in this order)</p> <p>Color (Munsell color chart) Moisture (dry, damp, moist, wet, saturated) Density/Consistency (from above right hand side) Name (sand, silty sand, clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc., if applicable) Cementation (weak, moderate, or strong, if applicable, ASTM D 2488) Geologic Origin (till, marine clay, alluvium, etc.) Unified Soil Classification Designation Groundwater level</p>				<p>Rock Quality Designation (RQD):</p> <p>RQD = $\frac{\text{sum of the lengths of intact pieces of core} > 100 \text{ mm}}{\text{length of core advance}}$</p> <p>*Minimum NQ rock core (1.88 in. OD of core)</p> <p>Correlation of RQD to Rock Mass Quality</p> <table border="1"> <thead> <tr> <th>Rock Mass Quality</th> <th>RQD</th> </tr> </thead> <tbody> <tr> <td>Very Poor</td> <td><25%</td> </tr> <tr> <td>Poor</td> <td>26% - 50%</td> </tr> <tr> <td>Fair</td> <td>51% - 75%</td> </tr> <tr> <td>Good</td> <td>76% - 90%</td> </tr> <tr> <td>Excellent</td> <td>91% - 100%</td> </tr> </tbody> </table> <p>Desired Rock Observations: (in this order)</p> <p>Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Lithology (igneous, sedimentary, metamorphic, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90) -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m) -tightness (tight, open or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock mass quality (very poor, poor, etc.) ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A Recovery</p>		Rock Mass Quality	RQD	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%															
Rock Mass Quality	RQD																															
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<p>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</p>				<p>Sample Container Labeling Requirements:</p> <table border="1"> <tbody> <tr> <td>PIN</td> <td>Blow Counts</td> </tr> <tr> <td>Bridge Name / Town</td> <td>Sample Recovery</td> </tr> <tr> <td>Boring Number</td> <td>Date</td> </tr> <tr> <td>Sample Number</td> <td>Personnel Initials</td> </tr> <tr> <td>Sample Depth</td> <td></td> </tr> </tbody> </table>		PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth																		
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-101 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11.2	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 2" and 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 2" SS, 300/16 3" SS
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: N153296.02 E2172345.66	Casing ID/OD: HW	Water Level*: 8.9

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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.5 - 2.5	15/11/14/12	25	25	SSA	10.6	0-0.3 ft. Asphalt wearing course 0.3-0.5 ft. Asphalt base course Top 1": Weathered asphalt base course	G#238384 A-1-b WC=6.7%	
								8.2	Bottom 11": Light and dark brown, damp, medium dense, fine to medium sandy GRAVEL, little silt, trace clay (Subgrade Fill).		
5	2MD	24/0	5.0 - 7.0	7/7/26/9	33	33			a=Advanced casing with HW spinning shoe. Lost water at 3 ft., cobble fill at 3.5 ft based on drill behavior. Intermittent water return to 5 ft. No recovery, resampled with 3" spoon. Gray, damp 3" cobble, little gravel, trace silt (Rock Fill).		
									Boulder approximately 7 to 8 ft. based on drill behavior. Cobble fill below based on drill behavior. Intermittent water return 8 to 10 ft.		
10	3D	1/1	10.0 - 10.1	50-(1") _b				0.4	b=Used 3" split spoon. Gray, wet, coarse GRAVEL, trace coarse sand, trace silt (Rock Fill).		
	R1	60/60	12.0 - 17.0	RQD = 42%			NQ-2		Top of Bedrock Elev. 0.4 ft. Roller coned ahead to 12 ft. Seam based on drill behavior 11.8 to 11.9 ft. R1: 12.0-12.9': Dark gray to greenish black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular. 12.9-17.0': Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Rock Mass Quality = Poor	PLT _{q_p} =26,800 psi	
	R2	50/50	17.0 - 21.2	RQD = 50%					R1: Core Times (min:sec) 12.0-13.0': (2:55) 13.0-14.0': (2:30) 14.0-15.0': (2:15) 15.0-16.0': (2:40) 16.0-17.0': (3:30) Recovery = 100%	UCT _{q_p} =31,567 psi PLT _{q_p} =37,000 psi	
20									R2: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft., approximately 1 inch thick and subhorizontal. Rock Mass Quality = Poor		
25								-10.0			

Remarks:

- CLD provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.
- PLT_{q_p} = Peak compressive strength estimated from Point Load Index Test.
- UCT_{q_p} = Peak compressive strength from Uniaxial Compressive Strength Test.

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-102 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11.4	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: N153303.70 E2172331.97	Casing ID/OD: HW	Water Level*: 8.8

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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/12	0.5 - 2.5	19/10/11/10	-		SSA	10.7		0-0.4 ft. Asphalt wearing course. 0.4-0.5 ft Asphalt base course 3 inch spoon: Top 3": Weathered asphalt base course		
	2D	1/0	2.5 - 2.6	50-(1")			HW _a	8.9		Middle 8": Light brown, damp, sandy fine to coarse GRAVEL, little silt (Roadway Base). Bottom 2": Gray, damp, coarse GRAVEL, litte fine to coarse sand, little silt (Rock Fill).		
5	3D	24/0	5.0 - 7.0	7/10/8/1	-					3 inch spoon: No recovery. Rock fill based on drill behavior. HW _a =Advanced HW casing with spinning shoe. No blows performed. Lost water at 4 ft, loose based on drill behavior. 3 inch spoon: No recovery, final blow drove spoon to 7.5 ft., hole stayed open for 4D. 6.5-7.5 ft: Loose based on drill behavior, lost water return. Gray, wet, fine to coarse GRAVEL, trace silt (Rock Fill). Dense at 8.5, lost water return. Gray and dark green, wet, fine to coarse GRAVEL, trace fine to coarse sand (Rock Fill).		
	4D	24/3	7.5 - 9.5	14/12/6/10	-					Top of Bedrock at Elev. 0.3 ft. Roller cone ahead to 11.5 ft. Water returned with casing at 11.5 ft. R1: Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality: Poor R1: Core Times (min:sec) 11.5-12.5': (2:15) 12.5-13.5': (1:50) 13.5-14.5': (2:10) 14.5-15.5': (2:30) 15.5-16.5': (6:25) Recovery = 83% R2: Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Calcite veins up to 1/4 inch thick common. Highly fractured zone 17.5 to 18 ft. deep. Rock Mass Quality = Poor R2: Core Times (min:sec) 16.5-17.5': (1:50)		
10	5D	13/4	10.0 - 11.1	10/26/(50-1")				0.3				
	R1	60/50	11.5 - 16.5	RQD = 45%			NQ-2					
15												
	R2	40/39	16.5 - 19.8	RQD = 33%								
20												
	R3	19/17	19.8 - 21.4	RQD = 68%							UCT _{q_p} =13,189 psi	
25								-10.1				

Remarks:

- MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.
- UCT_{q_p} = Peak compressive strength from Uniaxial Compressive Strength Test.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-102 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11.4	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: N153303.70 E2172331.97	Casing ID/OD: HW	Water Level*: 8.8

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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25										17.5-18.5': (3:35) 18.5-19.5': (2:50) 19.5-19.8': (0:30) Recovery = 98% Core barrel plugged at 19.8 ft, picked up 7" of R1. R3: Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality = Fair R3: Core Times (min:sec) 19.8-21.5': (4:15) <hr style="width: 100%;"/> Bottom of Exploration at 21.5 feet below ground surface.		
26												
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Remarks:

- MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.
- UCT_{q_p} = Peak compressive strength from Uniaxial Compressive Strength Test.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-103 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11.9	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: N153317.75 E2172305.90	Casing ID/OD: HW	Water Level*: 8.9

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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.5 - 2.5	26/12/11/22	-	SSA	11.2		Asphalt wearing course. Asphalt base course. 3 inch spoon: Top 3": Weathered asphalt base course.		
							9.4		Bottom 8": Light brown, moist, fine to coarse SAND, little fine gravel, little silt (Subgrade Fill).		
5	2D	24/3	5.0 - 7.0	6/10/28/11	-		2.5		Gravel and cobbles based on drill behavior. Lost water return at 3 ft. a = Advanced HW casing with spinning shoe. 3 inch spoon: Gray, damp, single 3" cobble, trace silt. Spoon bent during driving (Rock Fill). No return, cobbles based on drill behavior.		
10							2.9		Wood in wash at approximately 9 to 10.3 ft. No return at 10.4 ft.		
							1.5				
	3D	24/10	11.0 - 13.0	9/12/6/13	-		0.9		3 inch spoon: Gray, wet, fine to coarse sandy GRAVEL, little silt (Emerged Marine Sediments). 3 inch spoon: Gray, wet, fine to coarse GRAVEL, some fine to coarse sand, little silt (Emerged Marine Sediments). b = Drove NW casing Wash turns red-gray at approximately 14.5 ft., alternates to blue-gray. 3 inch spoon: Blue-gray, wet, fine to medium SAND, some silt, trace fine gravel (Emerged Marine Sediments). HW Casing bent at bottom, rods not fitting through NW casing. Driller removed NW casing and resumed spinning HW.	G#238385 A-1-a WC=10.7%	
15	4D	24/7	13.0 - 15.0	48/39/50-5"	-	44 _b					
						96					
	5D	6/6	15.0 - 15.5	110(6")		HW _a				G#238387 A-2-4 WC=10.7%	
20	R1	60/36	20.0 - 25.0	RQD = 32%		NQ-2	-6.4		Top of Bedrock at Elev. -6.4 ft. Roller cone ahead to 20 ft. Casing not seating well at 19 ft. R1: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating. Rock Mass Quality = Poor R1: Core Times (min:sec) 20.0-21.0': (2:05) 21.0-22.0': (2:20)	PLT _{pq} =11,900 psi PLT _{pq} =18,800 psi	
25											

Remarks:
1. MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88. 2. PLT_{pq} = Peak compressive strength estimated from Point Load Index Test.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-103 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11.9	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: N153317.75 E2172305.90	Casing ID/OD: HW	Water Level*: 8.9

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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	R2	60/27	25.0 - 30.0	RQD = 7%							22.0-23.0': (2:10) 23.0-24.0': (2:40) 24.0-25.0': (5:30) Recovery = 60% R2: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs. Rock Mass Quality = Very Poor R2: Core Times (min:sec) 25.0-26.0': (2:10) 26.0-27.0': (2:05) 27.0-28.0': (2:35) 28.0-29.0': (2:15) 29.0-30.0': (1:40) Recovery = 45%	
30									-18.1			
35												
40												
45												
50												

Remarks:

1. MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88. 2. PLT_{q_p} = Peak compressive strength estimated from Point Load Index Test.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-104 PIN: 19305.00
--	---	---

Driller: Maine Test Borings	Elevation (ft.): 12.3	Auger ID/OD: 4.5" SSA/6.5" HSA
Operator: M. Porter	Datum: NAVD88	Sampler: 2" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A
Boring Location: N153318.89 E2172279.54	Casing ID/OD: N/A	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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0									11.7	0-0.4 ft.: Asphalt wearing course. 0.4-0.6 ft.: Asphalt base course	G#238388 A-4 WC=21.9%	
	1D	24/12	1.0 - 3.0	17/12/7/6	19	19				Light brown, damp, medium dense, fine to medium SAND, some gravel, some silt (Subgrade Fill). Decreasing gravel with depth.		
	2D	24/14	3.0 - 5.0	9/6/6/13	12	12				Light brown, damp, medium dense, SAND, some silt, trace gravel, trace clay (Subgrade Fill).		
5	3D	4/4	5.0 - 5.3	50-(4")	-				6.9	Red-brown, damp, silty fine SAND, trace organics (roots) (Subgrade Fill).		
										Gravel and Rock fill at 5.4 ft. based on auger behavior. Auger cuttings contain decreasing sand fraction with depth.		
										Auger cuttings primarily broken rock/gravel at 8 ft.		
10	4D	1/0	10.0 - 10.1	50-(1")						No recovery - dense rock fill.		
									0.2	Auger. refusal at Elev. 0.2 ft		
										Bottom of Exploration at 12.1 feet below ground surface.		

Remarks:
1. MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-105 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 10.5	Auger ID/OD: 4.5" SSA/6.5" HSA
Operator: M. Porter	Datum: NAVD88	Sampler: 2" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A
Boring Location: N153261.04 E2172384.42	Casing ID/OD: N/A	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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0												
	1D	24/8	1.0 - 3.0	19/21/15/13	36	36			9.5	0-0.4 ft.: Asphalt wearing course 0.4-0.7 ft.: Asphalt base course		
	2D	24/6	3.0 - 5.0	15/10/6/8	16	16				Top 1": Weathered asphalt base course Bottom 7": Light brown and gray, damp, dense GRAVEL, some fine to coarse sand, trace silt (Subgrade Fill). Light brown and gray, fine to coarse sandy GRAVEL, trace silt (Subgrade Fill). Spoon tip plugged with gravel.		G#238386 A-1-a WC=4.7%
5	3D	8/4	5.0 - 5.7	5/(50-2")	-				5.5	Brown, wet, coarse GRAVEL, some fine sand, little silt (Rock Fill)		
									3.9	Auger refusal at Elev. 3.9 ft.		
										Bottom of Exploration at 6.6 feet below ground surface.		
10												
15												
20												
25												

Remarks:

1. MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-106 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 10.5	Auger ID/OD: 4.5" SSA/6.5" HSA
Operator: M. Porter	Datum: NAVD88	Sampler: 2" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A
Boring Location: N153247.51 E2172407.71	Casing ID/OD: N/A	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, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	9.5	0-0.6 ft.: Asphalt wearing course 0.6-0.8ft.: Asphalt base course		
	1D	24/14	1.0 - 3.0	16/23/18/17	41	41			Light brown-gray, damp, dense, fine to medium SAND, some gravel, little silt (Subgrade Fill).		
	2D	24/12	3.0 - 5.0	16/11/6/3	17	17			Brown, damp, medium dense, fine to medium SAND, some silt, little gravel (Subgrade Fill).		
5	3D	24/15	5.0 - 7.0	3/3/4/32	7	7	HSA	5.5	Dark gray, wet, loose, fine to medium sandy SILT, little clay, trace organics (bark and roots) (Emerged Marine Sediments).	G#238389 A-4 WC=46.6%	
								3.0	Gravel/broken rock based on auger behavior. Auger Refusal at Elev. 3.0 ft.		
									Bottom of Exploration at 7.5 feet below ground surface.		
10											
15											
20											
25											

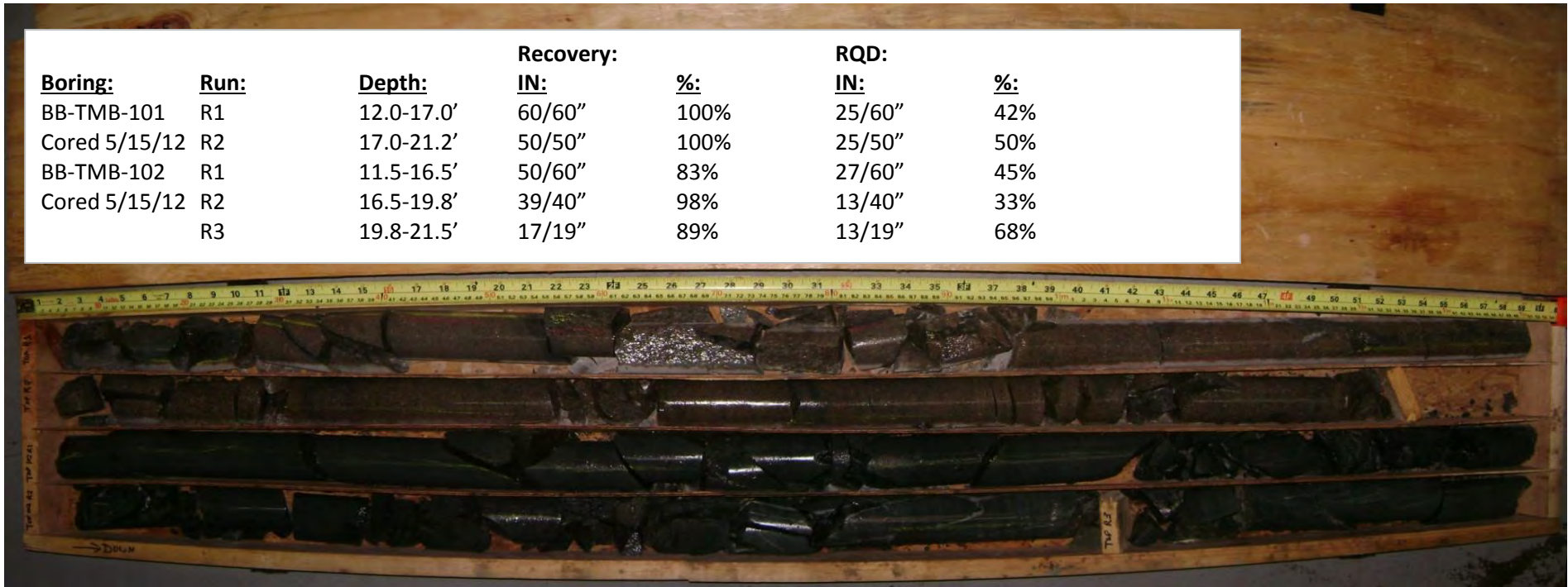
Remarks:

1. MaineDOT provided coordinates of test boring locations in NAD83 (96) ME 2000 West Zone Coordinate System and ground surface elevations in NAVD88.

APPENDIX C
ROCK CORE PHOTOGRAPHS

**APPENDIX C
Clark Bridge Replacement
Tremont, Maine
MaineDOT WIN: 19305.00**

Photo No. 1



Row 1 = BB-TMB-101 Run 1: 12.0 - 17.0 ft-bgs.

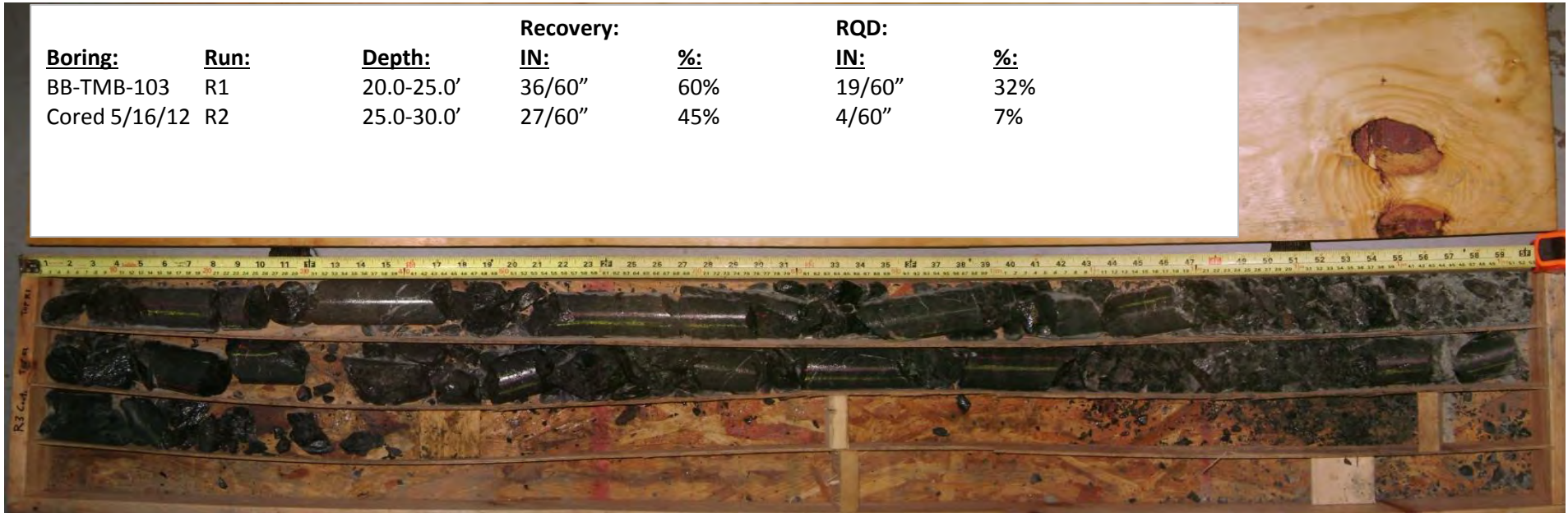
Row 2 = BB-TMB-101 Run 2: 17.0 - 21.2 ft-bgs.

Row 3 = BB-TMB-102 Run 1: 11.5 - 16.5 ft-bgs.

Row 4 = BB-TMB-102 Run 2: 16.5 - 19.5 ft-bgs and Run 3: 19.5 - 21.4 ft-bgs.

APPENDIX C
Clark Bridge Replacement
Tremont, Maine
MaineDOT WIN: 19305.00

Photo No. 2



Row 1 = BB-TMB-103 Run 1: 20.0 - 25.0 ft-bgs.

Row 2 = BB-TMB-103 Run 2: 25.0 - 30.0 ft-bgs.

Row 3 = BB-TMB-103 Run 2: 25.0 - 30.0 ft-bgs continued.

APPENDIX D
LABORATORY TEST RESULTS OF SOIL SAMPLES

APPENDIX D
LABORATORY TEST RESULTS FOR SOIL SAMPLES

Presented in the following order

INDEX AND CLASSIFICATION TESTING:

MOISTURE CONTENT

GRAIN SIZE

INDEX AND CLASSIFICATION TESTING

MOISTURE CONTENT



Client: Golder Associates	Project No: GTX-11871	
Project: VHB Tremont	Tested By: jek	
Location: Tremont, ME	Sample Type: ---	Checked By: jdt
Boring ID: ---	Test Date: 06/15/12	Sample Id: ---
Sample ID: ---		
Depth : ---		

Moisture Content of Soil - ASTM D 2216-05

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-TMB-101	1D	---	Moist, dark brown silty gravel with sand	6.7
BB-TMB-103	3D	---	Moist, gray gravel with silt and sand	10.7
BB-TMB-103	5D	---	Moist, dark olive gray silty sand	10.7
BB-TMB-104	2D	---	Moist, dark yellowish brown silty sand	21.9
BB-TMB-105	1D	---	Moist, brown gravel with silt and sand	4.7
BB-TMB-106	3D	---	Moist, very dark brown sandy silt	46.6

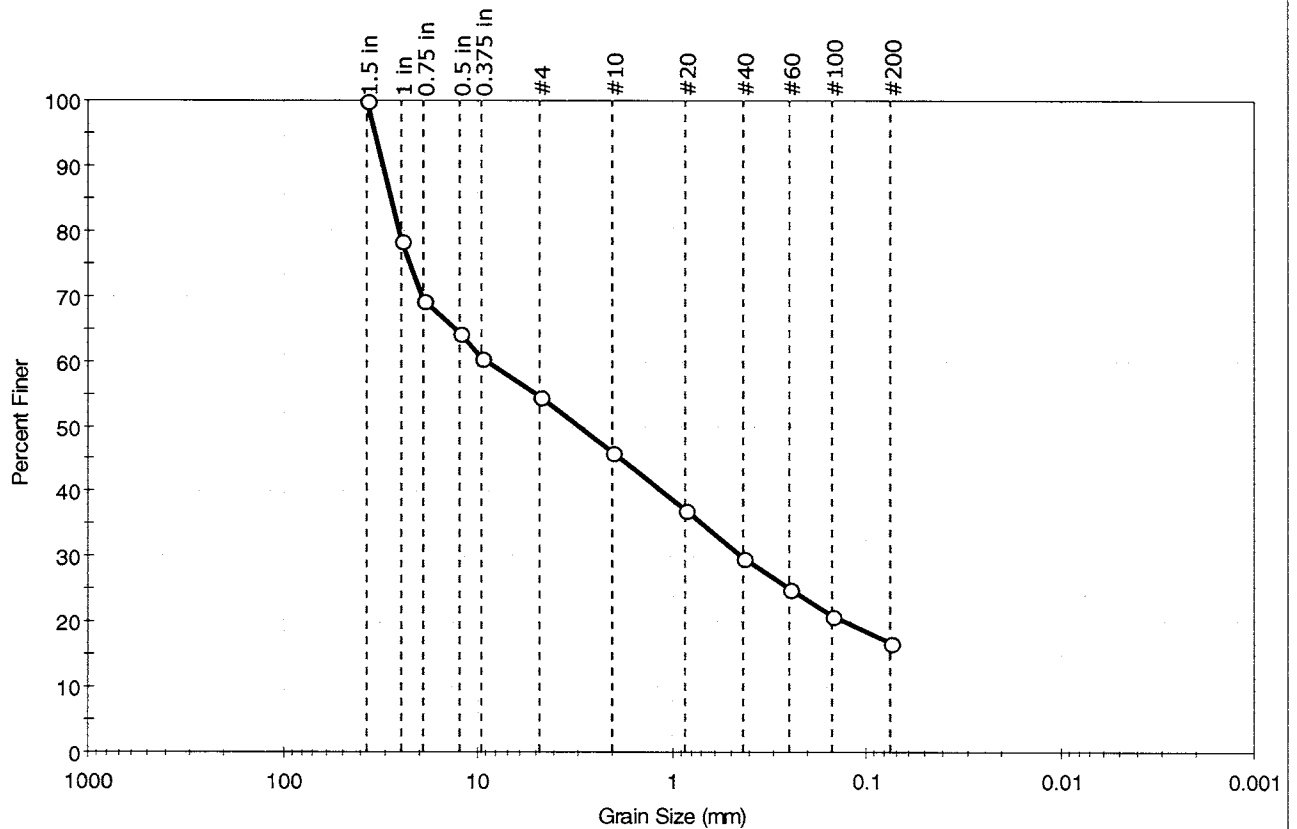
Notes: Temperature of Drying : 110° Celsius

GRAIN SIZE



Client: Golder Associates	Project: VHB Tremont		Location: Tremont, ME		Project No: GTX-11871
Boring ID: BB-TMB-101	Sample Type: jar	Tested By: jbr		Checked By: jdt	
Sample ID:1D	Test Date: 06/04/12	Test Id: 238384		Depth : ---	
Test Comment: ---					
Sample Description: Moist, dark brown silty gravel with sand					
Sample Comment: ---					

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



%Cobble	%Gravel	%Sand	%Silt & Clay Size
—	45.3	37.8	16.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	78		
0.75 in	19.00	69		
0.5 in	12.50	64		
0.375 in	9.50	61		
#4	4.75	55		
#10	2.00	46		
#20	0.85	37		
#40	0.42	30		
#60	0.25	25		
#100	0.15	21		
#200	0.075	17		

Coefficients	
D ₈₅ = 28.3100 mm	D ₃₀ = 0.4276 mm
D ₆₀ = 8.8425 mm	D ₁₅ = N/A
D ₅₀ = 2.9753 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

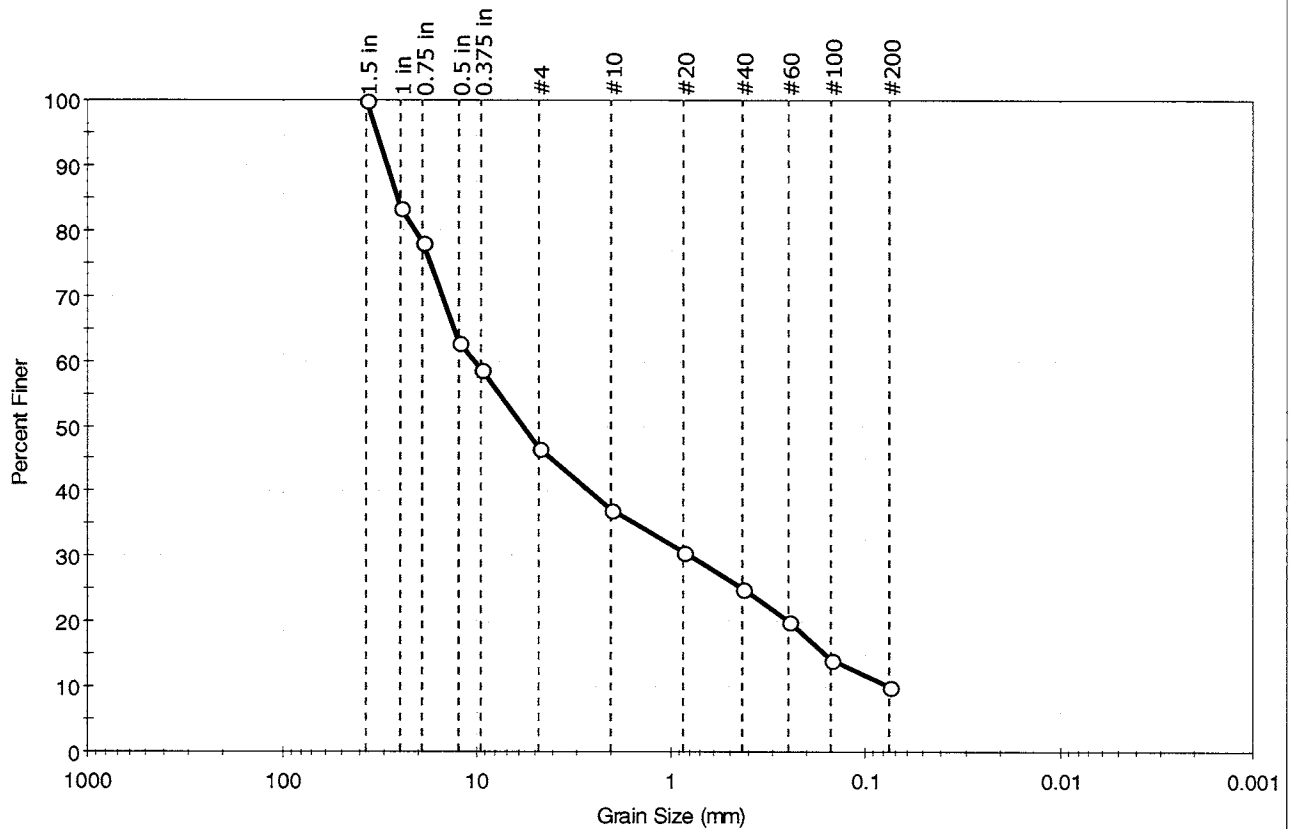
Classification	
ASTM	N/A
AASHTO	Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description
Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD



Client: Golder Associates	Project: VHB Tremont	Location: Tremont, ME	Project No: GTX-11871
Boring ID: BB-TMB-103	Sample Type: jar	Tested By: jbr	Checked By: jdt
Sample ID:3D	Test Date: 06/04/12	Test Id: 238385	
Depth : ---			
Test Comment: ---			
Sample Description: Moist, gray gravel with silt and sand			
Sample Comment: ---			

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



%Cobble	%Gravel	%Sand	%Silt & Clay Size
—	53.4	36.4	10.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	84		
0.75 in	19.00	78		
0.5 in	12.50	63		
0.375 in	9.50	59		
#4	4.75	47		
#10	2.00	37		
#20	0.85	31		
#40	0.42	25		
#60	0.25	20		
#100	0.15	14		
#200	0.075	10		

Coefficients	
D ₈₅ = 25.9278 mm	D ₃₀ = 0.7824 mm
D ₆₀ = 10.3165 mm	D ₁₅ = 0.1605 mm
D ₅₀ = 5.7699 mm	D ₁₀ = 0.0728 mm
C _u = 141.710	C _c = 0.815

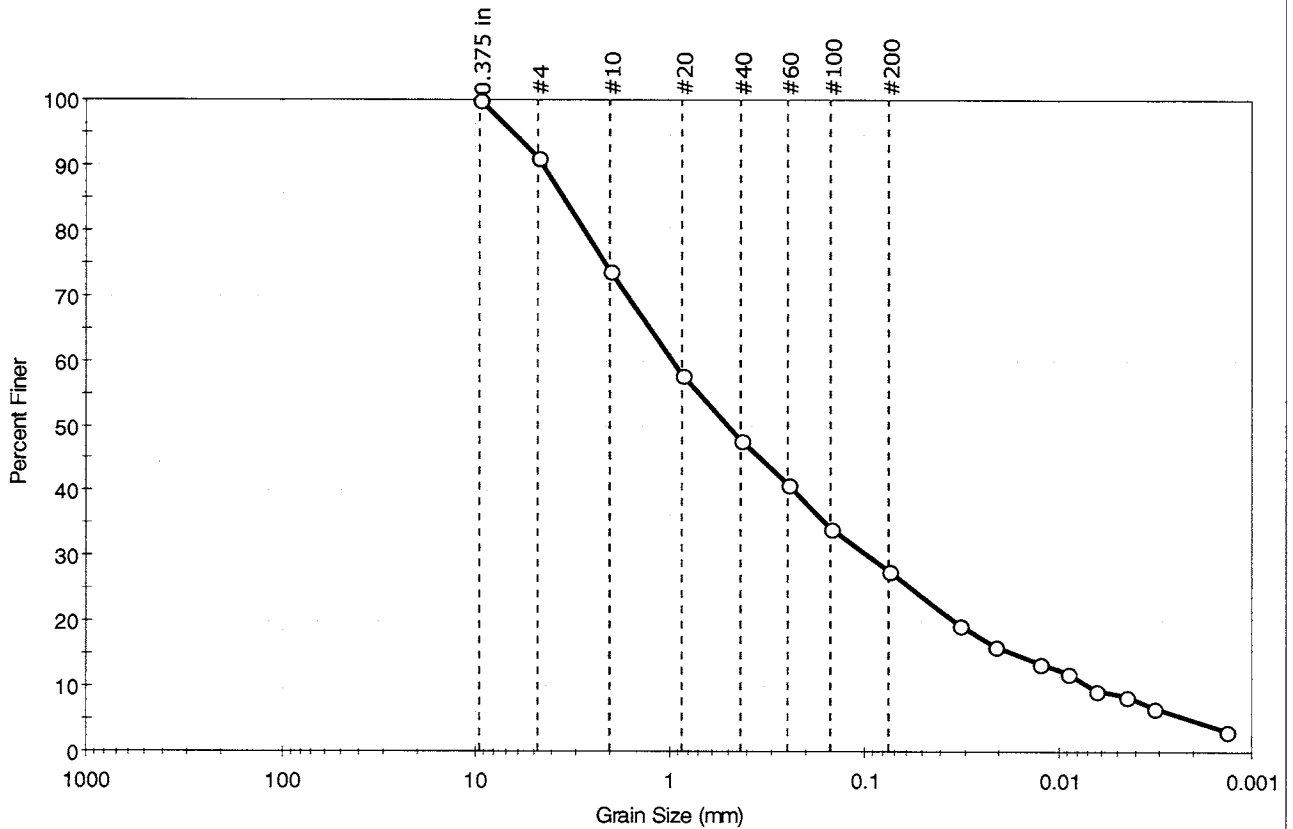
Classification	
ASTM	N/A
AASHTO	Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description	
Sand/Gravel Particle Shape :	ROUNDED
Sand/Gravel Hardness :	HARD



Client: Golder Associates	Project: VHB Tremont	Location: Tremont, ME	Project No: GTX-11871
Boring ID: BB-TMB-103	Sample Type: jar	Tested By: jbr	Checked By: jdt
Sample ID: 5D	Test Date: 06/11/12	Test Id: 238387	
Depth: ---			
Test Comment: ---			
Sample Description: Moist, dark olive gray silty sand			
Sample Comment: ---			

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



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	9.0	63.2	27.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	91		
#10	2.00	74		
#20	0.85	58		
#40	0.42	48		
#60	0.25	41		
#100	0.15	34		
#200	0.075	28		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0324	19		
---	0.0213	16		
---	0.0125	14		
---	0.0089	12		
---	0.0064	9		
---	0.0045	8		
---	0.0032	7		
---	0.0014	3		

Coefficients	
D ₈₅ = 3.5045 mm	D ₃₀ = 0.0955 mm
D ₆₀ = 0.9533 mm	D ₁₅ = 0.0163 mm
D ₅₀ = 0.4924 mm	D ₁₀ = 0.0068 mm
C _u = N/A	C _c = N/A

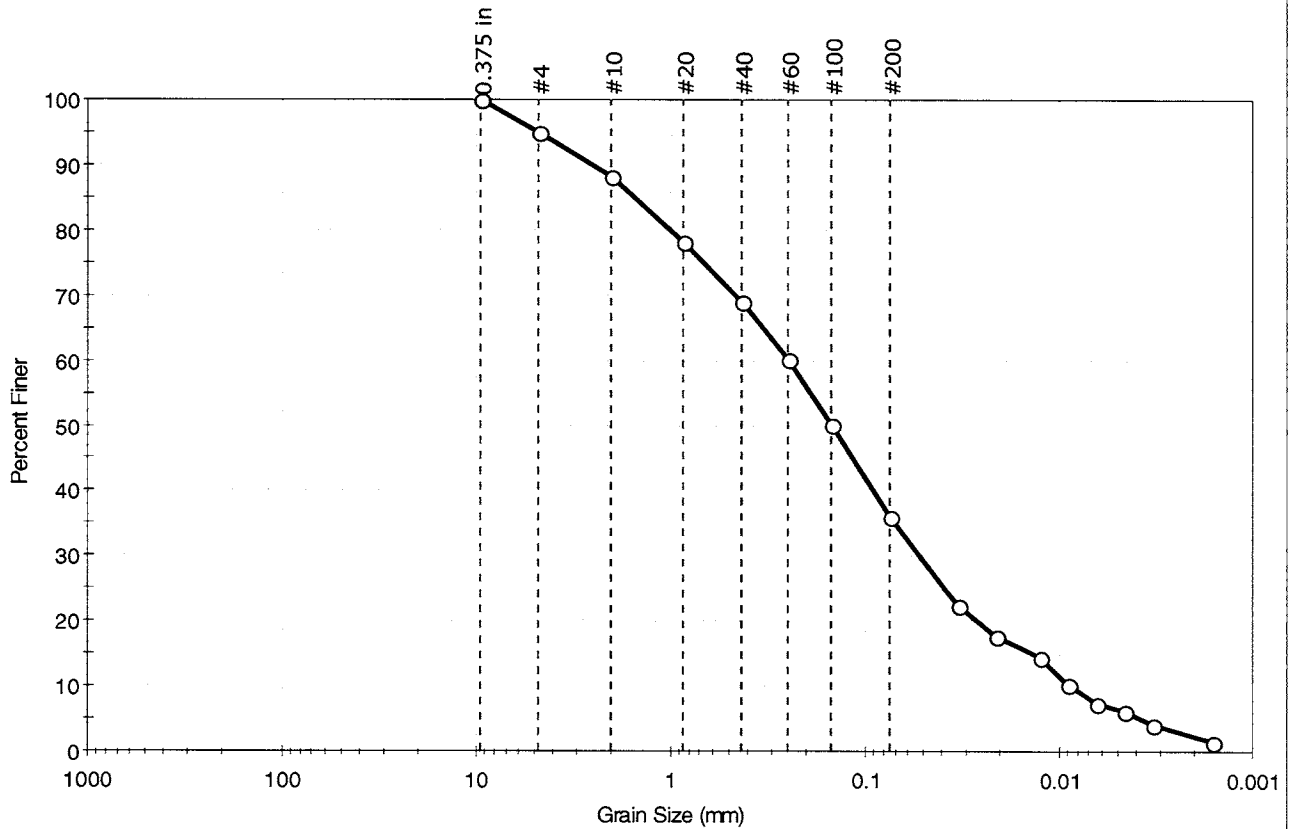
Classification	
ASTM	N/A
AASHTO	Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description
Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---



Client: Golder Associates	Project: VHB Tremont	Location: Tremont, ME	Project No: GTX-11871
Boring ID: BB-TMB-104	Sample Type: jar	Tested By: jbr	Checked By: jdt
Sample ID: 2D	Test Date: 06/08/12	Test Id: 238388	
Depth: ---			
Test Comment: ---			
Sample Description: Moist, dark yellowish brown silty sand			
Sample Comment: ---			

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



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	5.0	59.0	36.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	95		
#10	2.00	88		
#20	0.85	78		
#40	0.42	69		
#60	0.25	60		
#100	0.15	50		
#200	0.075	36		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0334	23		
---	0.0214	18		
---	0.0127	14		
---	0.0091	10		
---	0.0065	8		
---	0.0046	6		
---	0.0033	4		
---	0.0016	1		

<u>Coefficients</u>	
D ₈₅ = 1.5117 mm	D ₃₀ = 0.0523 mm
D ₆₀ = 0.2468 mm	D ₁₅ = 0.0140 mm
D ₅₀ = 0.1488 mm	D ₁₀ = 0.0088 mm
C _u = N/A	C _c = N/A

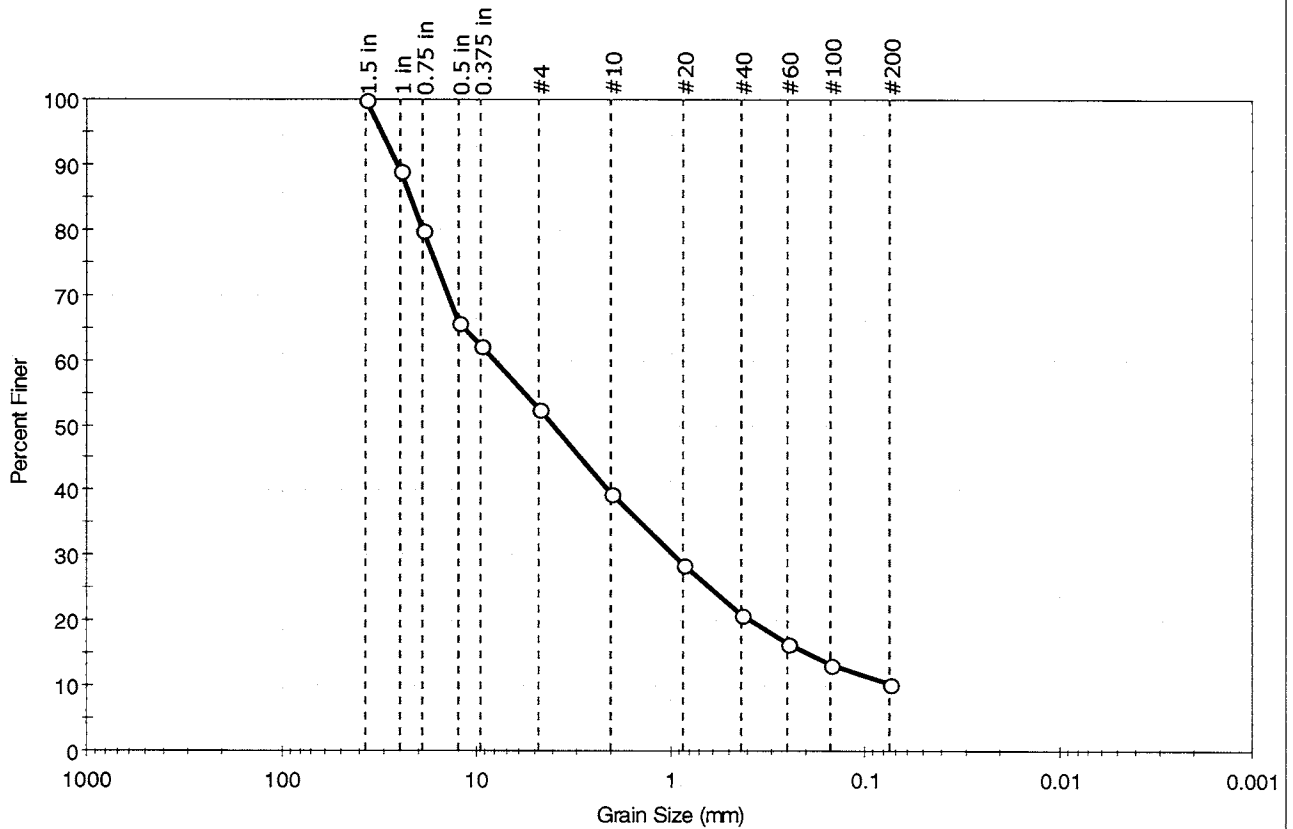
<u>Classification</u>	
ASTM	N/A
AASHTO	Silty Soils (A-4 (0))

<u>Sample/Test Description</u>
Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD



Client: Golder Associates	Project: VHB Tremont	Location: Tremont, ME	Project No: GTX-11871
Boring ID: BB-TMB-105	Sample Type: jar	Tested By: jbr	Checked By: jdt
Sample ID:1D	Test Date: 06/04/12	Test Id: 238386	
Depth : ---			
Test Comment: ---			
Sample Description: Moist, brown gravel with silt and sand			
Sample Comment: ---			

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



%Cobble	%Gravel	%Sand	%Silt & Clay Size
—	47.6	42.1	10.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	89		
0.75 in	19.00	80		
0.5 in	12.50	66		
0.375 in	9.50	62		
#4	4.75	52		
#10	2.00	39		
#20	0.85	29		
#40	0.42	21		
#60	0.25	16		
#100	0.15	13		
#200	0.075	10		

Coefficients

D ₈₅ = 22.1065 mm	D ₃₀ = 0.9528 mm
D ₆₀ = 8.1204 mm	D ₁₅ = 0.1978 mm
D ₅₀ = 4.0266 mm	D ₁₀ = 0.0691 mm
C _u = 117.517	C _c = 1.618

Classification

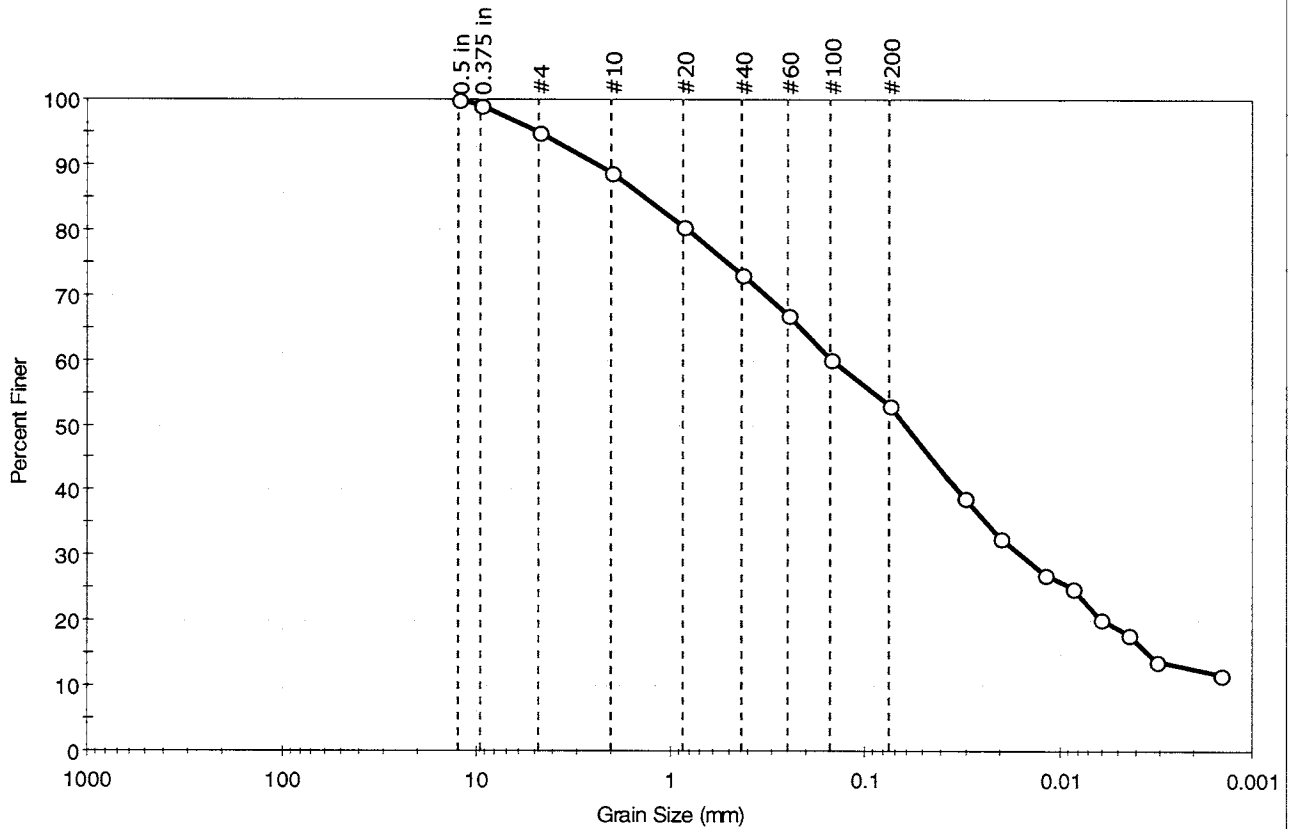
ASTM	N/A
AASHTO	Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description
 Sand/Gravel Particle Shape : ROUNDED
 Sand/Gravel Hardness : HARD



Client: Golder Associates	Project: VHB Tremont	Location: Tremont, ME	Project No: GTX-11871
Boring ID: BB-TMB-106	Sample Type: jar	Tested By: jbr	Checked By: jdt
Sample ID: 3D	Test Date: 06/08/12	Test Id: 238389	
Depth: ---			
Test Comment: ---			
Sample Description: Moist, very dark brown sandy silt			
Sample Comment: ---			

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



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	5.0	42.1	52.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	95		
#10	2.00	89		
#20	0.85	80		
#40	0.42	73		
#60	0.25	67		
#100	0.15	60		
#200	0.075	53		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0314	39		
---	0.0203	33		
---	0.0119	27		
---	0.0085	25		
---	0.0061	20		
---	0.0044	18		
---	0.0032	14		
---	0.0015	12		

<u>Coefficients</u>	
D ₈₅ = 1.3628 mm	D ₃₀ = 0.0156 mm
D ₆₀ = 0.1475 mm	D ₁₅ = 0.0034 mm
D ₅₀ = 0.0623 mm	D ₁₀ = 0.0007 mm
C _u = N/A	C _c = N/A

<u>Classification</u>	
ASTM	N/A
AASHTO	Silty Soils (A-4 (0))

<u>Sample/Test Description</u>
Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD

APPENDIX E
LABORATORY TEST RESULTS OF ROCK CORE SAMPLES
Presented in the following order

POINT LOAD STRENGTH INDEX
COMPRESSIVE STRENGTH AND ELASTIC MODULI

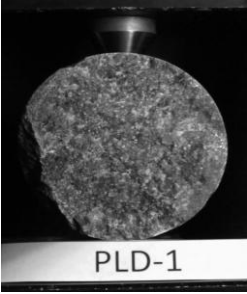

POINT LOAD STRENGTH INDEX



Client:	Golder Associates	Test Date:	06/12/12
Project Name:	VHB Tremont	Tested By:	jsc
Project Location:	Tremont, ME	Checked By:	mpd
GTX #:	11871	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Specimen Diameter (D), in.	Specimen Length (L), in.	Failure Load (P), lbs.	D_e^2 , in ²	D_e , in.	I_s , psi	F	$I_{s(50mm)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-101	R1	13.30-13.90	PLD - 1	Diametral	2.00	6.09	4640	3.98	2.00	1166	1.006	1173	23	26,800

PLD - 1 before 	PLD - 1 after 	Intact material failure
--	--	-------------------------

Notes:

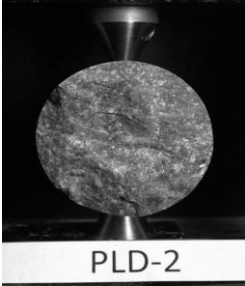

- Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
- D_e = the equivalent core diameter
- I_s = the uncorrected point load strength index
- F = the size correction factor
- $I_{s(50)}$ = the size corrected point load strength index



Client:	Golder Associates	Test Date:	06/12/12
Project Name:	VHB Tremont	Tested By:	jsc
Project Location:	Tremont, ME	Checked By:	mpd
GTX #:	11871	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Specimen Diameter (D), in.	Specimen Length (L), in.	Failure Load (P), lbs.	D_e^2 , in ²	D_e , in.	I_s , psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-101	R2	18.80-19.20	PLD - 2	Diametral	1.99	4.30	6378	3.96	1.99	1609	1.005	1617	23	37,000

PLD - 2 before 	PLD - 2 after 	Intact material failure
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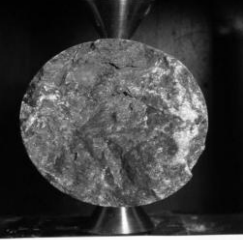

- Notes:
- Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
 - D_e = the equivalent core diameter
 - I_s = the uncorrected point load strength index
 - F = the size correction factor
 - $I_{s(50)}$ = the size corrected point load strength index



Client:	Golder Associates	Test Date:	06/12/12
Project Name:	VHB Tremont	Tested By:	jsc
Project Location:	Tremont, ME	Checked By:	mpd
GTX #:	11871	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Specimen Diameter (D), in.	Specimen Length (L), in.	Failure Load (P), lbs.	D_e^2 , in ²	D_e , in.	I_s , psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-103	R1	20.80-21.20	PLD - 3	Diametral	1.99	4.77	2055	3.97	1.99	518	1.005	521	23	11,900

<p>PLD - 3 before</p> <div style="text-align: center;">  <p>PLD-3</p> </div>	<p>PLD - 3 after</p> <div style="text-align: center;">  </div>	<p>Intact material failure</p>
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Notes:

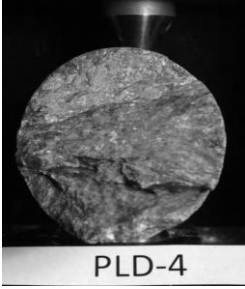

- Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
- D_e = the equivalent core diameter
- I_s = the uncorrected point load strength index
- F = the size correction factor
- $I_{s(50)}$ = the size corrected point load strength index



Client:	Golder Associates	Test Date:	06/12/12
Project Name:	VHB Tremont	Tested By:	jsc
Project Location:	Tremont, ME	Checked By:	mpd
GTX #:	11871	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Specimen Diameter (D), in.	Specimen Length (L), in.	Failure Load (P), lbs.	D_e^2 , in ²	D_e , in.	I_s , psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-103	R2	21.20-21.70	PLD - 4	Diametral	2.00	5.65	3248	3.98	2.00	815	1.006	821	23	18,800

<p>PLD - 4 before</p>  <p style="text-align: center;">PLD-4</p>	<p>PLD - 4 after</p> 	<p>Intact Failure</p>
--	---	-----------------------

Notes:

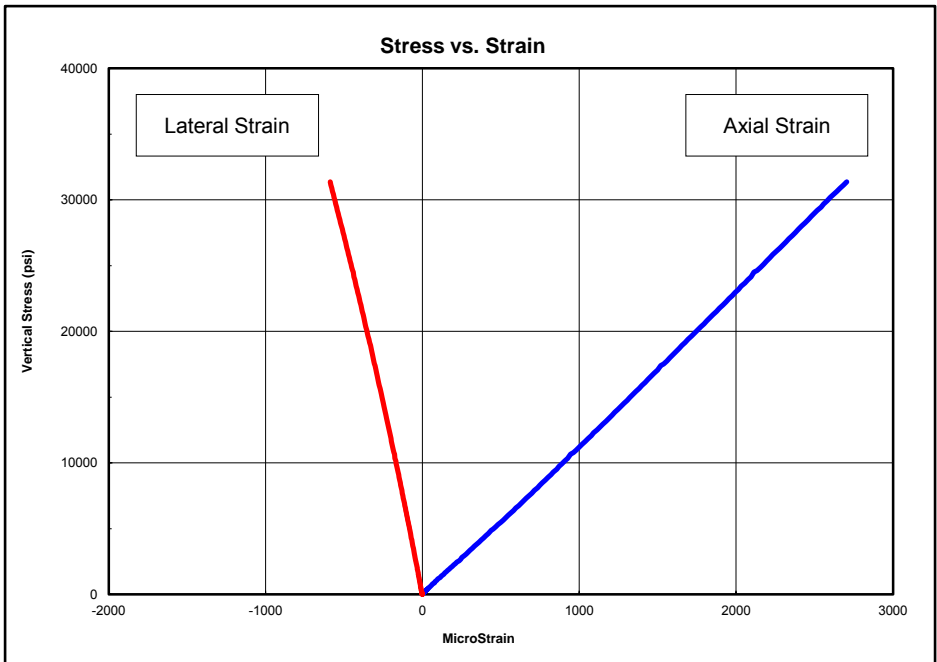
- Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
- D_e = the equivalent core diameter
- I_s = the uncorrected point load strength index
- F = the size correction factor
- $I_{s(50)}$ = the size corrected point load strength index

COMPRESSIVE STRENGTH AND ELASTIC MODULI



Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/8/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-101
Sample ID:	R2
Depth, ft:	18.19-18.56
Sample Type:	rock core
Sample Description:	See photograph Intact material failure

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



Peak Compressive Stress: 31,567 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-10000	11,200,000	0.19
10000-20000	11,800,000	0.22
20000-30000	11,900,000	0.25

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: Golder Associates	Test Date: 6/5/2012
Project Name: VHB Tremont	Tested By: daa
Project Location: Tremont, ME	Checked By: mpd
GTX #: 11871	
Boring ID: BB-TMB-101	
Sample ID: R2	
Depth: 18.19-18.56 ft	
Visual Description: See photographs	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-08

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)			
Specimen Length, in:	1 4.41	2 4.41	Average 4.41	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES			
Specimen Diameter, in:	1.99	1.99	1.99	Maximum difference must be < 0.020 in. Straightness Tolerance Met? YES			
Specimen Mass, g:	594.18						
Bulk Density, lb/ft ³ :	165						
Length to Diameter Ratio:	2.2	Length to Diameter Ratio 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	0.00070	0.00070	0.00050	0.00050	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00050	-0.00070	-0.00080
Diameter 2, in (rotated 90°)	-0.00090	-0.00080	-0.00070	-0.00050	-0.00040	-0.00030	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00040	0.00040	0.00060	0.00080
Difference between max and min readings, in: 0° = 0.00150 90° = 0.00170															
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	0.00060	0.00060	0.00040	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00050	-0.00080	-0.00100
Diameter 2, in (rotated 90°)	-0.00090	-0.00080	-0.00070	-0.00050	-0.00040	-0.00020	-0.00020	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060
Difference between max and min readings, in: 0° = 0.0016 90° = 0.0015 Maximum difference must be < 0.0020 in. Difference = \pm 0.00085 Flatness Tolerance Met? YES															

		<p>DIAMETER 1</p> <p>End 1: Slope of Best Fit Line: -0.00084 Angle of Best Fit Line: -0.04813</p> <p>End 2: Slope of Best Fit Line: -0.00081 Angle of Best Fit Line: -0.04641</p> <p>Maximum Angular Difference: 0.00172</p> <p>Parallelism Tolerance Met? YES Spherically Seated</p>
		<p>DIAMETER 2</p> <p>End 1: Slope of Best Fit Line: 0.00092 Angle of Best Fit Line: 0.05271</p> <p>End 2: Slope of Best Fit Line: 0.00085 Angle of Best Fit Line: 0.04870</p> <p>Maximum Angular Difference: 0.00401</p> <p>Parallelism Tolerance Met? YES Spherically Seated</p>

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)						<i>Maximum angle of departure must be \leq 0.25°</i>	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00150	1.990	0.00075	0.043	YES		
Diameter 2, in (rotated 90°)	0.00170	1.990	0.00085	0.049	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00160	1.990	0.00080	0.046	YES		
Diameter 2, in (rotated 90°)	0.00150	1.990	0.00075	0.043	YES		

Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/8/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-101
Sample ID:	R2
Depth, ft:	18.19-18.56



After cutting and grinding

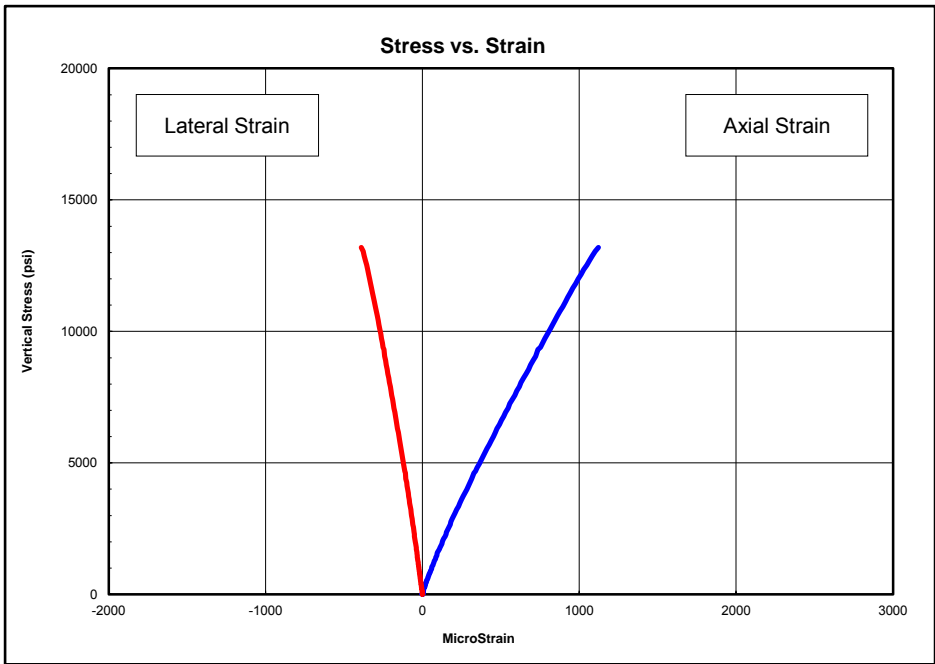


After break



Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/11/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-102
Sample ID:	R2
Depth, ft:	19.20-19.57
Sample Type:	rock core
Sample Description:	See photograph Discontinuity failure

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



Peak Compressive Stress: 13,189 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-4000	13,300,000	0.32
4000-8000	11,600,000	0.33
8000-12000	10,800,000	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: Golder Associates	Test Date: 6/8/2012
Project Name: VHB Tremont	Tested By: daa
Project Location: Tremont, ME	Checked By: mpd
GTX #: 11871	
Boring ID: BB-TMB-102	
Sample ID: R2	
Depth: 19.20-19.57 ft	
Visual Description: See photographs	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-08

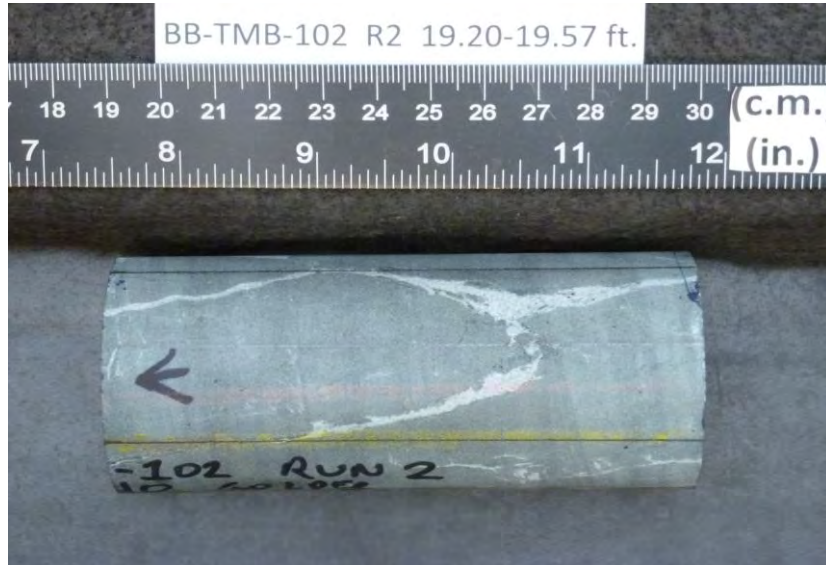
BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)			
Specimen Length, in:	1 4.36	2 4.36	Average 4.36	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES			
Specimen Diameter, in:	1.99	1.99	1.99	Maximum difference must be < 0.020 in. Straightness Tolerance Met? YES			
Specimen Mass, g:	629.78						
Bulk Density, lb/ft ³ :	177						
Length to Diameter Ratio:	2.2	Length to Diameter Ratio 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	0.00070	0.00060	0.00040	0.00040	0.00030	0.00010	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00050	-0.00070	-0.00090	-0.00120
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00030	-0.00040	-0.00050	-0.00060
Difference between max and min readings, in: 0° = 0.00190 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	0.00060	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00040	-0.00060	-0.00080	-0.00100	-0.00120
Diameter 2, in (rotated 90°)	0.00030	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060
Difference between max and min readings, in: 0° = 0.0018 90° = 0.0009 Maximum difference must be < 0.0020 in. Difference = \pm 0.00095 Flatness Tolerance Met? YES															

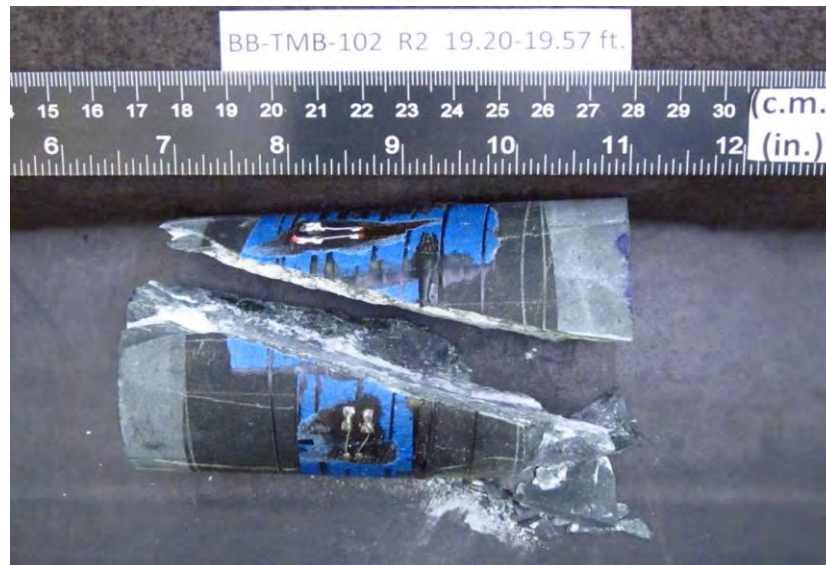
		<p>DIAMETER 1</p> <p>End 1: Slope of Best Fit Line: -0.00097 Angle of Best Fit Line: -0.05558</p> <p>End 2: Slope of Best Fit Line: -0.00102 Angle of Best Fit Line: -0.05844</p> <p>Maximum Angular Difference: 0.00286</p> <p>Parallelism Tolerance Met? YES Spherically Seated</p>
		<p>DIAMETER 2</p> <p>End 1: Slope of Best Fit Line: -0.00045 Angle of Best Fit Line: -0.02578</p> <p>End 2: Slope of Best Fit Line: -0.00047 Angle of Best Fit Line: -0.02693</p> <p>Maximum Angular Difference: 0.00115</p> <p>Parallelism Tolerance Met? YES Spherically Seated</p>

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)						<i>Maximum angle of departure must be \leq 0.25°</i>	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00190	1.990	0.00095	0.055	YES		
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00180	1.990	0.00090	0.052	YES		
Diameter 2, in (rotated 90°)	0.00090	1.990	0.00045	0.026	YES		

Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/11/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-102
Sample ID:	R2
Depth, ft:	19.20-19.57



After cutting and grinding



After break

APPENDIX F
GEOPHYSICAL SURVEY REPORT

**GEOPHYSICAL SURVEY
CLARK BRIDGE REPLACEMENT PROJECT
STATE ROUTE 102, TREMONT, MAINE**

Prepared for:

Golder Associates, Inc.
670 North Commercial Street., Suite 103
Manchester, New Hampshire 03101

Prepared by:

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

File 12SG06
June, 2012

HAGER-RICHTER GEOSCIENCE, INC.

CONSULTANTS IN GEOLOGY AND GEOPHYSICS
8 INDUSTRIAL WAY - D10
SALEM, NEW HAMPSHIRE 03079-5820
TELEPHONE (603) 893-9944
FAX (603) 893-8313

June 19, 2012
File 12SG06

Jay R. Smerekanicz, P.G.
Senior Consulting Geologist and Associate
Golder Associates, Inc.
670 North Commercial Street., Suite 103
Manchester, New Hampshire 03101

Tel: 603-668-0880 Ext.23
Fax: 603-668-1199
Cell: 603-493-6274
Email: Jay_Smerekanicz@golder.com

RE: Geophysical Survey
Clark Bridge Replacement Project
State Route 102, Tremont, Maine

Dear Mr. Smerekanicz:

In this letter, we report the results of a geophysical survey conducted by Hager-Richter Geoscience, Inc. (Hager-Richter) along a portion of State Route 102 in Tremont, Maine for Golder Associates, Inc. (Golder) in May, 2012. The geophysical survey was performed in support of a geotechnical investigation by Golder for the replacement of Clark Bridge carrying State Route 102 over Marshall Creek.

INTRODUCTION

The Site is a portion of State Route 102 in Tremont, Maine that includes Clark Bridge. The bridge is located approximately 700 feet northwest of the junction of Tremont Road (Route 102) and Flat Iron Road. The general location of the Site is shown in Figure 1. Clark Bridge, built in 1924, is a two-lane bridge that carries State Route 102 over Marshall Creek, a tidal creek that drains into Bass Harbor.

Clark Bridge is an approximately 15-foot long concrete span that rests on stone abutments. According to the plans from 1924 provided by Golder, the foundation for the northwest approach to the bridge is approximately 35 feet long and consists largely of timber cribbing. The foundation for the approximately 105-foot long southeast approach is shown as timber cribbing, stone fill, and fieldstone retaining walls, and appears to contain a second bridge span, possibly constructed of timber. Timber cribbing, fieldstone retaining walls, and timber spans were not observable at the time of the survey. Rather, the approaches appear to be cobble and boulder embankment causeways. Figure 2 is a site plan showing the area of interest and the limits of the geophysical survey.

In support of a geotechnical investigation of the site, Golder required information on the depth and configuration of bedrock in the vicinity of the existing bridge, and if possible whether

Geophysical Survey
Clark Bridge Replacement Project
State Route 102, Tremont, Maine
File 12SG06 Page 2

the timber cribbing, fieldstone retaining walls and timber spans are present in the core of the causeways. Golder installed six borings (BB-TMB-101, BB-TMB-102, BB-TMB-103, BB-TMB-104, BB-TMB-105, and BB-TMB-106) in the roadway near the bridge. Refusal on bedrock or top of bedrock determined by coring was encountered at depths of approximately 6.5 feet to 18.3 feet below ground surface. Preliminary logs for the borings provided by Golder are attached to this report in Appendix 1.

OBJECTIVE

The objective of the geophysical survey was to determine (1) the depth and configuration of the bedrock surface and (2) whether timber cribbing, fieldstone retaining walls, and timber spans are present in the core of the bridge approach causeways.

THE SURVEY

The geophysical survey was conducted using ground penetrating radar (GPR) and seismic refraction methods. Michael Howley and Eric Rickert of Hager-Richter conducted the geophysical survey on May 17, 2012. The fieldwork was coordinated with Mr. Jay Smerekanicz, P. G., of Golder, who was on site for the duration of the survey and specified the area of interest. Mr. Michael Henrick, also of Golder, was present for the initiation of the field work and coordinated traffic control services. Data analysis and interpretation were completed at the Hager-Richter offices. Original data and field notes will be retained in the Hager-Richter files for a minimum of three years.

GPR data were acquired along traverses spaced two feet apart oriented parallel to the travel lanes, and along traverses spaced 50 feet apart oriented perpendicular to the travel lanes. The area of interest extended approximately 80 feet northwest and 140 southeast of Clark Bridge along the paved roadway. In addition, seismic refraction profiling was conducted along one 235-foot long seismic line located along the north shoulder of the roadway. Golder provided site plans showing site features and surface topography. The locations of borings installed by Golder were recorded using a differential corrected GPS. Figure 2 is a modified site plan showing the limits of the area surveyed with GPR, the location of the seismic refraction line, and the other site features.

EQUIPMENT

GPR. The GPR survey was conducted using a Sensors and Software Noggin SmartCart Plus digital GPR system equipped with a survey wheel to trigger recording of data at equal

horizontal distances. The GPR system was used with a 250 MHz antenna and a 60 nsec¹ time window. The GPR data were processed using EKKO Mapper 4™ software licensed by Sensors and Software.

Seismic Refraction. For the seismic refraction survey, we used two 24-channel Geometrics Geode digital seismographs, coupled to 48 geophones spaced 5 feet apart. Seven shot points were used per seismic spread - three located internal to the spread, one at each end of the spread, and two offset shots located in-line but outside of the spread of geophones. The seismic source was a sledgehammer striking a metal plate. The seismic refraction data were processed using IXRefrax™ software licensed by Interpex.

LIMITATIONS OF THE METHODS

HAGER-RICHTER GEOSCIENCE, INC. MAKES NO GUARANTEE THAT THE DEPTH OF BEDROCK WAS ACCURATELY DETERMINED IN THIS SURVEY. HAGER-RICHTER GEOSCIENCE, INC. IS NOT RESPONSIBLE FOR DETERMINING THE DEPTH OF BEDROCK WHERE THE INTERFACE CANNOT BE DETECTED BECAUSE OF SITE CONDITIONS. THE BEDROCK DEPTHS DETERMINED SHOULD NOT BE USED FOR CONTRACT BEDROCK REMOVAL QUANTITIES.

GPR. There are limitations of the GPR technique: (1) surface conditions, (2) electrical conductivity and thickness of the subsurface layers, (3) electrical properties of the target(s), and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us in most cases.

The condition of the survey surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. For exterior sites, a surface covered with obstacles such as automobiles, dumpsters, thick leaf debris, materials piles, etc. limit the survey access. Similarly, for interior sites, a surface covered with obstacles such as desks, benches, laboratory equipment, etc. also limit access. Some floor coverings may limit the coupling of the GPR antenna with the subsurface.

The electrical conductivity of the subsurface determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. The GPR signal does not penetrate clay-rich soils or soils contaminated with road salt. In some cases, the GPR signal may not penetrate below concrete pavement, and some asphalts are electrically conducting.

¹ns, abbreviation for nanosecond, 1/1,000,000,000 second. Light and the GPR signal require about 1 ns to travel 1 ft in air. The GPR signal requires about 3.5 ns to travel 1 ft in unsaturated sandy soil.

A strong contrast in the electrical conductivities of the ground and the target (for examples, UST, pipe, void, dry well, drum, contaminant plume) is required to obtain a reflection of the GPR signal. If the contrast is too small, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted on the basis of the size of the target.

Seismic Refraction. Like all geophysical methods, the seismic refraction method is based on the assumption that the local geology is uncomplicated. In particular, the seismic refraction method assumes that interfaces between geologic materials correlate with sharp increases in seismic velocity and that the interfaces between geologic units are relatively flat-lying. The method is not very sensitive to lateral variations within layers, and relatively subtle features such as fracture zones within bedrock are generally difficult to detect unless there is a topographic expression of the feature. The accuracy of the method is degraded in areas with strong topographic relief and/or where the interfaces have apparent dips greater than about 20°. *In general, the standard error of depths determined is about 10% or 2 feet, whichever is greater.*

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

RESULTS

The geophysical survey of the roadway in the vicinity of Clark Bridge consisted of a ground penetrating radar (GPR) survey across the area of interest and a seismic refraction profile along the shoulder of the road. The GPR survey was conducted in an area measuring 20-feet by 235-feet with traverses spaced 2 feet apart oriented parallel to the travel lanes, and along traverses spaced 50 feet apart oriented perpendicular to the travel lanes. The seismic refraction profile consisted of one 235-foot long seismic refraction survey line located on the north shoulder of the roadway. The area surveyed with GPR and the location of the seismic refraction line are shown in Figure 2. The interpretation of the GPR data are shown in Figure 3, and an example GPR profile is shown in Figure 4. The results of the seismic refraction survey are shown in profile form in Figure 5 and are listed in Table 1.

GPR. Apparent GPR signal penetration was generally fair to good across the area of interest, with two-way traveltimes reflections received for 45 to 50 ns of the 60 ns records

recorded. Based on site-specific time-to-depth conversions for the GPR signal at the Site, the GPR signal penetration is estimated to have been approximately 7 to 8 feet.

GPR reflections consistent with those expected for the top of bedrock are only evident in the GPR records for the northwestern edge of the survey area. The depth of bedrock based on boring logs provided by Golder ranges from 6.5 feet to 18.3 feet below ground surface. The bedrock depths indicated by borings in most of the area of interest is either greater than the effective penetration of the GPR signal, or the contrast between the bedrock surface and the overlying soils is not sufficient to generate a detectable GPR reflection.

GPR reflections consistent with a shallow possible buried structure are present in the GPR records, and the location of the possible structure is shown on Figure 3. The structure is located approximately where the former timber span bridge is shown on the 1924 plans supplied by Golder. Reflections indicating several other linear features are present in the GPR records acquired at the site, and their locations are indicated on Figure 3. Reflections from a possible buried surface are interpreted to be caused by the interface between the sand and gravel road bed and the boulder and cobble fill beneath, as indicated in the boring logs. Reflections from the interpreted interface are present northwest of Clark Bridge and southeast of the interpreted timber span structure. Linear reflectors are present in the GPR records oriented perpendicular to the roadway, and are shown in Figure 3. The linear reflectors are interpreted as the edges of the possible timber cribbing that is shown in the 1924 plans.

Figure 4 is an example GPR profile indicating the location of the Clark Bridge, the interpreted buried structure, the linear features, and possible buried surface.

Seismic Refraction Survey. The seismic survey consisted of one 235-foot long seismic refraction line. The location of the seismic line is shown in Figure 2. The results of the seismic survey are shown in profile form in Figure 5 and are listed in Table 1.

The quality of the seismic refraction data ranges from good to excellent. A measure of the accuracy of the data can be obtained by comparing the bedrock depths determined seismically with depths reported from nearby borings that encountered refusals on assumed bedrock, or comparing bedrock depths with results of other geophysical methods. For the present survey, six borings (BB-TMB-101, BB-TMB-102, BB-TMB-103, BB-TMB-104, BB-TMB-105, and BB-TMB-106) were drilled to refusal in the roadway within 30 feet north and 100 feet south of the bridge. The six borings encountered bedrock or refusal interpreted as bedrock at depths ranging from approximately 6.5 feet to 18.3 feet below the road surface. The locations of the borings along the seismic line and depth of bedrock determined from each boring are shown on the seismic profile in Figure 5.

Materials with two distinct velocity ranges were detected at the Site. The upper material

exhibits a velocity ranging from 1,600 to 2,000 feet per second (fps) and is interpreted to consist of unsaturated and saturated soils and sediments. The lower material exhibits a velocity range of 12,500 fps to 15,800 fps and is interpreted to be competent bedrock. Where the top of bedrock is highly fractured and/or deeply weathered, it might exhibit lower velocities that cannot be detected as a distinct layer on the basis of the seismic refraction data. Thus, the top of rock determined on the basis of seismic refraction data generally is the top of competent bedrock, which might be located somewhat below the geologic contact between the overburden and bedrock.

Comparison of seismic refraction data with results from the GPR survey are not possible due to the lack of GPR reflections from bedrock for most of the area surveyed, as stated above. Based on the good quality of the seismic data for this project and on the results from other similar seismic refraction surveys, we estimate the accuracy (standard deviation) of the apparent depths of competent bedrock determined by the seismic refraction survey in most locations to be about $\pm 10\%$ of the depth of bedrock, or ± 2 feet), whichever is greater.

The seismic profile shown in Figure 5 does show that bedrock is deeper in the vicinity of the existing bridge. The depth of competent bedrock along the seismic lines varies between about 6 and 16 feet below ground surface. The elevation of competent bedrock along the seismic refraction survey line varies between approximately 7 feet and -5 feet for a total relief of 12 feet. The elevation of the road at discrete points along the seismic profiles was determined from the plans supplied by Golder. Bedrock depths and elevations at station spacings of 5 feet along the seismic line are listed in Table 1.

The profile shown in Figure 5 represents a non-unique model for bedrock elevation, and the elevation of competent bedrock at any particular location between actual data points may differ from that shown. Bedrock elevations based on additional data, such as additional borings or seismic data, may differ significantly from those shown on the plates. **The bedrock model shown as profiles or listed as tabular data should not be used for contract bedrock removal quantities.**

CONCLUSIONS

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc. along a portion of Route 102 at Clark Bridge in Tremont, Maine in May, 2012, we conclude that:

- Depth of bedrock below the surveyed portion of the roadway varies between about 6 and 16 feet and is deepest in the vicinity of Clark Bridge
- A shallow possible buried structure, possibly the former timber span bridge, was

Geophysical Survey
Clark Bridge Replacement Project
State Route 102, Tremont, Maine
File 12SG06 Page 7

detected on the basis of GPR records

- Multiple linear features crossing the roadway, interpreted as the timber cribbing used in construction of the causeways, were detected on the basis of GPR records
- A buried surface, interpreted as the interface between the roadbed material and the boulder and cobble fill below, was detected on the basis of GPR records

LIMITATIONS

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

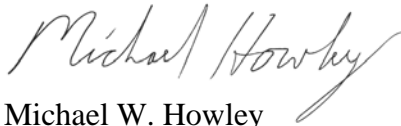
H-R has used reasonable care, skill, competence and judgment in the performance of its services for this project consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed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

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

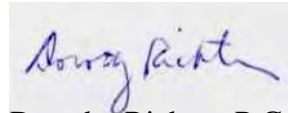
Geophysical Survey
Clark Bridge Replacement Project
State Route 102, Tremont, Maine
File 12SG06 Page 8

If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with you on this project. We look forward to working with you again in the future.

Sincerely yours,
HAGER-RICHTER GEOSCIENCE, INC.



Michael W. Howley
Geophysicist



Dorothy Richter, P.G.
President

Attachments: Table 1, Figures 1-5, Appendix A

Table 1
Depth and Elevation of Bedrock from Seismic Refraction
Clark Bridge Replacement Project
State Route 102, Tremont, Maine

Profile Distance (feet)	Easting (feet)	Northing (feet)	Bedrock Depth (feet)	Approximate Bedrock Elevation (feet)
0	1023668.1	213777.5	6.9	7.1
5	1023672.5	213775.0	7.2	6.3
10	1023676.8	213772.6	7.3	6.1
15	1023681.2	213770.2	7.5	5.8
20	1023685.6	213767.7	7.9	5.3
25	1023690.0	213765.3	8.0	5.1
30	1023694.3	213762.8	8.4	4.6
35	1023698.7	213760.4	8.4	4.4
40	1023703.1	213758.0	9.1	3.6
45	1023707.5	213755.5	10.1	2.4
50	1023711.8	213753.1	11.0	1.3
55	1023716.2	213750.7	12.1	0.1
60	1023720.6	213748.2	13.0	-1.0
65	1023725.0	213745.8	13.9	-2.1
70	1023729.3	213743.3	14.6	-3.1
75	1023733.7	213740.9	15.4	-4.1
80	1023738.1	213738.5	15.8	-4.8
85	1023742.4	213736.0	15.5	-4.5
90	1023746.8	213733.6	14.0	-3.0
95	1023751.2	213731.1	12.9	-1.9
100	1023755.6	213728.7	11.8	-0.8
105	1023759.9	213726.3	11.5	-0.5
110	1023764.3	213723.8	11.3	-0.3
115	1023768.7	213721.4	11.2	-0.2
120	1023773.1	213719.0	11.2	-0.2
125	1023777.4	213716.5	11.0	0.0
130	1023781.8	213714.1	10.6	0.4
135	1023786.2	213711.6	10.6	0.4
140	1023790.6	213709.2	10.4	0.6
145	1023794.9	213706.8	10.2	0.7
150	1023799.3	213704.3	9.8	1.1
155	1023803.7	213701.9	9.3	1.6

Table 1 continued				
Profile Distance (feet)	Easting (feet)	Northing (feet)	Bedrock Depth (feet)	Approximate Bedrock Elevation (feet)
160	1023808.0	213699.4	8.9	1.9
165	1023812.4	213697.0	8.5	2.3
170	1023816.8	213694.6	8.3	2.4
175	1023821.2	213692.1	8.0	2.7
180	1023825.5	213689.7	8.0	2.7
185	1023829.9	213687.2	7.7	2.9
190	1023834.3	213684.8	7.6	3.0
195	1023838.7	213682.4	7.4	3.2
200	1023843.0	213679.9	7.5	3.1
205	1023847.4	213677.5	7.3	3.3
210	1023851.8	213675.1	7.2	3.4
215	1023856.2	213672.6	7.2	3.4
220	1023860.5	213670.2	7.4	3.2
225	1023864.9	213667.7	7.4	3.3
230	1023869.3	213665.3	7.3	3.4
235	1023873.7	213662.9	7.3	3.5

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. Easting and Northing coordinates for the seismic lines are given in feet in Maine State Plane East 1801 (NAD1983), and were determined using differentially corrected GPS positions acquired at the time of the survey. Elevations are relative to the surface elevations determined from site plans supplied by Golder.

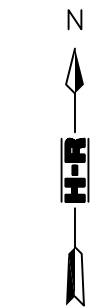
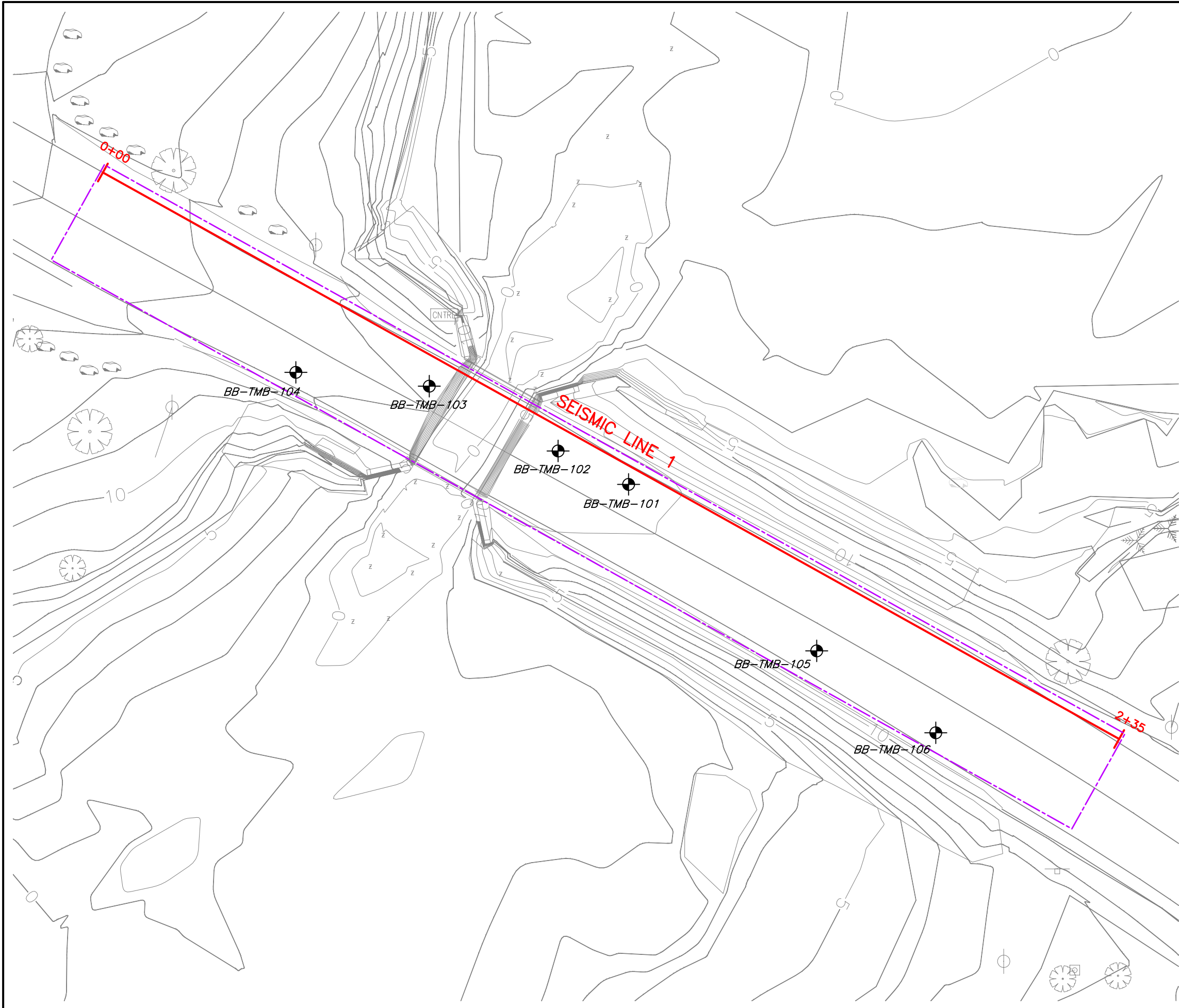


LOCATION




NOTE:

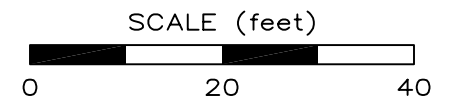
Modified from Google Earth aerial photograph.

<p>Figure 1 General Site Location Clark Bridge Replacement Project State Route 102, Tremont, Maine</p>	
File 12SG06	June, 2012
<p>HAGER-RIECHER GEOSCIENCE, INC. Salem, New Hampshire</p>	



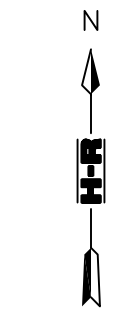
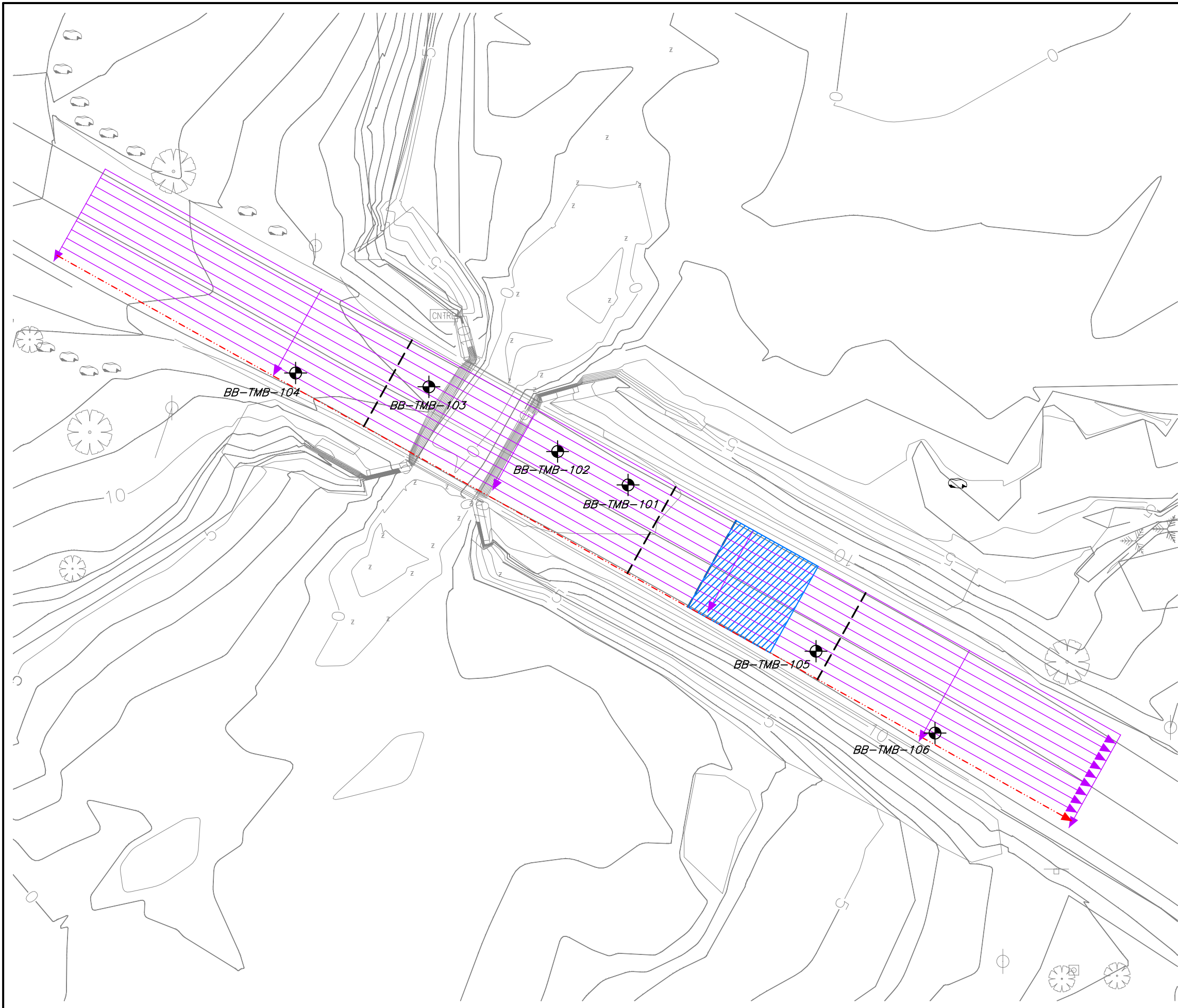
LEGEND

-  SEISMIC LINE
-  APPROXIMATE LIMITS OF GPR SURVEY AREA
-  BORING








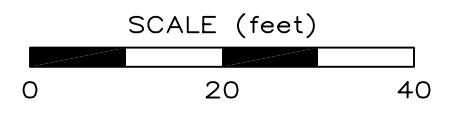
NOTE:
 Modified from site plan provided by Golder Associates, Inc., entitled 3DTopo_13Mar12.dwg.

Figure 2 Site Plan Clark Bridge Replacement Project State Route 102, Tremont, Maine	
File 12SG06	June, 2012
HAGER-RICHTER GEOSCIENCE, INC. Salem, New Hampshire	



LEGEND

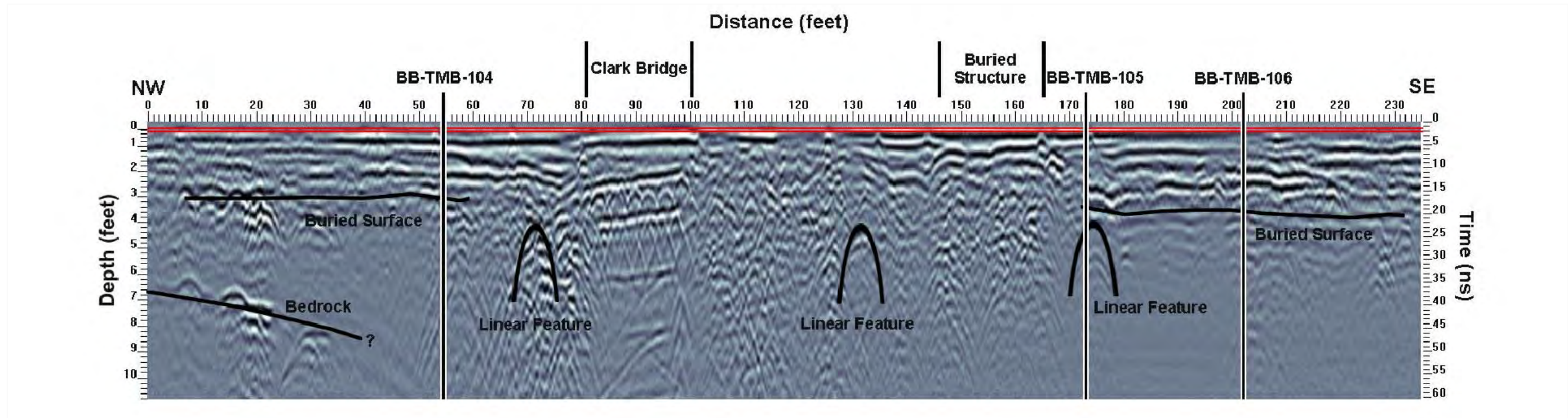
-  GPR TRAVERSE
-  GPR TRAVERSE SHOWN IN FIGURE 4
-  POSSIBLE LINEAR FEATURE
-  POSSIBLE BURIED STRUCTURE
-  BORING



NOTE:

Modified from site plan provided by Golder Associates, Inc., entitled 3DTopo_13Mar12.dwg.

<p>Figure 3 GPR Survey & Interpretation Clark Bridge Replacement Project State Route 102, Tremont, Maine</p>	
File 12SG06	June, 2012
<p>HAGER-RICHTER GEOSCIENCE, INC. Salem, New Hampshire</p>	



NOTES:

1. GPR data were acquired using a Sensors and Software Noggin Smart Cart digital GPR system with a 250 MHz antenna.
2. Estimated depths represent distance below ground surface
3. Locations of GPR records shown in Figure 3.

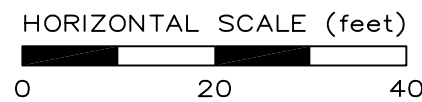
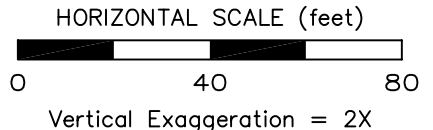
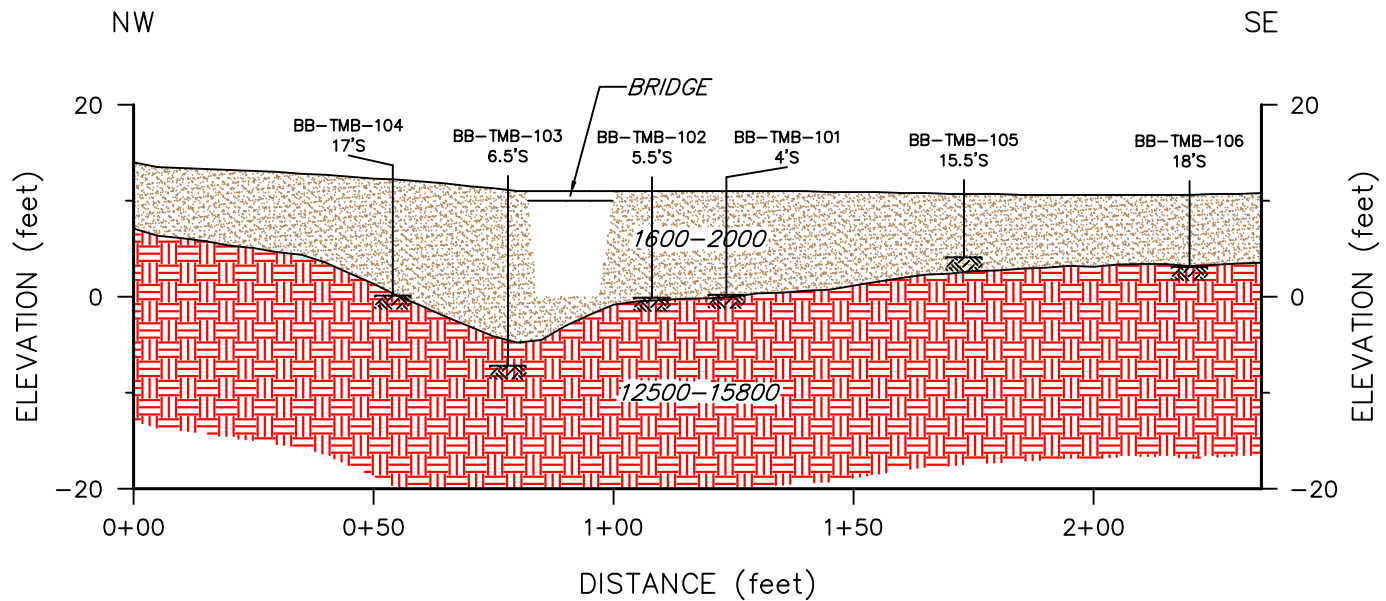


Figure 4
 Example GPR Record
 Clark Bridge Replacement Project
 State Route 102, Tremont, Maine

File 12SG06	June, 2012
HAGER-RICHTER GEOSCIENCE, INC. Salem, New Hampshire	



NOTES:

1. Estimated standard deviation of depth of bedrock is $\pm 10\%$ or 2 feet, whichever is greater.
2. The depths determined for bedrock are depths of competent rock; weathered and/or fractured bedrock might occur at shallower depths.
3. Surface elevations determined from plans provided by Golder Associates, Inc., entitled 3DTopo_13Mar12.dwg.
4. Data were analyzed using the Generalized Reciprocal Method.

LEGEND



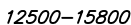

-  Unsaturated soils
-  Competent bedrock
-  Velocity (fps)
-  Boring with identification, distance from traverse, and depth of bedrock based on logs provided by Golder Associates, Inc.

Figure 5
Seismic Line
Clark Bridge Replacement Project
State Route 102, Tremont, Maine

File 12SG06 June, 2012

HAGER-RICHTER GEOSCIENCE, INC.
Salem, New Hampshire

APPENDIX A

Draft Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-101 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88?	Sampler: 2" and 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 2" SS, 300/16 3" SS
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9

Hammer Efficiency Factor: 0.6 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions:
D = Split Spoon Sample R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
W1P/W2P = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows				
0	1D	24/12	0.0 - 0.3 0.3 - 0.5 0.5 - 2.5	15/11/14/12	25	25	SSA	10.4		0-0.3 ft. Asphalt wearing course 0.3-0.5 ft. Asphalt base course Top 1" Weathered asphalt base course		
								8.0		Bottom 11": Light and dark brown, damp, medium dense, gravely fine to medium SAND, little silt, trace clay (Roadway Base).		
5	2MD	24/0	5.0 - 7.0	7/7/26/9	33	33				Lost water at 3 ft., cobble fill at 3.5 ft based on drill behavior. Intermittent water return to 5 ft. No recovery, resampled with 3" spoon. Gray, damp 3" cobble, little gravel, trace silt (Rock Fill). Boulder approximately 7 to 8 ft. based on drill behavior. Cobble fill below based on drill behavior. Intermittent water return 8 to 10 ft.		
								0.2		b=Used 3" split spoon. Gray, wet, coarse GRAVEL, trace coarse sand, trace silt (Rock Fill).		
10	3D _b	1/1	10.0 - 10.1	50-1"						Top of Bedrock Elev. ____ ft. Roller coned ahead to 12 ft. Seam based on drill behavior 11.8 to 11.9 ft. R1: Rock Mass Quality = Poor R1: Core Times (min:sec) 12.0-13.0': (2:55) 13.0-14.0': (2:30) 14.0-15.0': (2:15) 15.0-16.0': (2:40) 16.0-17.0': (3:30) Recovery = 100% R2: Rock Mass Quality = Poor R2: Core Times (min:sec) 17.0-18.0': (3:35) 18.0-19.0': (4:15) 19.0-20.0': (4:20) 20.0-21.0': (3:55) 21.0-21.2': Not timed. Recovery = 100%		
	R1	60/60	12.0 - 17.0	RQD = 42%			NQ-2	-10.2		Bottom of Exploration at 21.2 feet below ground surface.		
15												
20	R2	50/50	17.0 - 21.2	RQD = 50%								
25												

Remarks:

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

Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88?	Sampler: 2" and 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.8

Hammer Efficiency Factor: 0.6 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt W1P/W2P = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/80%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/12	0.0 - 0.4 0.4 - 0.5 0.5 - 2.5	19/10/11/10	-		SSA	10.3		0-0.4 ft. Asphalt wearing course. 0.4-0.5 ft Asphalt base course Top 3": Weathered asphalt base course		
	2D	1/0	2.5 - 2.6	50-1"			HW _a	8.5		Middle 8": Light brown, damp, sandy fine to coarse GRAVEL, little silt (Roadway Base). Bottom 2": Gray, damp, coarse GRAVEL, litte fine to coarse sand, little silt (Rock Fill).		
5	3D	24/0	5.0 - 7.0	7/10/8/1	-					No recovery. Rock fill based on drill behavior. HW _a = Advanced HW casing with spinning shoe. No blows performed. Lost water at 4 ft, loose based on drill behavior. No recovery, final blow drove spoon to 7.5 ft., hole stayed open for 4D. 6.5-7.5 ft: Loose based on drill behavior, lost water return. Gray, wet, fine to coarse GRAVEL, trace silt (Rock Fill). Dense at 8.5, lost water return.		
	4D	24/3	7.5 - 9.5	14/12/6/10	-					Gray and dark green, wet, fine to coarse GRAVEL, trace fine to coarse sand (Rock Fill).		
10	5D	13/4	10.0 - 11.1	10-26-50/1"				-0.1		Top of Bedrock at Elev. ____ ft. Roller cone ahead to 11.5 ft. Water returned with casing at 11.5 ft. R1: Rock Mass Quality: Poor R1: Core Times (min:sec) 11.5-12.5': (2:15) 12.5-13.5': (1:50) 13.5-14.5': (2:10) 14.5-15.5': (2:30) 15.5-16.5': (6:25) Recovery = 83% R2: Rock Mass Quality = Poor R2: Core Times (min:sec) 16.5-17.5': (1:50) 17.5-18.5': (3:35) 18.5-19.5': (2:50) 19.5-19.8': (0:30) Recovery = 98% Core barrel plugged at 19.8 ft, picked up 7" of R1. R3: Rock Mass Quality = Fair R3: Core Times (min:sec) 19.8-21.5': (4:15)		
	R1	60/50	11.5 - 16.5	RQD = 45%			NQ-2					
15	R2	40/39	16.5 - 19.8	RQD = 33%								
20	R3	19/17	19.8 - 21.4	RQD = 68%				-10.5				
25										Bottom of Exploration at 21.5 feet below ground surface.		

Remarks:


DRAFT

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: <u>BB-TMB-103</u> PIN: <u>19305.00</u>
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Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD887	Sampler: 2" and 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9

Hammer Efficiency Factor: 0.6 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions:
D = Split Spoon Sample R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_u(lab) = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
WIP/W2P = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/80%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.		
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows							
25	R2	60/27	25.0 - 30.0	RQD = 7%							R2: Rock Mass Quality = Very Poor R2: Core Times (min:sec) 25.0-26.0': (2:10) 26.0-27.0': (2:05) 27.0-28.0': (2:35) 28.0-29.0': (2:15) 29.0-30.0': (1:40) Recovery = 45%			
30								-19.0	30.0				Bottom of Exploration at 30.0 feet below ground surface.	
35														
40														
45														
50														

Remarks:

DRAFT

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-104 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 12.5	Auger ID/OD: 4.5" SSA/6.5" HSA
Operator: M. Porter	Datum: NAVD88?	Sampler: 2" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A
Boring Location: Not Surveyed	Casing ID/OD: N/A	Water Level*: Dry

Hammer Efficiency Factor: 0.6 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions:
D = Split Spoon Sample R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_u(lab) = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
W1P/W2P = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0			0.0 - 0.4 0.4 - 0.6 1.0 - 3.0	7/12/7/6	19	19	SSA	11.9	0-0.4 ft.: Asphalt wearing course. 0.4-0.6 ft.: Asphalt base course		
	1D	24/12							Light brown, damp, medium dense, fine to medium SAND, some gravel, some silt. Decreasing gravel with depth (Roadway Base).		
	2D	24/14	3.0 - 5.0	9/6/6/13	12	12	HSA		Light brown, damp, medium dense, fine to medium SAND, some silt (Roadway Base).		
5	3D	4/4	5.0 - 5.3	50-4"	-			7.1	Red-brown, damp, silty fine SAND, trace organics (roots) (Roadway Base).		
									Gravel and Rock fill at 5.4 ft. based on auger behavior. Auger cuttings contain decreasing sand fraction with depth.		
									Auger cuttings mostly broken rock/gravel at 8 ft.		
10	4D	1/0	10.0 - 10.1	50-1"					No recovery - dense rock fill.		
								0.4	Auger refusal at 12.1 ft, very hard.		
									Bottom of Exploration at 12.1 feet below ground surface.		
15											
20											
25											

DRAFT

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-105
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 10.5	Auger ID/OD: 4.5" SSA/6.5" HSA	
Operator: M. Porter	Datum: NAVD88?	Sampler: 2" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon	
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A	
Boring Location: Not Surveyed	Casing ID/OD: N/A	Water Level*: Dry	
Hammer Efficiency Factor: 0.6	Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>		

Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing W1P/W2P = Weight of one/two person(s)	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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0			0.0 - 0.4									
	1D	24/8	0.4 - 0.7 1.0 - 3.0	19/21/15/13	36	36			9.5	0-0.4 ft.: Asphalt wearing course 0.4-0.7 ft.: Asphalt base course		
	2D	24/6	3.0 - 5.0	15/10/6/8	16	16				Top 1": Weathered asphalt base course Bottom 7": Light brown and gray, damp, dense GRAVEL, some fine to coarse sand, trace silt (Roadway Base). Light brown and gray, fine to coarse sandy GRAVEL, trace silt. Spoon tip plugged with gravel (Roadway Base).		
5	3D	8/4	5.0 - 5.7	5/50-2"	-				5.5	Brown, wet, coarse GRAVEL, some fine sand, little silt (Rock Fill)		
									3.9	Auger refusal at Elev. ____ ft.		
										Bottom of Exploration at 6.6 feet below ground surface.		
10												
15												
20												
25												

Remarks:



Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-106 PIN: 19305.00
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Driller: Maine Test Borings	Elevation (ft.): 10.5	Auger ID/OD: 4.5" SSA/6.5" HSA
Operator: M. Porter	Datum: NAVD88?	Sampler: 2" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 Split Spoon
Date Start/Finish: 5/16/12	Drilling Method: Auger boring	Core Barrel: N/A
Boring Location: Not Surveyed	Casing ID/OD: N/A	Water Level*: 8.9

Hammer Efficiency Factor: 0.6 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_u(lab) = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt W1P/W2P = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0			0.0 - 0.6						9.5	0-0.6 ft.: Asphalt wearing course		
		24/14	0.6 - 0.8							0.6-0.8ft.: Asphalt base course		
			1.0 - 3.0	16-23-18-17	41	41				Light brown-gray, damp, dense, fine to medium SAND, some gravel, little silt (Roadway Base).	1.0	
		24/12	3.0 - 5.0	16-11-6-3	17	17				Brown, damp, medium dense, fine to medium SAND, some silt, little gravel (Roadway Base).		
5		24/15	5.0 - 7.0	3/3/4/32	7	7	HSA		5.5	Dark gray, wet, loose, SILT, little clay, little fine sand, trace organics (bark and roots) (Emerged Marine Sediments).	5.0	
									3.7			
									3.0	Gravel/broken rock based on auger behavior. Auger Refusal at Elev. ___ ft.	6.8	
										Bottom of Exploration at 7.5 feet below ground surface.	7.5	
10												
15												
20												
25												

Remarks:

APPENDIX G
RMR AND GSI CALCULATIONS

Golder Associates

SUBJECT ESTIMATE OF SITE-WIDE RMR AND GSI, CLARK BRIDGE REPLACEMENT, TREMONT, ME

Job No. 123-87440
Ref. 2

Made by JRS
Checked JOK
Reviewed WSP

Date 7/13/12
Sheet 1 of 5

PROBLEM: ESTIMATE SITE-WIDE ROCK MASS RATING (RMR) AND GEOLOGIC STRENGTH INDEX (GSI) FOR CLARK BRIDGE REPLACEMENT PROJECT USING GEOLOGIC FIELD MAPPING DATA, ROCK CORE LOGS, LABORATORY TESTING, AND PUBLISHED REPORTS. TWO LITHOLOGIES ARE PRESENT: 1) GRANITE GNEISS AS EXPOSED IN OUTCROPS ON EAST SIDE OF CENTERLINE OF MARSHALL BROOK AND IN BB-TMB-101; AND 2) META VOLCANICS IN BORINGS 13B-TMB-102 AND BB-TMB-103 ON WEST SIDE OF CENTER LINE MARSHALL BROOK (NO BEDROCK EXPOSURE OF THIS LITHOLOGY).

- REFERENCES:
- 1) GILMAN, R.A., CHAPMAN, C.A., LOWEL, T.V. AND BURNS, H.W. JR., 1988. THE GEOLOGY OF MOUNT DESERT ISLAND - A VISITOR'S GUIDE TO THE GEOLOGY OF ACADIA NATIONAL PARK. MAINE GEOLOGICAL SURVEY, BULL. 38, 50 P.
 - 2) GRAHAM, J., 2010. ACADIA NATIONAL PARK: GEOLOGIC RESOURCES INVENTORY REPORT. NATURAL RESOURCE REPORT NPS/NRPL/GRO/NRR-2010/232, NATIONAL PARK SERVICE, FT. COLLINS, CO., 63 P.
 - 3) LUBICK, N., 1994. THE SILURIAN (?) GRANBERY ISLAND VOLCANIC SERIES ON MT. DESERT ISLAND, MAINE. IN WOODWARD, H.H., ED. THE KEEK GEOLOGY CONSORTIUM, 7th KEEK SYMPOSIUM VOLUME, BELMONT COLLEGE, P. 25-28.
 - 4) BIEMIAWSKI, Z.T., 1989. ENGINEERING ROCK MASS CALCULATIONS, JOHN WILEY + SONS, NY, 251 P. (ATTACHMENT B)
 - 5) WYLLIE, D.C., 1999. FOUNDATIONS ON ROCK, 2nd ED., IE + FAN SPON PRESS, LONDON, 401 P. (ATTACHMENT E)
 - 6) WYLLIE, D.C. AND MAH, C., 2004. ROCK SLOPE ENGINEERING, 4th ED., SPON PRESS, NY, 431 P.
 - 7) MARINOS, V., MARINOS, P. AND HOCK, E., 2005. THE GEOLOGIC STRENGTH INDEX: APPLICATIONS AND LIMITATIONS. BULLETIN OF ENGINEERING GEOLOGY AND ENVIRONMENT, VOL. 64, P. 55-65. (ATTACHMENT D)
 - 8) GOLDER ASSOCIATES INC., JULY 2012. GEOLOGIC MAPPING FIELD NOTES, BORING LOGS, DATA TABLES AND LABORATORY TESTING RESULTS, CLARK BRIDGE REPLACEMENT PROJECT FILES, NO. 123-87440. (ATTACHMENT A)
 - 9) GOLDER ASSOCIATES INC., JULY 12, 2012. RMR CALCULATIONS FOR ROCK CORE RUNS, CLARK BRIDGE, TREMONT, ME.

GEOLOGY SUMMARY:

TWO LITHOLOGIES ARE PRESENT, BOTH OCCURRING WITHIN THE UPPER MEMBER OF THE SILURIAN AND CRANBERRY ISLAND VOLCANIC SERIES (REFS. 1+2). THESE CONSIST OF A UNIFORM PINK, VERY FINE TO FINE-GRAINED GRANITE (I.E. A FELSITE), AND GRAY TO BROWN DACITE LAVA FLOWS (I.E. META VOLCANICS) (REF. 3). AS THESE TWO LITHOLOGIES HAVE DIFFERENT GEOTECHNICAL PROPERTIES, THEY WILL BE EVALUATED SEPARATELY FOR THESE CALCULATIONS.

PROCEDURE:

FOLLOW METHODS PRESENTED IN REFS. 4-7 TO ESTIMATE RMR AND GSI. USE SITE DATA FROM REF. 8+9. SEE ATTACHMENT A FOR SITE DATA.

RMR: PART A - CLASSIFICATION PARAMETERS AND RATINGS (REF. 4-6)

A-1) STRENGTH OF INTACT ROCK:

GRANITE GUESS: FIELD ESTIMATES OF INTACT ROCK STRENGTH ARE R5 IN OUTCROPS (15,000 - 36,000 PSI / 100 - 250 MPa) AND R4 IN ROCK CORE (7,000 - 15,000 PSI / 50 - 100 MPa). UCS LABORATORY DATA (1 TEST) INDICATE INTACT ROCK STRENGTH OF 31,567 PSI (217.7 MPa). PLT LABORATORY DATA (2 TESTS) INDICATE INTACT ROCK STRENGTH OF 26,800 AND 37,000 PSI (184.8 AND 255.1 MPa), AVERAGE OF 31,900 PSI (220 MPa). SUMMARY:

DATA SOURCE	FIELD DESCRIPTION	PSI	MPa
OUTCROP	R5	15,000 → 36,000	100 → 250 MPa
ROCK CORE	R4	7,000 → 15,000	50 → 100 MPa
UCS	n/s	31,567	217.7
PLT	n/s	26,800 → 37,000	184.8 → 255.1
\bar{X}_{UCS}	n/s	31,567	217.7
\bar{X}_{PLT}	n/s	31,900	220 MPa

RATING: CHOOSE RATING OF (14) BASED ON LAB AND OUTCROP DATA

META VOLCANICS: FIELD ESTIMATES OF INTACT ROCK STRENGTH (ROCK CORE ONLY) ARE R3 TO R4 (3,500 → 15,000 PSI / 25 → 100 MPa). UCS LABORATORY DATA (2 TEST) INDICATE INTACT ROCK STRENGTH OF 13,189 PSI (91.0 MPa). PLT LABORATORY DATA (2 TESTS) INDICATE INTACT ROCK STRENGTH OF 11,900 AND 18,800 PSI (82.1 AND 129.7 MPa), AVERAGE OF 15,350 PSI (105.9 MPa). SUMMARY:

Golder Associates

SUBJECT ESTIMATE OF SITE-WIDE RMR AND GSI, CLARK BRIDGE REPLACEMENT, TREMONT, ME

Job No. 123-1740

Made by JRS

Date 7/13/12

Ref. 2

Checked JOL

Sheet 3 of 5

Reviewed MSP

PROCEDURES:	DATA SOURCE	FIELD DESCRIPTION	PSI	MPa
(CONTINUED)	OUTCROP	n/a	n/a	n/a
	ROCK CORE	R3 → R4	3,500 → 15,000	25 → 100
	UCS	n/a	13,189	91.0
	PLT	n/a	11,900 → 18,800	82.1 → 129.7
	\bar{X}_{UCS}	n/a	13,189	91.0
	\bar{X}_{PLT}	n/a	15,350	105.9

RATING: CHOOSE RATING OF (9) BASED ON FIELD AND LABORATORY DATA.

A-2) RQD: GRANITE GNEISS: 2 BORINGS (BB-TMB-101) ARE 42% AND 50%, FOR AVERAGE OF 46%. RATING = (9)

META VOLCANICS: 2 BORINGS (BB-TMB-102/102) ARE 45%, 33%, 65%, 32% AND 7%, FOR AVERAGE OF 36.4%. RATING = (7)

A-3) JOINT SPACING: GRANITE GNEISS: ROCK OUTCROP MAPPING DATA INDICATE JOINT SPACING RANGES FROM 0.1 TO 5 FT (3 TO 152 cm); ROCK CORE DATA INDICATE JOINT SPACING RANGES FROM VERY CLOSE TO CLOSELY SPACED (<5 TO 30 cm). OUTCROP DATA AVERAGE IS 1.131 FT (34.5 cm). RATING FOR 25 cm SPACING = (8)

META VOLCANICS: ROCK CORE DATA INDICATE JOINT SPACING RANGES FROM VERY CLOSE TO CLOSELY SPACED (<5 TO 30 cm). NO OUTCROP EXPOSED, SO NO OUTCROP DATA. RATING FOR 20 cm SPACING = (8)

A-4) JOINT CONDITION: GRANITE GNEISS: ROCK OUTCROP MAPPING DATA INDICATE MOST JOINTS ARE SMOOTH, SOME ROUGH AND RARELY VERY ROUGH. JOINTS ARE MAINLY PLANAR, SOME CURVED/IRREGULAR PLANAR. INFILLING IS RARE (CHLORITE, SAND/GRAVEL, QUARTZ). APERTURE RANGES FROM <0.1 TO 2 INCH (AVG. 0.154 INCH OR 3.9 mm). ROCK IS SLIGHTLY WEATHERED. BOREHOLE DATA INDICATE ROCK IS MODERATELY TO SLIGHTLY WEATHERED, AND JOINTS SMOOTH TO VERY ROUGH, AND PLANAR. BASED ON THIS DESCRIPTION CHOOSE RATING = (22)

META VOLCANICS: NO ROCK OUTCROP DATA BOREHOLE DATA INDICATE ROCK IS MODERATELY TO SLIGHTLY WEATHERED, AND CONTAINS HIGHLY FRACTURED ZONES. JOINTS ARE SMOOTH TO VERY ROUGH, CONTAIN SOFT MINERALIZATION (E.G. TALC, CHLORITE), AND ARE PLANAR, IRREGULAR AND CURVED IN SHAPE. BASED ON THIS DESCRIPTION CHOOSE RATING = (15)

PROCEDURES: A-5) GROUNDWATER : FOR BOTH LITHOLOGIES, FOUNDATIONS WILL BE BELOW GROUNDWATER / RIVER LEVEL. SO CHOOSE RATING = 4 (CONTINUED)

RMR: PART B - RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

B) GRANITE GUESS DISCONTINUITY MAPPING FROM OUTCROPS INDICATES THREE PRIMARY JOINT SETS EXIST, ALONG WITH TWO SECONDARY SETS (SEE REF. 8). SUMMARY:

SET	CLASS	DIP	DIP DIRECTION	VERTICALITY
1m	PRIMARY	86	315	NEAR VERTICAL
2m	PRIMARY	85	204	NEAR VERTICAL
3m	SECONDARY	76	261	STEEP
4m	SECONDARY	77	182	STEEP
5m	PRIMARY	02	065	NEAR HORIZONTAL

SETS 1m, 2m AND 5m ARE NEARLY ORTHOGONAL AND FORM A BLOCKY TEXTURE TO ROCK MASS. SETS 3m AND 4m ARE STEEP AND COULD PROVIDE PLANAR OR WEDGE FAILURE.

BORRHOLE DATA INDICATE ORTHOGONAL SETS MOSTLY OBSERVED (I.E., VERTICAL AND HORIZONTAL) IN GRANITE, WITH RARE STEEP SETS.

BASED ON THESE OBSERVATIONS, CHOOSE FAIR CONDITIONS FOR FOUNDATIONS, SO RATING = 7

META VOLCANICS: BORRHOLE LOGS INDICATE STEEPLY INCLINED JOINTS ARE PRESENT (SECONDARY SETS 3m, 4m), IN ADDITION TO ORTHOGONAL SETS (1m, 2m + 5m). THEREFORE CONSIDER UNFAVORABLE JOINT ORIENTATION CONDITIONS, SO RATING = -15

RMR SUMMARY:

$$RMR_{\text{GRANITE}} = \sum 14 + 9 + 8 + 22 + 4 - 7 = 50$$

$$RMR_{\text{META VOLCANICS}} = \sum 9 + 7 + 8 + 15 + 4 - 15 = 28$$

THESE RESULTS COMPARE FAVORABLY WITH RMR ESTIMATES FROM ROCK CORE EVALUATION (REF. 9).

PROCEDURES:
(CONTINUED)

GSI CLASSIFICATION: TWO INPUT PARAMETERS ARE USED FOR ESTIMATE (REF. 7)

GRANITE GNEISS: 1) STRUCTURE: ROCK MASS IS BLOCKY, DEFINED BY TWO NEAR VERTICAL JOINT SETS, ONE NEAR HORIZONTAL BASAL JOINT SET, AND TWO STEEP SECONDARY SETS (REF. 8). ROCK MASS TYPE DEFINED AS VERY BLOCKY - INTERLOCKED, PARTIALLY DISTURBED MASS WITH MULT-FACETED ANGULAR BLOCKS FORMED BY 4 OR MORE JOINT SETS.

2) JOINT SURFACE CONDITIONS: JOINTS EXPOSED IN OUTCROP ARE MOSTLY SMOOTH, AND WALL ROCK IS FRESH (UNWEATHERED) TO MODERATELY WEATHERED. BDR ROCK DATA INDICATE SMOOTH TO VERY ROUGH JOINTS, AND SLIGHTLY TO MODERATELY WEATHERED WALL ROCK. CONSIDER FAIR CONDITIONS - SMOOTH, MODERATELY WEATHERED AND ALTERED SURFACES.

∴ GSI RANGE 36 → 54, SAY AVG. = **45**

METAVOLCANICS:

1) STRUCTURE: NO OUTCROP DATA SO MUST RELY ON BOREHOLE DATA: ALL 5 JOINT SETS APPARENT IN CORE, INCLUDING HIGHLY FRACTURED ZONES. LOWER RQD INDICATES DECREASING INTERLOCKING, SO CHOOSE VERY BLOCKY/BLOCKY - DISTURBED - SEAMY CATEGORIES - ANGULAR BLOCKS FORMED BY MANY DISCONTINUITY SETS.

2) JOINT SURFACE CONDITIONS: SOME JOINTS CONTAIN WEAK MINERALS (CHLORITE, TALC), AND ARE HIGHLY WEATHERED. CONSIDER POOR CATEGORY JOINT SURFACE CONDITION.

∴ GSI RANGE OF 27 → 35, SAY AVG. = **31**

SEE ATTACHMENT D FOR SUMMARY SHEET OF GSI.

NOTE: RMR/GSI RATINGS ARE BASED ON SITE-WIDE OBSERVATIONS, RMR AND GSI SHOULD BE REFINED FOR EACH FOUNDATION STRUCTURE ONCE DETAILED DESIGN IS COMPLETE.

Table 4: Summary of Laboratory Rock Core Testing Results

Preliminary Geotechnical Investigation

Clark Bridge

Tremont, Maine

MaineDOT Win: 19305.00

DRAFT

Test Boring Designation ¹	Existing Ground Surface Elevation ² (ft-msl)	Top of Bedrock Elevation ^{2,3} (ft-msl)	Sample Number	Approximate Sample Depth Below Top of Bedrock (ft)	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation (ft-msl)	Uniaxial Compressive Strength ⁴ (psi)	Point Load Index Test ⁵ (psi)
BB-TMB-101	11.2	0.4	R1	2.5 to 3.1	13.3 to 13.9	-2.1 to -2.7	-	26,800
			R2	7.4 to 7.8	18.2 to 18.6	-7.0 to -7.4	31,567	-
			R2	8.0 to 8.4	18.8 to 19.2	-7.6 to -8.0	-	37,000
BB-TMB-103	11.9	-6.4	R1	8.1 to 8.5	19.2 to 19.6	-7.8 to -8.2	13,189	-
			R2	2.5 to 2.9	20.8 to 21.2	-8.9 to -9.3	-	11,900
				2.9 to 3.4	21.2 to 21.7	-9.3 to -9.8	-	18,800

GRANITE GUESS

Notes:

- Test boring locations are shown on Figure 2, Boring Location Plan.
- As-drilled locations and elevations surveyed by VHB. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
- Top of bedrock refers to rock based on depth to casing refusal or observed rock surface in core barrel. Notes concerning weathered rock, if present, are provided on the boring logs.
- Test performed based on ASTM D7012 - Compressive Strength and Elastic Moduli on Intact Rock Core Specimens Under Conditions of Stress and Temperatures.
- Test performed based on ASTM D3988 - Point Load Test Type. Value listed is the estimated compressive strength based on the point load index determined from the test.
- Laboratory testing performed by Express, Inc.
- Laboratory test results are provided in Appendix D.
- ft = feet; psi = pounds per square inch; msl = mean sea level.

METAVOLCANICS

$\bar{X} = 22,378 \text{ psi} \quad (154.3 \text{ MPa})$
 $\bar{X}_{\text{GRANITE}} = 31,567 \text{ psi} \quad (217.7 \text{ MPa})$
 $\bar{X}_{\text{VOLCANICS}} = 15,350 \text{ psi} \quad (105.9 \text{ MPa})$

Prepared By: _____
 Checked By: _____
 Reviewed By: _____

ATTACHMENT A
 REF. 8
 A-1/18



Table 2: Summary of Rock Core Quality
 Preliminary Geotechnical Data Report
 Clark Bridge
 Tremont, Maine
 MaineDOT WIN: 19305.00

DRAFT

Test Boring Designation	Existing Ground Surface Elevation ¹ (ft-msl)	Run			TCR ²		RQD ³		Intact Rock						
		No.	Depth Below Ground Surface (ft)		Length (ft)	Length (ft)	Percent	Length (ft)	Percent	Designation	Weathering	Strength	Rock Mass Rating ⁴	Description	
			Start	End											
BB-TMB-101	11.2	R1	12.0	17.0	5.0	5.0	100%	2.1	42%	Peer	Moderately Weathered to Slightly Weathered	R4		12.0 to 2.9 ft-bgs: Dark gray to greenish-black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular. 12.9 to 17.0 ft-bgs: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very close spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide.	
			R2	17.0	21.2	4.2	4.2	100%	2.1	50%	Poor	Moderately Weathered to Slightly Weathered	R4		Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very close spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft-bgs, approximately 1 inch thick and subhorizontal.
			R1	11.5	16.5	5.0	4.2	83%	2.3	45%	Poor	Moderately Weathered to Slightly Weathered	R4		Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
BB-TMB-102	11.4	R2	16.5	19.8	3.3	3.2	98%	1.1	33%	Poor	Moderately Weathered to Slightly Weathered	R4		Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Calcite veins up to 1/4 inch thick common. Highly fractured zone 17.5 to 18 ft-bgs.	
			R3	19.8	21.5	1.7	1.4	85%	1.1	65%	Fair	Moderately Weathered to Slightly Weathered	R4		Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
			R1	20.0	25.0	5.0	3.0	60%	1.6	32%	Poor	Moderately Weathered to Slightly Weathered	R3		Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered, highly fractured, discontinuities close to very close spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating.
BB-TMB-103	11.9	R2	25.0	30.0	5.0	2.3	45%	0.3	7%	Very Poor	Moderately Weathered to Slightly Weathered	R3		Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs.	

- Notes:
- As-drilled elevations surveyed by VHB.
 - TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
 - RQD = rock quality designation. RQD is the total length of core pieces recovered with a length greater than or equal to twice the core diameter (4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length.
 - Rock Mass Rating System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. For the rating adjustment for joint orientation, values of Fair (-7) were typically assigned for gently dipping, Fair to Unfavorable (-11) for moderately to steeply dipping, and Unfavorable (-15) for steeply dipping discontinuity angles observed in the core samples. Correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table. Reference Appendix C for the RMR calculations.
 - ft=feet, msl=mean sea level

Prepared By: MRH
 Checked By:
 Reviewed By:



Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-101
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 2" and 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fail: 140/30 2" SS, 300/16 3" SS	
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: 11W	Water Level*: 8.9	

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_u(lab) = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N_{uncorrected} = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = Weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WIP/WZP = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/60%) * N_{uncorrected} C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N _{uncorrected}	N ₆₀	Casing Blows				
0	1D	24/12	0.0 - 0.3 0.3 - 0.5 0.5 - 2.5	15/11/14/12	25	25	SSA	10.4		0-0.3 ft. Asphalt wearing course 0.3-0.5 ft. Asphalt base course Top 1": Weathered asphalt base course	
								8.0		Bottom 11": Light and dark brown, damp, medium dense, gravelly fine to medium SAND, little silt, trace clay (Roadway Base).	
5	2MD	24/0	5.0 - 7.0	7/7/26/9	33	33				a = Advanced casing with 11W spinning shoe. Lost water at 3 ft., cobble fill at 3.5 ft based on drill behavior. Intermittent water return to 5 ft. No recovery, resampled with 3" spoon. Gray, damp 3" cobble, little gravel, trace silt (Rock Fill). Boulder approximately 7 to 8 ft. based on drill behavior. Cobble fill below based on drill behavior. Intermittent water return 8 to 10 ft.	
10	3D	1/1	10.0 - 10.1	50(1") _b				0.2		b = Used 3" split spoon. Gray, wet, coarse GRAVEL, trace coarse sand, trace silt (Rock Fill).	
	R1	60/60	12.0 - 17.0	RQD = 42%				10.8		Top of Bedrock Elev. _____ ft. Roller coned ahead to 12 ft. Seam based on drill behavior 11.8 to 11.9 ft. R1: 12.0-12.9': Dark gray to greenish black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular.	
15										12.9-17.0': Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Rock Mass Quality = Poor	
20	R2	50/50	17.0 - 21.2	RQD = 50%						R1: Core Times (min:sec) 12.0-13.0': (2:55) 13.0-14.0': (2:30) 14.0-15.0': (2:15) 15.0-16.0': (2:40) 16.0-17.0': (3:30) Recovery = 100% R2: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft., approximately 1 inch thick and subhorizontal. Rock Mass Quality = Poor	
25								-10.2			

Remarks:

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

A-3/18

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-101
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 2" and 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 2" SS, 300/16 3" SS	
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>	

Definitions
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample attempt
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Insitu Vane Shear Test attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. hammer
 WOR/C = Weight of rods or casing
 W1P/W2P = Weight of one/two person(s)
 S_v = Insitu Field Vane Shear Strength (psf)
 T_v = Pocket Torvane Shear Strength (psf)
 q_u = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw field SPT N-value
 Hammer Efficiency Factor = Annual Calibration Value
 N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
 S_{u(tab)} = 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 Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
25										R2: Core Times (min:sec) 17.0-18.0': (3:35) 18.0-19.0': (4:15) 19.0-20.0': (4:20) 20.0-21.0': (3:55) 21.0-21.2': Not timed. Recovery = 100% Bottom of Exploration at 21.2 feet below ground surface.	
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Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-102
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS	
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.8	

Hammer Efficiency Factor: 0.6
Hammer Type: Automatic Hydraulic Rope & Cathed

Definitions:
 R = Rock Core Sample
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample attempt
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. hammer
 WOR/C = Weight of rods or casing
 WIP/W2P = Weight of one/two person(s)

S_u = Insitu Field Vane Shear Strength (psf)
 T_v = Pocket Torvane Shear Strength (psf)
 q_u = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw field SPT N-value
 Hammer Efficiency Factor = Annual Calibration Value
 N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 N₆₀ = (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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.0 - 0.4 0.4 - 0.5 0.5 - 2.5	19/10/11/10	-		SSA	10.3	0-0.4 ft. Asphalt wearing course. 0.4-0.5 ft. Asphalt base course 3 inch spoon: Top 3": Weathered asphalt base course		
	2D	1/0	2.5 - 2.6	50(1")			HW _a	8.5	Middle 8": Light brown, damp, sandy fine to coarse GRAVEL, little silt (Roadway Base). Bottom 2": Gray, damp, coarse GRAVEL, little fine to coarse sand, little silt (Rock Fill).		
5	3D	24/0	5.0 - 7.0	7/10/8/1	-				3 inch spoon: No recovery. Rock fill based on drill behavior. HW _a = Advanced HW casing with spinning shoe. No blows performed. Lost water at 4 ft. loose based on drill behavior.		
	4D	24/3	7.5 - 9.5	14/12/6/10	-				3 inch spoon: No recovery, final blow drove spoon to 7.5 ft., hole stayed open for 4D 6.5-7.5 ft. Loose based on drill behavior, lost water return. Gray, wet, fine to coarse GRAVEL, trace silt (Rock Fill). Dense at 8.5, lost water return. Gray and dark green, wet, fine to coarse GRAVEL, trace fine to coarse sand (Rock Fill).		
10	5D	13/4	10.0 - 11.1	10/26/50(1")				-0.1			
	R1	60/50	11.5 - 16.5	RQD = 45%			NO-2		Top of Bedrock at Elev. _____ ft. Roller cone ahead to 11.5 ft. Water returned with casing at 11.5 ft. R1: Dark gray to greenish-back, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality: Poor R1: Core Times (min:sec) 11.5-12.5: (2:15) 12.5-13.5: (1:50) 13.5-14.5: (2:10) 14.5-15.5: (2:30) 15.5-16.5: (6:25) Recovery = 83%		
15	R2	40.4/39	16.5 - 19.9	RQD = 33%							
20	R3	20/17	19.8 - 21.5	RQD = 68%				-10.5			
25											

Remarks:

A-5/18

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-102
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS	
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: 11W	Water Level*: 8.8	

Hammer Efficiency Factor: 0.6

Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample, SSA = Solid Stem Auger, HSA = Hollow Stem Auger, RC = Roller Cone, WOH = Weight of 140lb hammer, WOR/C = Weight of rods or casing, W1P/W2P = Weight of one/two person(s)
 S_u = Instu 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, N₆₀ = SPT N-uncorrected corrected for hammer efficiency, N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
 D = Split Spoon Sample, MD = Unsuccessful Split Spoon Sample attempt, U = Thin Wall Tube Sample, MU = Unsuccessful Thin Wall Tube Sample attempt, V = Instu Vane Shear Test, PP = Pocket Penetrometer, MV = Unsuccessful Instu Vane Shear Test attempt
 S_{u(fab)} = Lab Vane Shear Strength (psf), WC = water content, percent, LL = Liquid Limit, PL = Plasticity Limit, PI = Plasticity Index, G = Grain Size Analysis, C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25											17.5-18.5': (3:35) 18.5-19.5': (2:50) 19.5-19.8': (0:30) Recovery = 98% Core barrel plugged at 19.8 ft, picked up 7" of R1. R3: Dark gray to greenish-back, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality = Fair R3: Core Times (min:sec) 19.8-21.5': (4:15)	
30												
35												
40												
45												
50												

Remarks:

A-6/18

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-103
Location: Tremont, Maine		PIN: 19305.00	
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS	
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	

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, PP = Pocket Penetrometer
 MV = Unsuccessful Insitu Vane Shear Test attempt

R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb hammer
 WOR/C = Weight of rods or casing
 W1P/W2P = Weight of one/two person(s)

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
 N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 N₆₀ = (Hammer Efficiency Factor/10%) * 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows / (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	ID	24/12	0.0 - 0.3 0.3 - 0.5 0.5 - 2.5	26/12/11/22	-		SSA	10.3	Asphalt wearing course. Asphalt base course. 3 inch spoon: Top 3": Weathered asphalt base course.		
								8.5	Bottom 8": Light brown, moist, fine to coarse SAND, little fine gravel, little silt (Roadway Fill).		
5	2D	24/3	5.0 - 7.0	6/10/28/11	-		HW _a	2.5	Gravel and cobbles based on drill behavior. Lost water return at 3 ft. Advanced HW casing with spinning shoe. 3 inch spoon: Gray, damp, single 3" cobble, trace silt. Spoon bent during driving (Rock Fill). No return, cobbles based on drill behavior.		
10								2.0			
								0.6			
	3D	24/10	11.0 - 13.0	9/12/6/13	-		V	0.0	Wood in wash at approximately 9 to 10.3 ft. No return at 10.4 ft.		
								0.0	3 inch spoon: Gray, wet, fine to coarse GRAVEL, little fine to coarse sand, little silt, trace clay (Emerged Marine Sediments).		
	4D	24/7	13.0 - 15.0	48/39/50(5")	-		44 _b		3 inch spoon: Gray, wet, fine to coarse GRAVEL, some fine to coarse sand, little silt (Emerged Marine Sediments).		
15							96		h = Drove NW casing Wash turns red-gray at approximately 14.5 ft., alternates to blue-gray.		
	5D	6/6	15.0 - 15.5	110(6")			HW _a		3 inch spoon: Blue-gray, wet, fine sandy SILT, little fine gravel (Emerged Marine Sediments). HW Casing bent at bottom, rods not fitting through NW casing. Driller removed NW casing and resumed spinning HW.		
								-7.3			
20									Top of Bedrock at Elev. ____ ft. Roller cone ahead of 20 ft. Casing not seating well at 19 ft. R1: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs. Rock Mass Quality = Poor R1: Core Times (min:sec) 20.0-21.0': (2.05)		
	R1	60/36	20.0 - 25.0	RQD = 32%			NO-2				
25											


Remarks:

A-7/18

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-103
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS	
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathode

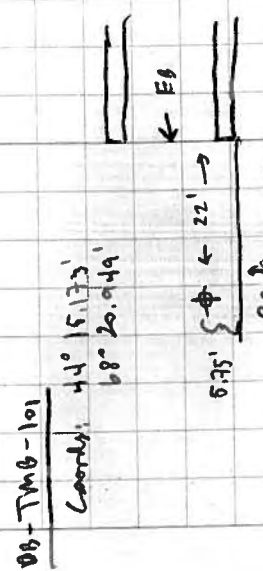
Definitions:
 D = Split Spoon Sample R = Rock Core Sample S_v = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = Weight of 140lb hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = Weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 WIP/WZP = Weight of one/two person(s) N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	R2	60/27	25.0 - 30.0	RQD = 7%				-19.0		21.0-22.0': (2:20) 22.0-23.0': (2:10) 23.0-24.0': (2:40) 24.0-25.0': (5:30) Recovery = 60% R2: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs. Rock Mass Quality = Very Poor R2: Core Times (min:sec) 25.0-26.0': (2:10) 26.0-27.0': (2:05) 27.0-28.0': (2:35) 28.0-29.0': (2:15) 29.0-30.0': (1:40) Recovery = 45%	
30										Bottom of Exploration at 30.0 feet below ground surface.	
35											
40											
45											
50											

Remarks:

A-8/18

5/17/12 Thurs 749 123-89440 V#48
 1015: finish collecting Section data - see sec
 behind clearly
 1030: picking up section gear



1045: start GRZ, start discontinuity
 measurements on SE side of bench
 DZ N $44^{\circ} 15.146'$ DZ N $44^{\circ} 15.171'$
 W $68^{\circ} 20.961'$ W $68^{\circ} 20.920'$
 927'

17h. by: (2) DZ N $44^{\circ} 15.116'$
 W $68^{\circ} 20.911'$
 - medium to coarse grained beige to light tan
 (ventured), m. gray (fine), reworked,
 granitic gneiss; in the greenish gray (ventured);
 medium to fine grained basalt - 1/2 in. veins, 9/102'
 Gneiss R5
 basalt R4
 1145: H.R. finished, GRZ can see old work
 Flies off site

5/17/12 Thurs 749 123-87440 V#48
 BB-TMB-102
 Coords: $44^{\circ} 15.170'$
 $68^{\circ} 20.952'$

- loose, not the same
 BB-TMB-105
 Coords: $44^{\circ} 15.170'$
 $68^{\circ} 20.936'$

BB-TMB-106
 Coords: $44^{\circ} 16.167'$
 $68^{\circ} 20.933'$

5/17/12 Thruses Jns 123-8746 v H8 Drmment

12	J	83	191	1'	<0.1"	none	mod	dry	0.5'	PL	SN
13	J	82	376	5'	<0.1"	none	mod	dry	1.0'	PL	SN
grows - in 654H											
14	J	80	191	4'	<0.1"	none	mod	dry	4'	PL	SN
15	J	66	152	3'	<0.1"	none	mod	dry	2"	PL	SN
16	J	86	198	5'	<0.1"	none	mod	dry	1'	PL	SN
17	J	73	332	10'	<0.1"	none	mod	dry	1'	PL	SN
18	J	70	231	1'	<0.1"	none	mod-low	cut	1.5'	PL	SN
19	J	85	168	10'	<0.1"	none	mod	dry	0.5'	PL	SN
20	J	73	179	5'	<0.1"	none	mod	cut	0.25'	PL	SN
grows											
21	J	71	275	25'	<0.1"	none	high	dry	0.5'	PL	R
22	J	83	033	15'	<0.1"	none	mod	dry	1'	PL	SN
23	J	78	292	3'	<0.1"	none	mod	dry	1'	PL	SN
24	J	81	320	15'	0.5"	new	low	cut	2'	PL	SN

5/17/12 Thruses Jns 123-8746 v H8 Drmment

1	J	85	161	6'	0.1	none	mod	dry	2"	PL	SN
2	J	73	245	10'	0.1	none	mod	dry	9"	PL	SN
3	J	90	085	50'	0.1	CH12	mod	dry	5"	PL	SN
4	J	87	1227	10'	0.2	none	mod	dry	1'	PL	SN
5	J	47	210	7'	-	none	mod	dry	5'	PL	SN
6	J	81	281	80'	0.25"	none	mod	cut	1'	PL	SN
7	J	90	077	5'	<0.1"	none	mod	dry	0.8'	PL	SN
8	J	82	322	15'	0.1"	none	mod	dry	0.2'	PL	SN
9	J	70	215	5'	<0.1"	none	mod	dry	0.1'	PL	SN
10	J	07	359	20'	<0.1"	none	high	damp	2-3'	SR112	VR
11	J	71	201	5'	<0.1"	none	mod	damp	2'	PL	SN

stress relief
soil

photo taken at time of joint sets

8/10/18

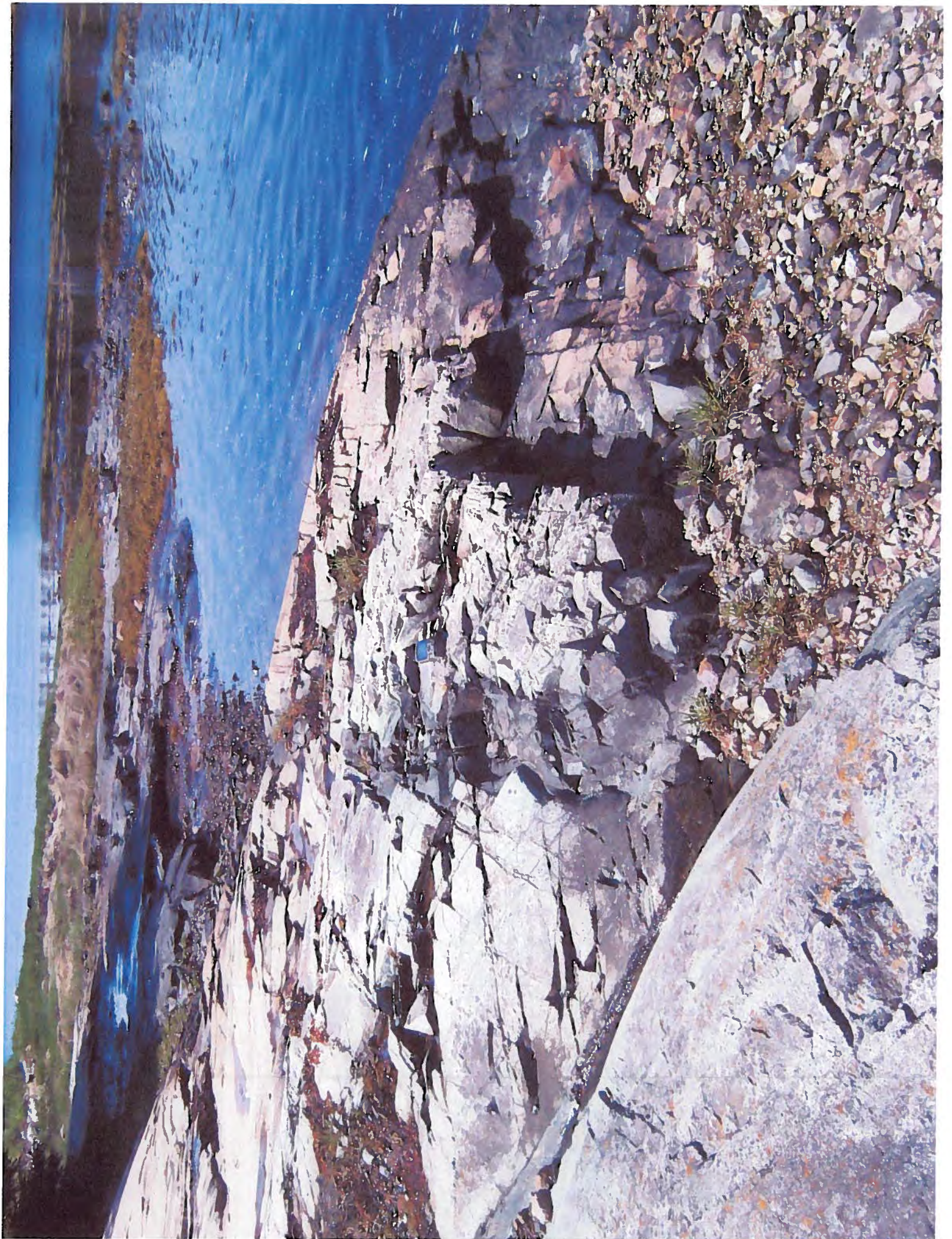




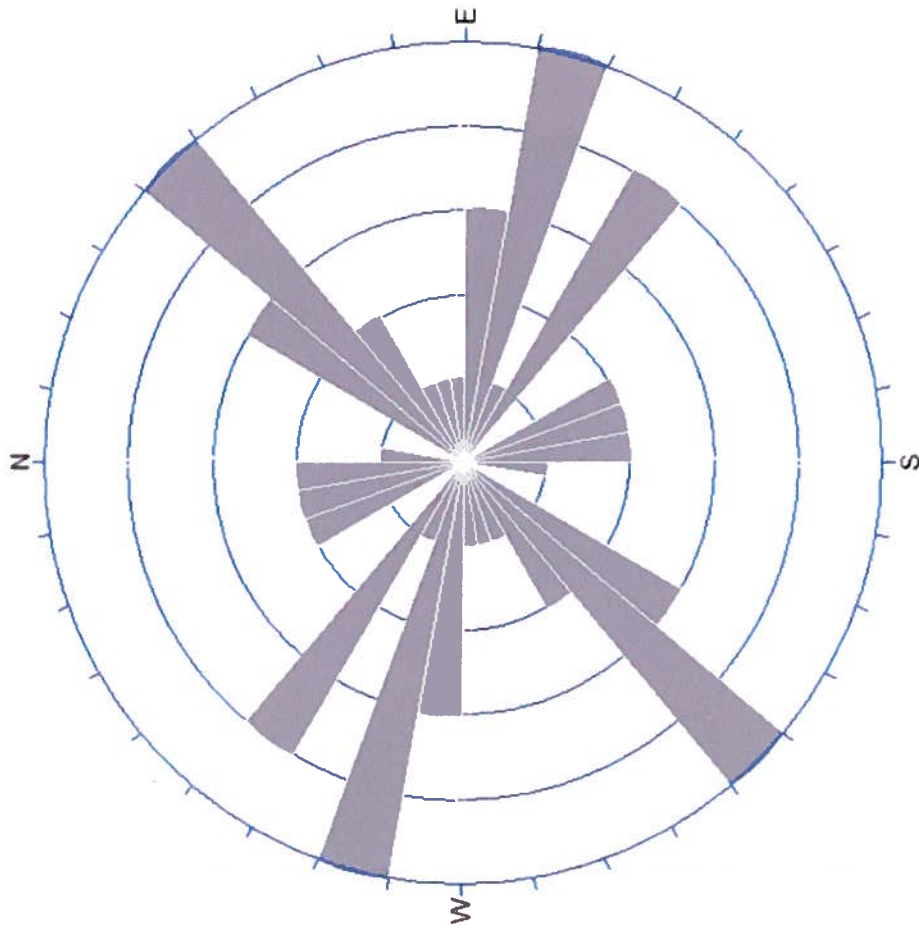
TABLE 1
DISCONTINUITY SUMMARY
CLARK BRIDGE REPLACEMENT PROJECT
TREMONT, MAINE

Discontinuity ID	Type	Dip [degrees]	Dip Direction uncorrected [degrees]	Dip Direction State Plane Grid North ⁽¹⁾ [degrees]	Persistence [ft]	Aperture [in]	Infilling	Strength ⁽²⁾	Surface Roughness	Fracture Shape	Water	Fracture Spacing [ft]
1	joint	85	161	144	6+	0.1	none	medium	smooth	planar	dry	0.2
2	joint	73	205	188	10+	0.1	none	medium	smooth	planar	dry	0.8
3	joint	89	85	68	50+	0.1	chlorite(?)	medium	smooth	planar	dry	0.4
4	joint	87	227	210	10+	0.2	none	medium	smooth	planar	dry	1
5	joint	47	210	193	<1	--	none	medium	smooth	planar	dry	5
6	joint	84	284	267	50+	0.25	none	medium	smooth	planar	wet	1
7	joint	89	47	30	5	<0.1	none	medium	smooth	planar	dry	0.5
8	joint	82	322	305	15+	0.1	none	medium	smooth	planar	dry	0.2
9	joint	89	215	198	5	<0.1	none	medium	smooth	planar	dry	0.1
10	joint	7	359	342	20	<0.1	none	high	very rough	irregular-planar	damp	2
11	joint	71	201	184	5	<0.1	none	medium	smooth	planar	damp	2
12	joint	83	151	134	4	<0.1	none	medium	smooth	planar	dry	0.5
13	joint	82	335	318	5	<0.1	none	medium	smooth	planar	dry	1
14	joint	80	194	177	4	<0.1	none	medium	smooth	planar	dry	4
15	joint	66	152	135	3	<0.1	none	medium	smooth	planar	dry	0.2
16	joint	86	198	181	5	<0.1	none	medium	smooth	planar	dry	1
17	joint	73	332	315	10	<0.1	none	medium	smooth	planar	dry	1
18	joint	89	234	217	1	<0.1	none	low-medium	smooth	planar	wet	1.5
19	joint	85	168	151	10	<0.1	none	medium	smooth	planar	dry	0.5
20	joint	73	179	162	5	<0.1	none	medium	smooth	planar	wet	0.25
21	joint	71	275	258	25	<0.1	none	high	rough	planar	dry	0.5
22	joint	83	33	16	15	<0.1	none	medium	smooth	planar	dry	1
23	joint	78	292	275	3	<0.1	none	medium	smooth	planar	dry	1
24	joint	81	320	303	15	0.5	sand/fine gravel	low	smooth	planar	wet	2
25	joint	74	265	248	3	<0.1	none	medium	smooth	planar	wet	1
26	joint	74	275	258	6	<0.1	none	medium	smooth	planar	wet	1
27	joint	85	232	215	20	0.1 - 1	quartz	medium	smooth	planar	wet	1
28	joint	8	142	125	3	<0.1	none	medium	rough	irregular-planar	dry	0.1
29	joint	81	320	303	10	<0.1	none	medium	rough	irregular-planar	dry	2
30	joint	85	331	314	15	<0.1	none	medium	rough	planar	damp	0.75
31	joint	55	285	268	3	0.1	none	medium	smooth	planar	dry	3
32	joint	88	215	198	20	<0.1	none	medium	smooth	curved-planar	dry	1.5
33	joint	80	214	197	20	<0.1	quartz	medium	smooth	planar	dry	0.3
34	joint	89	338	321	2	0.1	none	medium	rough	irregular-planar	dry	0.4
35	joint	87	45	28	5	0.2	quartz	medium	smooth	planar	dry	1.5
												0.4

Prepared by _____ $\bar{X} = 1.131$ FT
Checked by _____

$\bar{X} = 0.154$

- Notes:
 (1) Magnetic declination for site is 17 degrees west of true north.
 (2) Shear strength of infilling materials as follows:
 Low = friction angle < 20 degrees
 Medium = 20 degrees < friction angle < 30 degrees
 High = 30 degrees < friction angle
 ft = feet; in= inches

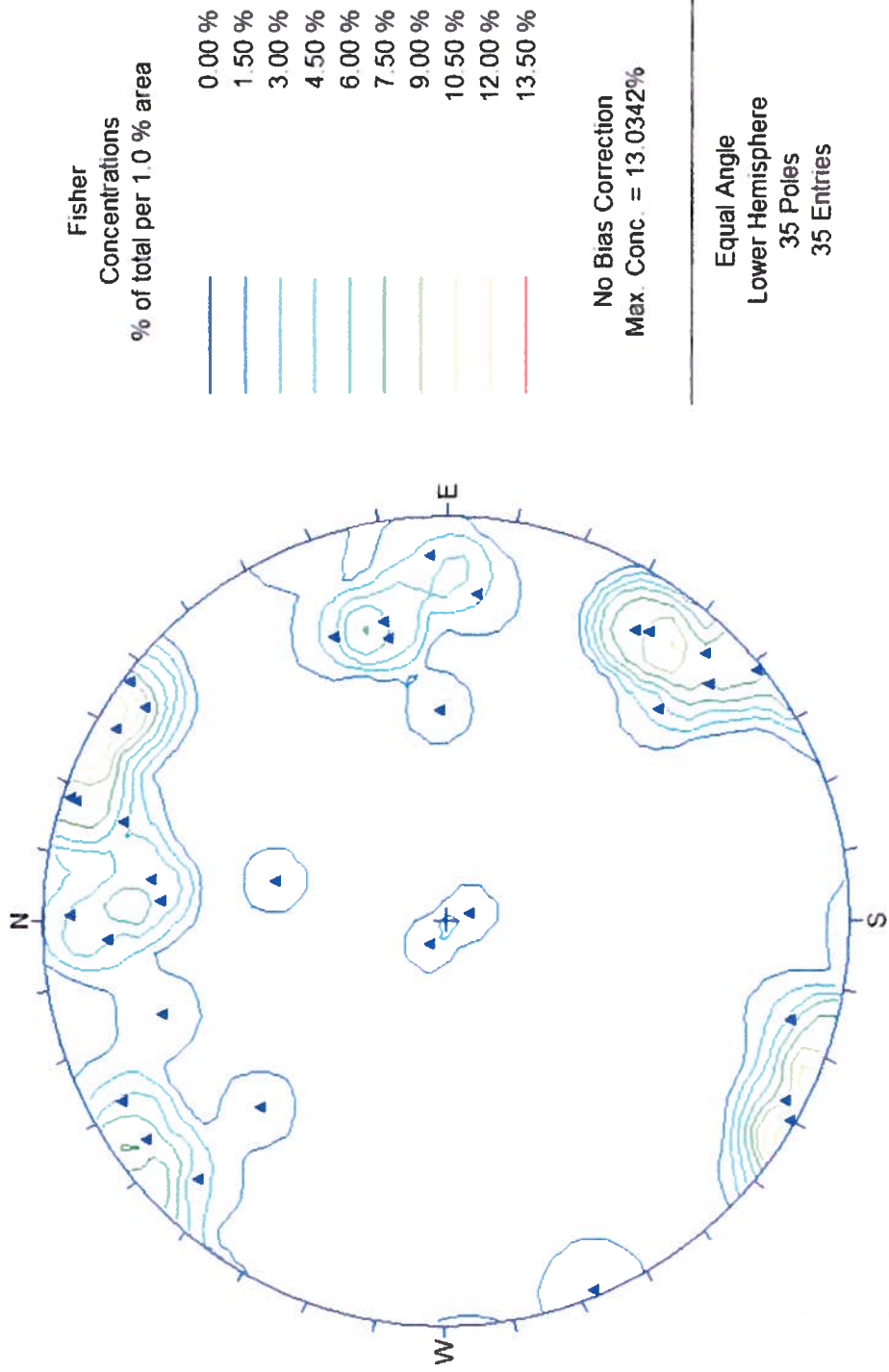


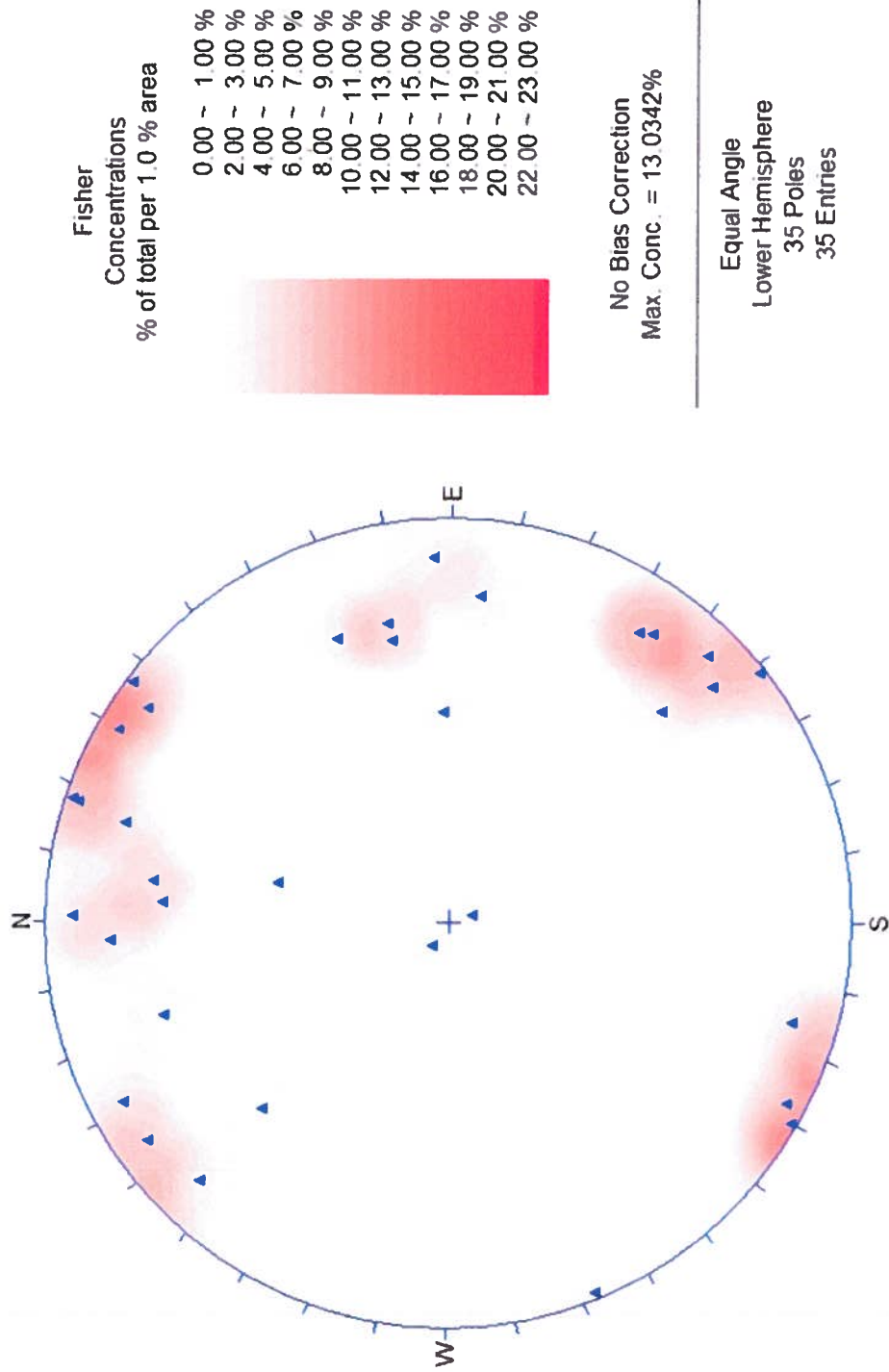
Apparent Strike
5 max planes / arc
at outer circle

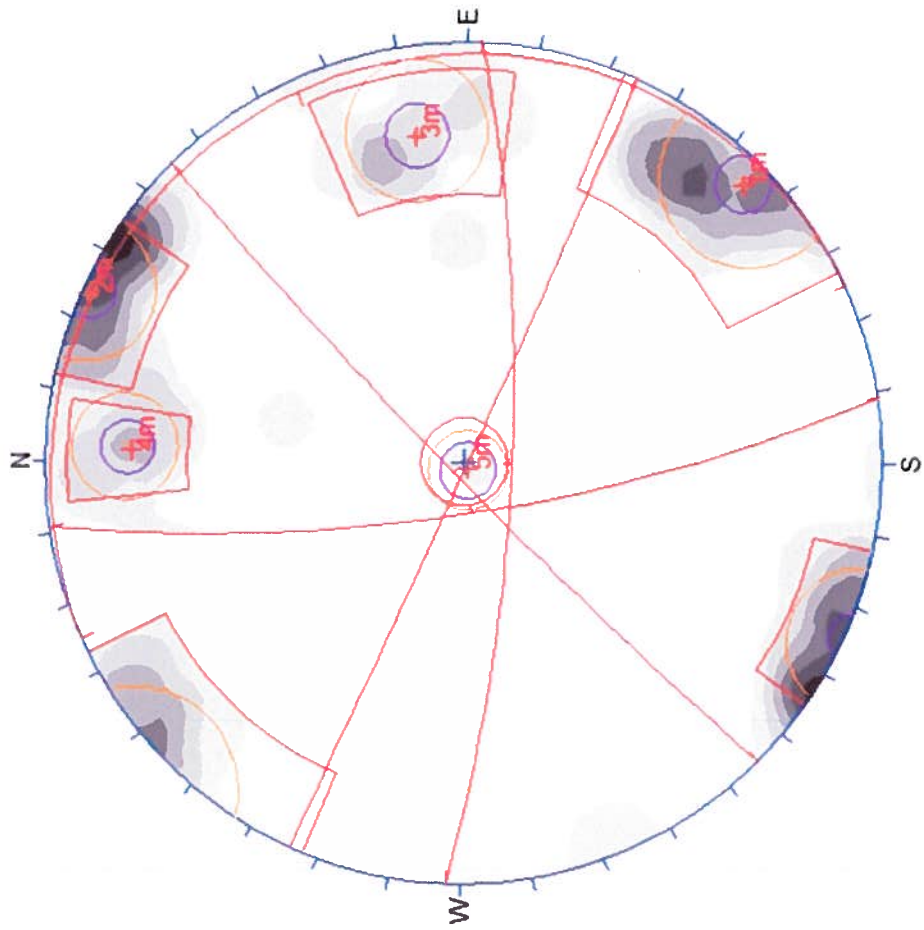
Trend / Plunge of
Face Normal = 0, 90
(directed away from viewer)

No Bias Correction

33 Planes Plotted
Within 45 and 90
Degrees of Viewing
Face







Orientations	
ID	Dip / Direction
1	m 86 / 315
2	m 89 / 204
3	m 76 / 261
4	m 77 / 182
5	m 02 / 065

Equal Angle
 Lower Hemisphere
 35 Poles
 35 Entries

TABLE 4.1 The Rock Mass Rating System (Geomechanics Classification of Rock Masses)^a

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter		Range of Values							
1	Strength of intact rock material	Point load strength index (MPa)	>10	4-10	2-4	1-2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100-250	50-100	25-50	5-25	1-5	<1
Rating		15	12	7	4	2	1	0	
2	Drill core quality RQD (%)	90-100	75-90	50-75	25-50	<25			
		Rating	20	17	13	8	3		
3	Spacing of discontinuities	>2 m	0.6-2 m	200-600 mm	60-200 mm	<60 mm			
		Rating	20	15	10	8	5		
4	Condition of discontinuities	Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous			
		Rating	30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10-25	25-125	>125		
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	<0.1	0.1-0.2	0.2-0.5	>0.5		
	General conditions	Completely dry	Damp	Wet	Dripping	Flowing			
	Rating	15	10	7	4	0			

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15-m span	1 yr for 10-m span	1 wk for 5-m span	10 h for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass (kPa)	>400	300-400	200-300	100-200	<100
Friction angle of the rock mass (deg)	>45	35-45	25-35	15-25	<15

^a After Bieniawski (1979).

B

CHART A Ratings for Strength of Intact Rock

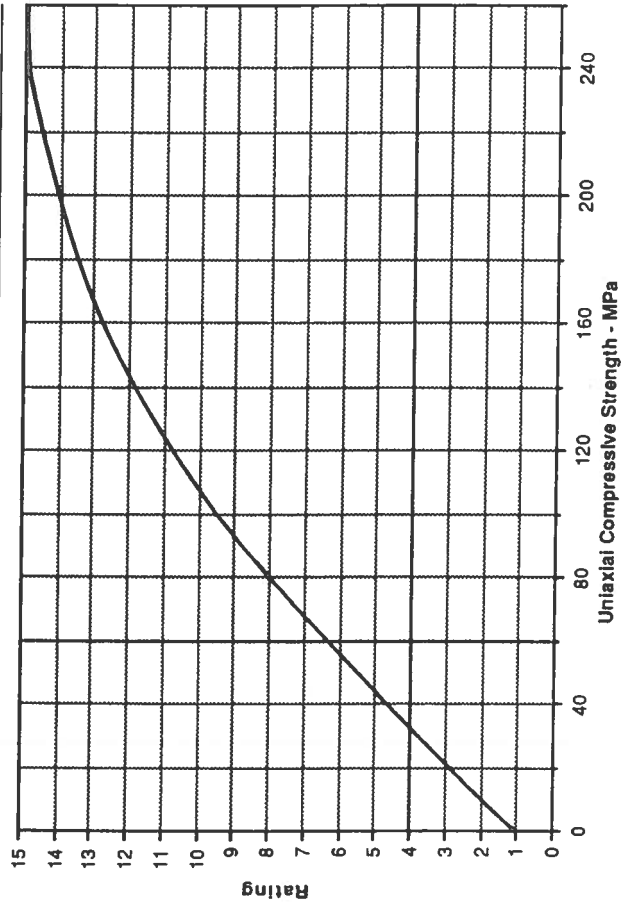


CHART B Ratings for RQD

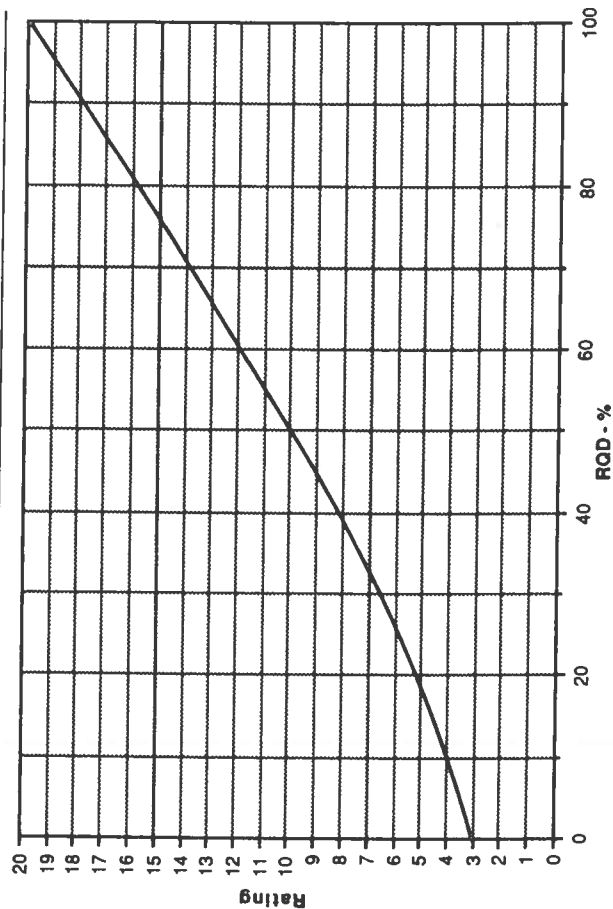


CHART C Ratings for Discontinuity Spacing

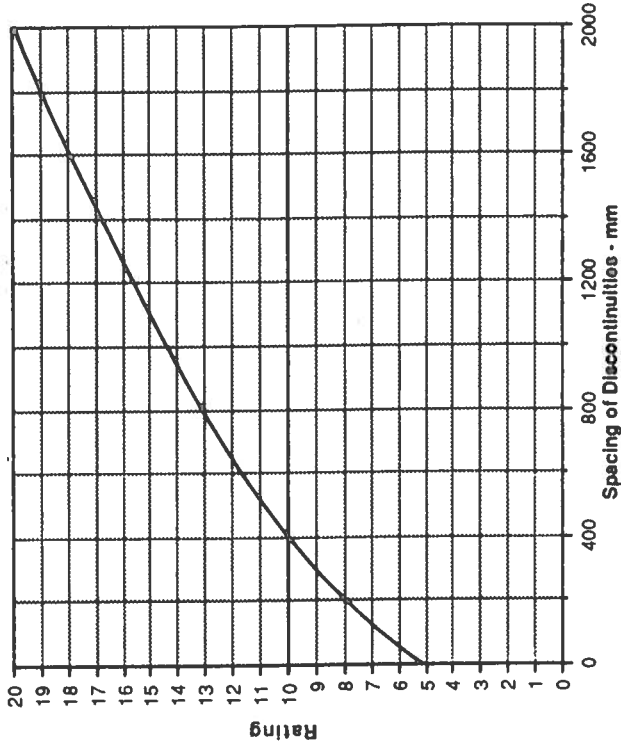


CHART D Chart for Correlation between RQD and Discontinuity Spacing

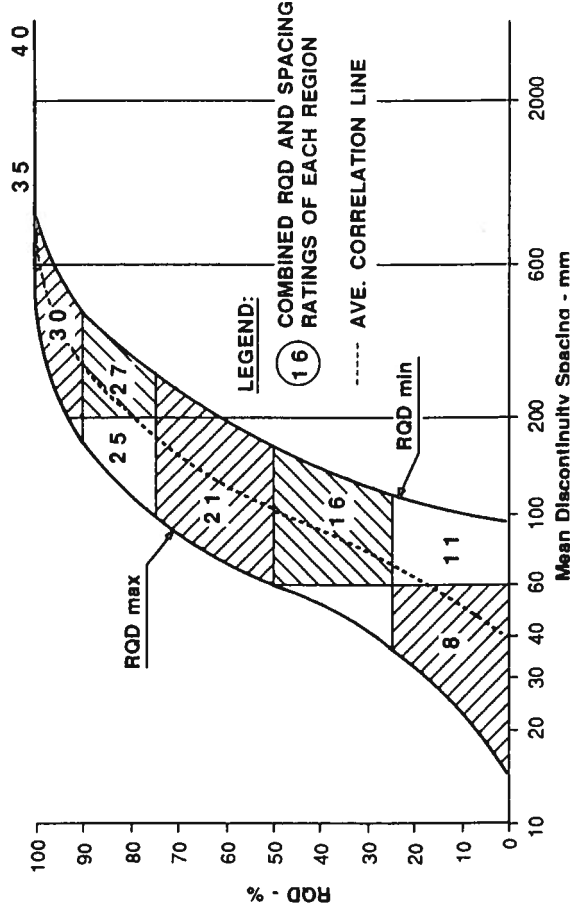


Table 3.5 RMR classification of jointed rock masses (extract from Bieniawski, 1974)

Parameter		Ranges of values					
A. Classification parameters and their ratings							
1	Strength of intact rock material Rating	Point load strength index Uniaxial compressive strength	>8 MPa >1.2 ksi 200 MPa 15	4-8 MPa 0.6-1.2 ksi 100-200 MPa 12	2-4 MPa 0.3-0.6 ksi 50-100 MPa 7	1-2 MPa 0.8-0.3 ksi 25-50 MPa 4	For this low range uniaxial compressive test is preferred 10-25 MPa 3-10 MPa 1-3 MPa 2 1 0
2	Drill core quality RQD Rating		90%-100% 20	75%-90% 17	50%-75% 13	25%-50% 8	<25% 3
3	Spacing of joints Rating		>3 m (>10 ft) 30	1-3 m (3-10 ft) 25	0.3-1 m (1-3 ft) 20	50-300 mm (2-12 in) 10	<50 mm (<2 in) 5
4	Condition of joints Rating		Very rough surfaces Not continuous No separation Hard joint wall rock 25	Slightly rough surfaces Separation <1 mm Hard joint wall rock 20	Slightly rough surfaces Separation <1 mm Soft joint wall rock 12	Slippen sided surfaces or Gouge <5 mm thick or Joints open 1-5 mm Continuous joints 6	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints 0
5	Ground water Rating		Completely dry 10	Moist only (interstitial water) 7	Water under moderate pressure 4	Severe water problem 0	
When calculating rock strength using Table 3.7, rating = 10; ground water pressures accounted for in stability analysis.							
orientations							
			Very favorable 0	Favorable -2	Fair -7	Unfavorable -15	Very unfavorable -25
When calculating rock strength using Table 3.7, adjustment = 0; joint orientation accounted for in stability analysis.							

ATTACHMENT C
REF. 5
C-1/6

Table 3.6 Classification of rock material strengths (ISRM, 1981)

Grade	Description	Field identification	Approximate range of compressive strength	
			MPa	(p.s.i)
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250	(>36 000)
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250	(15 000-36 000)
R4	Strong rock	Specimen requires more than one blow with a geological hammer to fracture it.	50-100	(7 000-15 000)
R3	Medium weak rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer	25-50	(3 500-7 000)
R2	Weak rock	Can be peeled with a pocket knife; shallow indentations made by firm blow with point of geological hammer	5-25	(725-3 500)
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife	1-5	(150-725)
R0	Extremely weak rock	Indented by thumbnail	0.25-1	(35-150)
S6	Hard clay	Indented with difficulty by thumbnail	>0.5	(>70)
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.5	(35-70)
S4	Stiff clay	Readily indented by thumb but penetrated only with great difficulty	0.1-0.25	(15-35)
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.1	(7-15)
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05	(4-7)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025	(<4)

shown in Table 3.6. The letter designations (R0 etc.) can be used on drill logs and field mapping sheets to record the rock strength values (Appendix II).



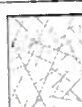



3.3.1 Compressive strength of intact rock

The compressive strength of intact rock can readily be measured using either a compression machine or

a point load tester (Fig. 3.11(a)). The compression machine gives the more precise results but it is necessary to prepare the samples in the manner described in Section 3.2.1 on modulus testing.

Estimation of compressive strength with the point load testing equipment has the advantage that tests can be conducted on lengths of unprepared core in axial and diametral directions, as well as on irregular lumps of rock (ISRM, 1985).

Table 4.3 GSI values characterizing blocky rock masses on the basis of particle interlocking and discontinuity condition

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)		SURFACE CONDITIONS				
<p>From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI=35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavorable orientation with respect to the excavation face, these will dominate the rock mass behavior. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.</p>		DECREASING SURFACE QUALITY →				
		VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
STRUCTURE	DECREASING INTERLOCKING OF ROCK PIECES ↓					
 INTACT OR MASSIVE—intact rock specimens or massive <i>in situ</i> rock with few widely spaced discontinuities	90				N/A	N/A
 BLOCKY—well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets	80					
 VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets						
 BLOCKY/DISTURBED/SEAMY—folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity						
 DISINTEGRATED—poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces						
 LAMINATED/SHEARED—lack of blockiness due to close spacing of weak schistosity or shear planes						
		N/A	N/A			10



ATTACHMENT D
REF. 7
D-1/1

Table 4.4 GSI values characterizing schistose metamorphic rock masses on the basis of foliation and discontinuity condition

VEF side with
POC slick coat fragr
FAIF wear
GOC wear
VER/ fresh
SURI DISC (Prec

GSI FOR HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH (Marinos P. and Hoek E., 2000)
From a description of the lithology, structure and surface conditions of the rock mass

Golder Associates

RMR CALCULATIONS FOR ROCK CORE RUNS, CLARK BRIDGE, SUBJECT TREMONT, ME		
Job No. 123-87440	Made by JRS	Date 7/12/12
Ref. 2	Checked JOL	Sheet 1 of 7
	Reviewed MSP	

PROBLEM: ESTIMATE ROCK MASS RATING (RMR) CLASSIFICATION FOR EACH CORE RUN (6) FOR THREE (3) BORINGS CONDUCTED FOR REPLACEMENT OF THE CLARK BRIDGE ON ROUTE 103 IN TREMONT, ME. USE ROCK CORE LOGS, LABORATORY TEST RESULTS AND FIELD MAPPING DATA.

- REFERENCES:
- 1) BIENIAWSKI, Z.T., 1989. ENGINEERING ROCK MASS CALCULATIONS, JOHN WILEY + SONS, NY (ATTACHMENT A)
 - 2) WYLLIE, D.C., 1999. FOUNDATIONS ON ROCK, 2nd ED., E + FN SPON PRESS, LONDON, 402 P. (ATTACHMENT B)
 - 3) WYLLIE, D.C. AND MAH, C., 2004. ROCK SLOPE ENGINEERING, 4th ED., SPON PRESS, NY, 431 P. (ATTACHMENT C)
 - 4) GOLDER ASSOCIATES INC., JULY 2012. FIELD NOTES, BORING LOGS, DATA TABLES AND LABORATORY TESTING RESULTS, CLARK BRIDGE REPLACEMENT PROJECT FILES, No. 123-87440. (ATTACHMENT D)
 - 5) MAINE DOT, 2008. KEY TO SOIL AND ROCK DESCRIPTION AND TERMS. (ATTACHMENT E)

PROCEDURE: FOLLOW METHODS PRESENTED IN REFS. 1+2 TO SELECT ROCK CHARACTERIZATION DATA FROM EACH CORE RUN TO CALCULATE RMR. USE TABLES AND CHARTS IN THESE REFERENCES IN CONJUNCTION WITH SITE DATA.

BB-TMB-101

Run 1: A-1: STRENGTH OF INTACT ROCK: R4 50-100 MPa (7,000-15,000 psi)
 PLT 8.1 MPa (1,173 psi, uncor.)
 184.8 MPa (26,800 psi, corr.)
 SAY 120 MPa (avg.)
 RATING = (11)

A-2: RQD = 42% RATING = (8)

A-3: JOINT SPACING CLOSE TO VERY CLOSE (<5cm + 30cm)
 SAY AVG = 20 cm
 RATING = (8)

A-4: JOINT CONDITIONS SMOOTH TO ROUGH TO VERY ROUGH, SL. TO MOD. WEATHERED
 SAY AVG = ROUGH
 RATING = (22)

PROCEDURE:
 (CONTINUED)

A-5: GROUNDWATER: FOUNDATIONS WILL BE BENEATH GROUNDWATER/RIVER LEVEL, ASSUME MODERATE WATER PRESSURE
 RATING = (4)

B: JOINT ORIENTATIONS: BASED ON OUTCROP DATA, THREE PRINCIPAL JOINT SETS (1M, 2M AND 5M) ARE ORTHOGONAL (2 VERTICAL, 1 HORIZONTAL). TWO OTHER SECONDARY SETS (3M, 4M) ARE STEEPLY INCLINED (76-77°); ASSUME FAIR ORIENTATION FOR FOUNDATIONS; LOG INDICATES HORIZONTAL + VERTICAL
 RATING = (-7)

$$RMR = \sum_{03-TMB-201 R1} 11 + 8 + 8 + 22 + 4 - 7 = \boxed{46}$$

RUN 2: A-1: STRENGTH OF INTACT ROCK:

R4	50 - 100 MPa	(7,000 - 15,000 psi)
VCS	217.7 MPa	(31,567 psi)
PLT	11.2 MPa	(1,617 psi, uncor.)
	255.2 MPa	(37,000 psi, com.)

SAY 150 MPa
 RATING = (12)
 RATING = (10)

A-2: RQD = 50%

A-3: JOINT SPACING

CLOSE TO VERY CLOSE (<5cm to 30cm)
 SAY AVG = 20 cm
 RATING = (8)

A-4: JOINT CONDITIONS

VERY ROUGH TO SMOOTH, SLIGHTLY TO MOD. WEATHERED
 SAY AVG. = ROUGH
 RATING = (22)

A-5: GROUNDWATER

FOUNDATIONS WILL BE BENEATH GROUNDWATER/RIVER LEVEL, ASSUME MODERATE WATER PRESSURE
 RATING = (4)

B: JOINT ORIENTATIONS

HORIZONTAL TO VERTICAL JOINTS; BASED ON MAPPING DATA DESCRIBED ABOVE, ASSUME FAIR CONDITIONS FOR FOUNDATIONS.
 RATING = (-7)

$$RMR = \sum_{03-TMB-202 R2} = 12 + 10 + 8 + 22 + 4 - 7 = \boxed{49}$$

PROCEDURE: BID-TMB-202
 (CONTINUED)

RUN 1

- A-1: STRENGTH OF IMPACT ROCK R4 50 - 100 MPa (7,000 - 15,000 PSI)
 NO LAB DATA
 SAY AV = 75 MPa
 RATING = (8)
- A-2: RQD = 45% RATING = (9)
- A-3: JOINT SPACING CLOSE TO VERY CLOSE (<5cm → 30cm)
 SAY AV = 20 cm
 RATING = (8)
- A-4: JOINT CONDITIONS SMOOTH TO VERY ROUGH, SLIGHTLY TO MODERATELY WEATHERED, TALC NOTED (POOR CONDITION)
 RATING = (10)
- A-5: GROUNDWATER MODERATE WATER PRESSURE
 RATING = (4)
- B: JOINT ORIENTATION SUBHORIZONTAL TO STEEP JOINTS NOTED IN LOG; ASSUME UNFAVORABLE CONDITIONS FOR FOUNDATIONS
 RATING = (-15)

$$RMR = \sum_{TMB-202 R2} = 8 + 9 + 8 + 10 + 4 - 15 = \boxed{24}$$

RUN 2:

- A-1: STRENGTH OF IMPACT ROCK R4 50 - 100 MPa (7,000 - 15,000 PSI)
 UCS 91 MPa (13,189 PSI)
 SAY 75 MPa
 RATING = (8)
- A-2: RQD = 33% RATING = (7)
- A-3: JOINT SPACING CLOSE TO VERY CLOSE (<5cm → 30cm)
 SAY AV = 20 cm
 RATING = (8)
- A-4: JOINT CONDITIONS SMOOTH TO VERY ROUGH, TALC + CHLORITE NOTED (POOR CONDITIONS)
 RATING = (10)

Golder Associates

RMR CALCULATIONS FOR ROCK CORE RUNS, CLARK BRIDGE,		
SUBJECT TREMONT, AK		
Job No. 123-87440	Made by JRS	Date 7/12/12
Ref.	Checked JOL	Sheet 4 of 7
	Reviewed WSP	

PROCEDURES:
(CONTINUED)

A-5: GROUNDWATER

MODERATE WATER PRESSURE

RATING = (4)

B: JOINT ORIENTATION

SUBHORIZONTAL TO STEEP, ASSUME UNFAVORABLE CONDITIONS FOR FOUNDATIONS

RATING = (-15)

$$RMR = \sum_{\text{BB TM0-202 R2}} = 8 + 7 + 8 + 10 + 4 - 15 = \boxed{22}$$

RUN 3:

A-1: STRENGTH OF INTACT ROCK

R4 50 - 100 MPa (7,000 - 15,000 PSI)

NO LAB DATA

ASSUME 75 MPa

RATING = (8)

A-2: RQD = 65%

RATING = (13)

A-3: JOINT SPACING

CLOSE TO VERY CLOSE (45 → 30 cm)

SAY AVG = 20 cm

RATING = (8)

A-4: JOINT CONDITIONS

SMOOTH TO VERY ROUGH, TALL + CHALKY NOTED (POOR CONDITIONS)

RATING = (10)

A-5: GROUNDWATER

MODERATE WATER PRESSURE

RATING = (4)

B: JOINT ORIENTATION

SUBHORIZONTAL TO STEEP, ASSUME

UNFAVORABLE CONDITIONS FOR FOUNDATIONS

RATING = (-15)

$$RMR = \sum_{\text{BB TM0-202 R3}} = 8 + 13 + 8 + 10 + 4 - 15 = \boxed{28}$$

Golder Associates

SUBJECT RMR CALCULATIONS FOR ROCK CORE RUNS, CLACK BRIDGE, TREMONT, ME		
Job No. 123-8740	Made by JRS	Date 7/12/12
Ref. 2	Checked JOL	Sheet 5 of 7
	Reviewed MJP	

PROCEDURE BB-TMB-103
(CONTINUED)

- RUN 1
- A-1: STRENGTH OF IMPACT ROCK

R3	25-50 MPa	(3,500-7,000 PSI)
PLT	3.6 MPa	(521 PSI, uncorr.)
	82 MPa	(11,900 PSI, corr.)
SAY	50 MPa	
RATING	= (5)	
 - A-2: RQD = 32%

RATING	= (7)
--------	-------
 - A-3: JOINT SPACING

CLOSE TO VERY CLOSE (<5cm → 30cm)	
SAY 20cm	
RATING	= (8)
 - A-4: JOINT CONDITION

ROUGH TO VERY ROUGH, SLIGHTLY TO MODERATELY WEATHERED	
RATING	= (22)
 - A-5: GROUNDWATER

MODERATE WATER PRESSURE	
RATING	= (4)
 - B: JOINT ORIENTATION

HORIZONTAL TO STEEP, ASSUME UNFAVORABLE CONDITIONS FOR FOUNDATIONS	
RATING	= (-15)

$$RMR \sum_{BB-TMB-103 R1} = 5 + 7 + 8 + 22 + 4 - 15 = \boxed{31}$$

- RUN 2
- A-1: STRENGTH OF IMPACT ROCK

R3	25-50 MPa	(3,500-7,000 PSI)
PLT	5.7 MPa	(821 PSI, uncorr.)
	129.7 MPa	(18,800 PSI, corr.)
SAY	75 MPa	
RATING	= (8)	
 - A-2: RQD = 7%

RATING	= (3)
--------	-------
 - A-3: JOINT SPACING

CLOSE TO VERY CLOSE (<5cm → 30cm)	
SAY 20cm	
RATING	= (8)

Golder Associates

SUBJECT RMR CALCULATIONS FOR ROCK CORE RUNS, CLARK BRIDGE, PREMONT, ME		
Job No. 123-8744	Made by JCS	Date 7/12/12
Ref. 2	Checked JOL	Sheet 6 of 7
	Reviewed MGP	

PROCEDURES:
(CONTINUED)

A-4: JOINT CONDITIONS

ROUGH TO VERY ROUGH, SLIGHTLY TO HIGHLY WEATHERED
RATING = (6)

A-5: GROUND WATER

MODERATE PRESSURE
RATING = (4)

B: JOINT ORIENTATION

HORIZONTAL TO STEEP, ASSUME UNFAVORABLE CONDITIONS FOR FOUNDATIONS
RATING = (-15)

$$RMR \sum_{BB-TMB-23R2} = 8 + 3 + 8 + 5 + 4 - 15 = \boxed{13}$$

SUMMARY

SEE PAGE 7 THESE CALCULATIONS FOR SUMMARY TABLE OF RMR'S.

JRS 123-874/0
7 of 7

RMR Calculation:

Boring:	Run:	Classification Parameters					Rating Adjustment for joint orientation	RMR:
		1	2	3	4	5		
		Strength of rock	RQD	Spacing of joints	Condition of Joints	Ground water		
BB-TMB-101	1	11	8	8	22	4	-7	846
	2	12	10	8	22	4	-7	849
BB-TMB-102	1	8	9	8	10	4	-15	24
	2	8	7	8	10	4	-15	822
	3	8	13	8	10	4	-15	828
BB-TMB-103	1	5	7	8	22	4	-15	831
	2	8	3	8	5	4	-15	813

RMR evaluated by JRS, 07/12/12

TABLE 4.1 The Rock Mass Rating System (Geomechanics Classification of Rock Masses)^a

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4-10		
		Uniaxial compressive strength (MPa)	>250	100-250		
Rating			15	12		
2	Drill core quality RQD (%)		90-100	75-90		
	Rating		20	17		
3	Spacing of discontinuities		>2 m	0.6-2 m	200-600 mm	60-200 mm
	Rating		20	15	10	8
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous
	Rating		30	25	20	10
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10-25	25-125
		Joint water pressure Ratio Major principal stress	0	<0.1	0.1-0.2	0.2-0.5
	General conditions	Completely dry	Damp	Wet	Dripping	Flowing
Rating			15	10	7	4

For this low range, uniaxial compressive test is preferred

5-25	1-5	<1
2	1	0
<25		
3		
<60 mm		
5		
0		

REF 1
ATT A

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15-m span	1 yr for 10-m span	1 wk for 5-m span	10 h for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass (kPa)	>400	300-400	200-300	100-200	<100
Friction angle of the rock mass (deg)	>45	35-45	25-35	15-25	<15

^a After Bieniawski (1979).

ATTACHMENT A
REF. 1

A-1/2

CHART A Ratings for Strength of Intact Rock

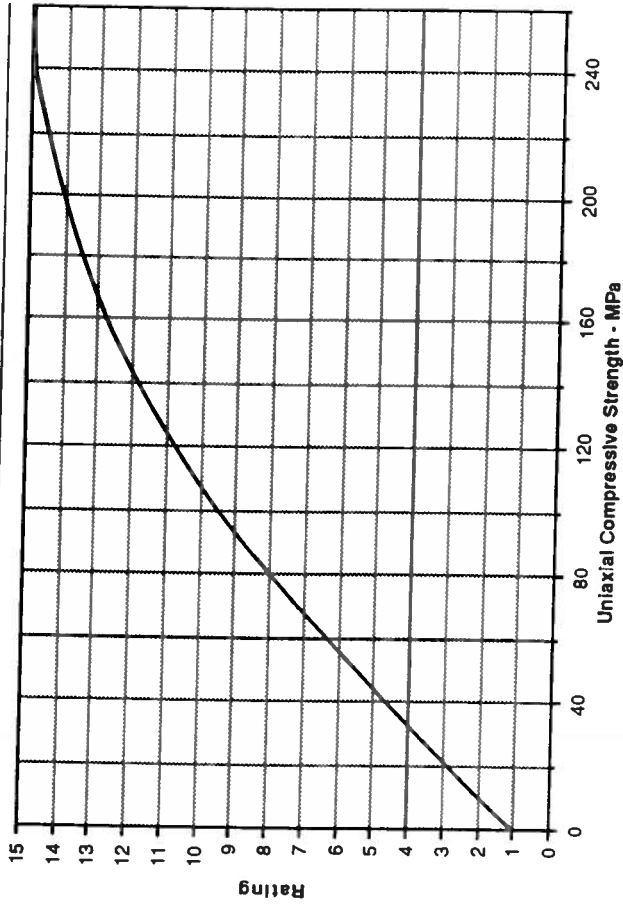


CHART C Ratings for Discontinuity Spacing

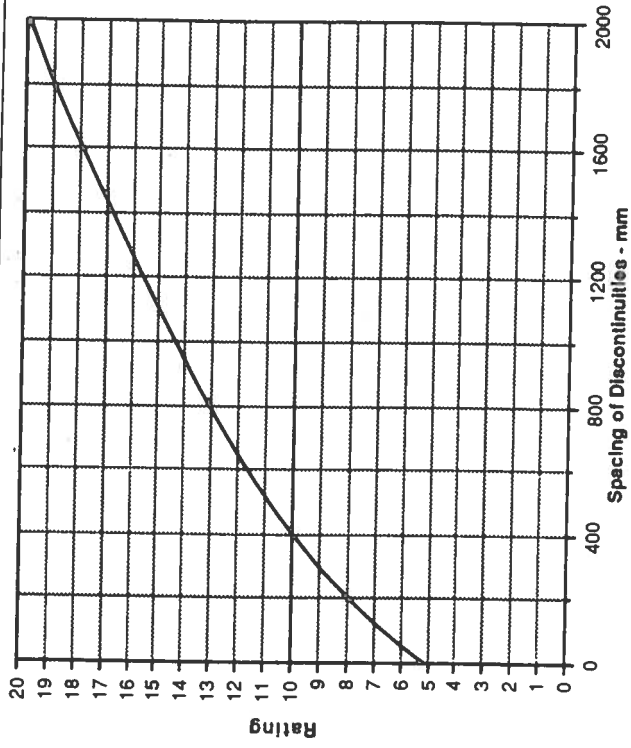


CHART B Ratings for RQD

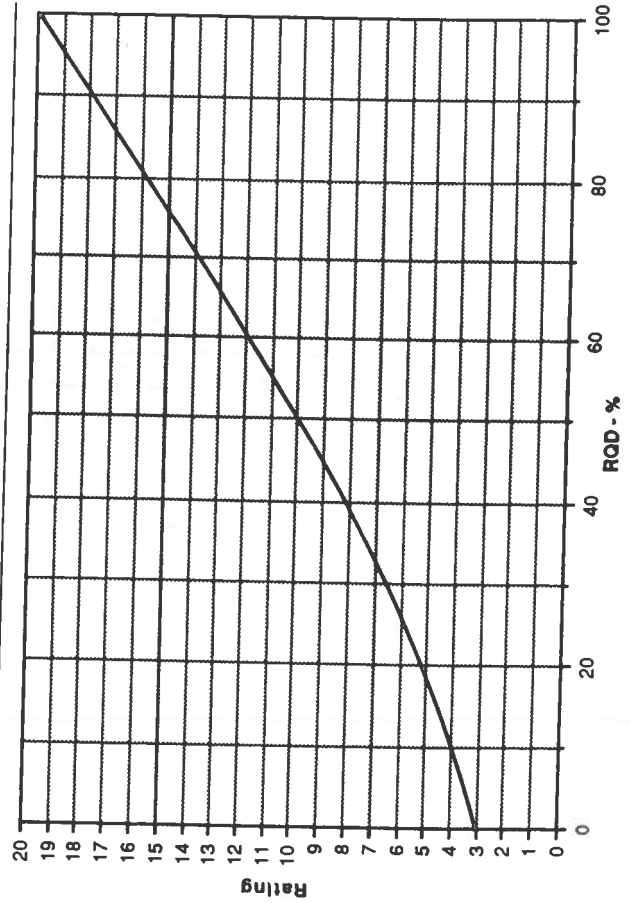
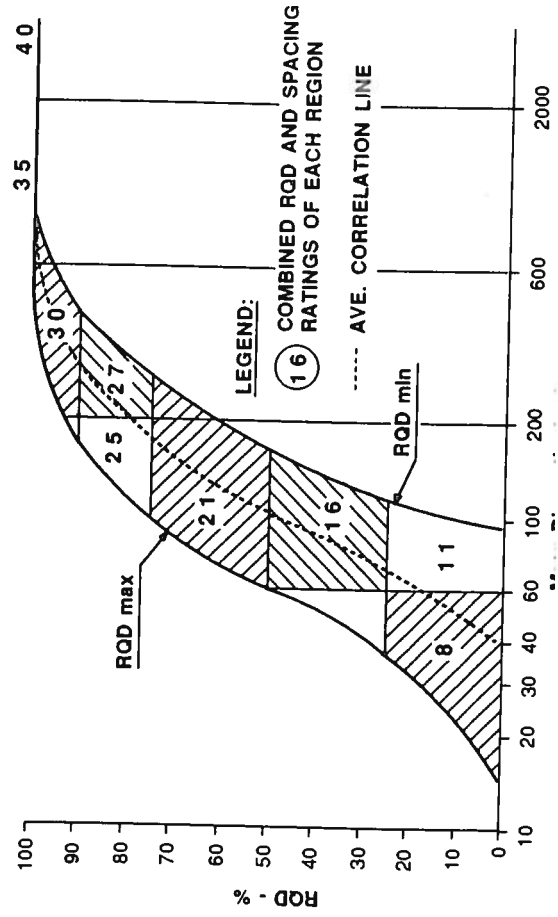


CHART D Chart for Correlation between RQD and Discontinuity Spacing



REF 2

ATT B

Table II.2 Grain size scale -

Description	Grain size
Boulders	200-600 mm (7.9-23.6 in)
Cobbles	60-200 mm (2.4-7.9 in)
Coarse gravel	20-60 mm (0.8-0.24 in)
Medium gravel	6-20 mm (0.2-0.8 in)
Fine gravel	2-6 mm (0.1-0.2 in)
Coarse sand	0.6-2 mm (0.02-0.1 in)
Medium sand	0.2-0.6 mm (0.008-0.02 in)
Fine sand	0.06-0.2 mm (0.002-0.008 in)
Silt, clay	<0.06 mm (<0.002 in)

Table II.3 Classification of rock material strengths

Grade	Description	Field identification	Approximate compressive (MPa)	Range of strength (psi)
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250	>36,000
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100-250	15,000-36,000
R4	Strong rock	Specimen requires more than one blow with a geological hammer to fracture it.	50-100	7000-15,000
R3	Medium weak rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer.	25-50	3500-7000
R2	Weak rock	Can be peeled with a pocket knife; shallow indentations made by firm blow with point of geological hammer.	5-25	725-3500
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife.	1-5	150-725
R0	Extremely weak rock	Indented by thumbnail.	0.25-1	35-150
S6	Hard clay	Indented with difficulty by thumbnail.	>0.5	>70
S5	Very stiff clay	Readily indented by thumbnail.	0.25-0.5	35-70
S4	Stiff clay	Readily indented by thumb but penetrated only with great difficulty.	0.1-0.25	15-35
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort.	0.05-0.1	7-15
S2	Soft clay	Easily penetrated several inches by thumb.	0.025-0.05	4-7
S1	Very soft clay	Easily penetrated several inches by fist.	<0.025	<4

ATTACHMENT 13

REF. 2

B-1/1

29

Table 3.6 Classification of rock material strengths (ISRM, 1981)

Grade	Description	Field identification	Approximate range of compressive strength	
			MPa	(p.s.i)
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250	(>36 000)
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250	(15 000-36 000)
R4	Strong rock	Specimen requires more than one blow with a geological hammer to fracture it.	50-100	(7 000-15 000)
R3	Medium weak rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer	25-50	(3 500-7 000)
R2	Weak rock	Can be peeled with a pocket knife; shallow indentations made by firm blow with point of geological hammer	5-25	(725-3 500)
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife	1-5	(150-725)
R0	Extremely weak rock	Indented by thumbnail	0.25-1	(35-150)
S6	Hard clay	Indented with difficulty by thumbnail	>0.5	(>70)
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.5	
S4	Stiff clay	Readily indented by thumb but penetrated only with great difficulty	0.1-0.25	
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.1	
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05	
S1	Very soft clay	Easily penetrated several inches by fist	<0.025	

REF 3
ATT C

shown in Table 3.6. The letter designations (R0 etc.) can be used on drill logs and field mapping sheets to record the rock strength values (Appendix II).

3.3.1 Compressive strength of intact rock

The compressive strength of intact rock can readily be measured using either a compression machine or

a point load tester (Fig. 3.11(a)). The compression machine gives the more precise results but it is necessary to prepare the samples in the manner described in Section 3.2.1 on modulus testing.

Estimation of compressive strength with the point load testing equipment has the advantage that tests can be conducted on lengths of unprepared core in axial and diametral directions, as well as on irregular lumps of rock (ISRM, 1985).

ATTACHMENT C
REF. 3
C-1/2

Table 3.5 RMR classification of jointed rock masses (extract from Bieniawski, 1974)

Parameter		Ranges of values					
A. Classification parameters and their ratings							
1	Strength of intact rock material Rating	Point load strength index Uniaxial compressive strength	>8 MPa >1.2 ksi	4-8 MPa 0.6-1.2 ksi	2-4 MPa 0.3-0.6 ksi	1-2 MPa 0.8-0.3 ksi	For this low range uniaxial compressive test is preferred 10-25 MPa 3-10 MPa 1-3 MPa
2	Drill core quality RQD Rating		>200 MPa 15	100-200 MPa 12	50-100 MPa 7	25-50 MPa 4	2 1 0
3	Spacing of joints Rating		90%-100% 20	75%-90% 17	50%-75% 13	25%-50% 8	<25% 3
4	Condition of joints Rating		>3 m (>10 ft) 30	1-3 m (3-10 ft) 25	0.3-1 m (1-3 ft) 20	50-300 mm (2-12 in) 10	<50 mm (<2 in) 5
5	Ground water Rating		Very rough surfaces Not continuous No separation Hard joint wall rock 25	Slightly rough surfaces Separation <1 mm Hard joint wall rock 20	Slightly rough surfaces Separation <1 mm Soft joint wall rock 12	Slicked sided surfaces or gouges <5 mm thick or Joints open 1-5 mm Continuous joints 6	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints 0
When calculating rock strength using Table 3.7, rating = 10; ground water pressures accounted for in stability analysis.							
B. Rating adjustment for joint orientations							
	Orientation of joints Adjustment for foundations		Very favorable 0	Favorable -2	Fair -7	Unfavorable -15	Very unfavorable -25
When calculating rock strength using Table 3.7, adjustment = 0; joint orientation accounted for in stability analysis.							

Table 2: Summary of Rock Core Quality
 Preliminary Geotechnical Data Report
 Clark Bridge
 Tremont, Maine
 MaineDOT WIN: 19305.00

DRAFT

Test Boring Designation	Existing Ground Surface Elevation ¹ (ft-msl)	Run			TCR ²		RQD ³		Intact Rock					
		No.	Depth Below Ground Surface (ft)		Length (ft)	Percent	Length (ft)	Percent	Weathering	Strength	Rock Mass Rating ⁴	Description		
			Start	End										
BB-TMB-101	11.2	R1	12.0	17.0	5.0	5.0	100%	2.1	42%	Poor	R4	Moderately Weathered to Slightly Weathered	12.0 to 2.9 ft-bgs: Dark gray to greenish-black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular. 12.9 to 17.0 ft-bgs: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft-bgs, approximately 1 inch thick and subhorizontal.	
			R2	17.0	21.2	4.2	4.2	100%	2.1	50%	Poor	R4	Moderately Weathered to Slightly Weathered	Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft-bgs, approximately 1 inch thick and subhorizontal.
			R1	11.5	16.5	5.0	4.2	83%	2.3	45%	Poor	R4	Moderately Weathered to Slightly Weathered	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
BB-TMB-102	11.4	R2	16.5	19.8	3.3	3.2	98%	1.1	33%	Poor	R4	Moderately Weathered to Slightly Weathered	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Calcite veins up to 1/4 inch thick common. Highly fractured zone 17.5 to 18 ft-bgs.	
			R3	19.8	21.5	1.7	1.4	85%	1.1	65%	Fair	R4	Moderately Weathered to Slightly Weathered	Dark gray to greenish-black, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite.
			R1	20.0	25.0	5.0	3.0	60%	1.6	32%	Poor	R3	Moderately Weathered to Slightly Weathered	Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered, highly fractured, discontinuities close to very closely spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating.
BB-TMB-103	11.9	R2	25.0	30.0	5.0	2.3	45%	0.3	7%	Very Poor	R3	Moderately Weathered to Slightly Weathered	Dark gray to greenish-black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered, highly fractured, discontinuities close to very closely spaced, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs.	

REF 1
ATT 0

- Notes:
- As-drilled elevations surveyed by VHB.
 - TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run in inches measured along the core axis. The percent RQD is the total length of RQD measured versus the run length.
 - RQD = rock quality designation. RQD is the total length of core pieces recovered with a length greater than or equal to twice the core diameter (4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length.
 - Rock Mass Rating System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. For the rating adjustment for joint orientation, values of Fair (-7) were typically assigned for gently dipping, Fair to Unfavorable (-11) for moderately to steeply dipping, and Unfavorable (-15) for steeply dipping discontinuity angles observed in the core samples. Correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table. Reference Appendix C for the RMR calculations.
 - ft=feet, msl=mean sea level

Prepared By: MRH
 Checked By:
 Reviewed By:



ATTACHMENT D
 REF 4. 0-1/33

Table 4: Summary of Laboratory Rock Core Testing Results
Preliminary Geotechnical Investigation
Clark Bridge
Tremont, Maine
MaineDOT Win: 19305.00

DRAFT

Test Boring Designation ¹	Existing Ground Surface Elevation ² (ft-msl)	Top of Bedrock Elevation ^{2,3} (ft-msl)	Sample Number	Approximate Sample Depth Below Top of Bedrock (ft)	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation (ft-msl)	Uniaxial Compressive Strength ⁴ (psi)	Point Load Index Test ⁵ (psi)
BB-TMB-101	11.2	0.4	R1	2.5 to 3.1	13.3 to 13.9	-2.1 to -2.7	-	26,800
			R2	7.4 to 7.8	18.2 to 18.6	-7.0 to -7.4	31,567	-
BB-TMB-102	11.4	0.3	R2	8.0 to 8.4	18.8 to 19.2	-7.6 to -8.0	-	37,000
			R2	8.1 to 8.5	19.2 to 19.6	-7.8 to -8.2	13,189	-
BB-TMB-103	11.9	-6.4	R1	2.5 to 2.9	20.8 to 21.2	-8.9 to -9.3	-	11,900
			R2	2.9 to 3.4	21.2 to 21.7	-9.3 to -9.8	-	18,800

Notes:

1. Test boring locations are shown on Figure 2, Boring Location Plan.
2. As-drilled locations and elevations surveyed by VHB. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
3. Top of bedrock refers to rock based on depth to casing refusal or observed rock surface in core barrel. Notes concerning weathered rock, if present, are provided on the boring logs.
4. Test performed based on ASTM D7012 - *Compressive Strength and Elastic Moduli on Intact Rock Core Specimens Under Varying States of Stress and Temperatures*.
5. Test performed based on ASTM D5731 - *Point Load Index*. Diameter Test Type. Value listed is the estimated compressive strength based on the point load index determined from the test.
6. Laboratory testing performed by GeoTesting Express, Inc.
7. Laboratory test results for rock core testing are provided in Appendix D.
8. ft = feet; psi = pounds per square inch; msl=mean sea level.

Prepared By: _____
 Checked By: _____
 Reviewed By: _____

D-2/33



Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-101
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 2" and 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 2" SS, 300/16 3" SS	
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>	

Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample attempt
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Insitu Vane Shear Test attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. hammer
 WOR/C = Weight of rods or casing
 WIP/W2P = Weight of one/two person(s)
 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
 N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 N₆₀ = (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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.0 - 0.3 0.3 - 0.5 0.5 - 2.5	15/11/14/12	25	25	SSA	10.4	0-0.3 ft. Asphalt wearing course 0.3-0.5 ft. Asphalt base course Top 1": Weathered asphalt base course		
								8.0	Bottom 11": Light and dark brown, damp, medium dense, gravelly fine to medium SAND, little silt, trace clay (Roadway Base). a=Advanced casing with HW spinning shoe. Lost water at 3 ft., cobble fill at 3.5 ft based on drill behavior. Intermittent water return to 5 ft. No recovery, resampled with 3" spoon. Gray, damp 3" cobble, little gravel, trace silt (Rock Fill). Boulder approximately 7 to 8 ft. based on drill behavior. Cobble fill below based on drill behavior. Intermittent water return 8 to 10 ft.		
5	2MD	24/0	5.0 - 7.0	7/7/26/9	33	33					
10	3D	1/1	10.0 - 10.1	50(1") _b				0.2	b=Used 3" split spoon. Gray, wet, coarse GRAVEL, trace coarse sand, trace silt (Rock Fill). Top of Bedrock Elev. ___ ft. Roller coned ahead to 12 ft. Seam based on drill behavior 11.8 to 11.9 ft. R1: 12.0-12.9: Dark gray to greenish black, very fine grained BASALT, hard, moderately to slightly weathered, discontinuities close to very close, moderately dipping to steep, rough to very rough, planar to irregular. 12.9-17.0: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Rock Mass Quality = Poor R1. Core Times (min:sec) 12.0-13.0: (2:55) 13.0-14.0: (2:30) 14.0-15.0: (2:15) 15.0-16.0: (2:40) 16.0-17.0: (3:30) Recovery = 100% R2: Dark pinkish gray, fine to medium grained GRANITE GNEISS, hard, moderately to slightly weathered, discontinuities close to very closely spaced, generally orthogonal, horizontal to vertical, planar, very rough to smooth, contain calcite and manganese oxide. Thin basalt dike at 17.8 ft., approximately 1 inch thick and subhorizontal. Rock Mass Quality = Poor		
	R1	60/60	12.0 - 17.0	RQD = 42%				-10.2			
15											
	R2	50/50	17.0 - 21.2	RQD = 50%							
20											
25											

Remarks:

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

D-3/33

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-101 PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 2" and 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 140/30 2" SS, 300/16 3" SS	
Date Start/Finish: 5/14-15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	
Hammer Efficiency Factor: 0.6	Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>		

Definitions: R = Rock Core Sample, SSA = Solid Stem Auger, HSA = Hollow Stem Auger, RC = Roller Cone, WOH = Weight of 140lb hammer, WOR/C = Weight of rods or casing, WIP/W2P = Weight of one/two person(s)
 S_u = Instu Field Vane Shear Strength (psf), T_v = Pocket Torvane Shear Strength (psf), q_u = Unconfined Compressive Strength (ksf), N_{uncorrected} = Raw field SPT N-value, Hammer Efficiency Factor = Annual Calibration Value, N₆₀ = SPT N-uncorrected corrected for hammer efficiency, N₆₀ = (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 Information										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)					
25												R2: Core Times (min:sec) 17.0-18.0': (3:35) 18.0-19.0': (4:15) 19.0-20.0': (4:20) 20.0-21.0': (3:55) 21.0-21.2': Not timed. Recovery = 100%	
30												Bottom of Exploration at 21.2 feet below ground surface.	
35													
40													
45													
50													

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement		Boring No.: BB-TMB-102	
		Location: Tremont, Maine		PIN: 19305.00	
Driller:	Maine Test Borings	Elevation (ft.):	11	Auger ID/OD:	4.5 inch
Operator:	M. Porter	Datum:	NAVD88	Sampler:	3" Split Spoon
Logged By:	M. Henrick	Rig Type:	Mobile B-53 Truck	Hammer Wt./Fall:	300/16.3" SS
Date Start/Finish:	5/15/12	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	Not Surveyed	Casing ID/OD:	HW	Water Level*:	8.8
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>			

Definitions: R = Rock Core Sample, SSA = Solid Stem Auger, HSA = Hollow Stem Auger, RC = Roller Cone, WOH = Weight of 140lb. hammer, WOR/C = Weight of rods or casing, WIP/WZP = Weight of one/two person(s)
 S_u = Insitu Field Vane Shear Strength (psf), T_v = Pocket Torvane Shear Strength (psf), q_u = Unconfined Compressive Strength (ksf), N-uncorrected = Raw field SPT N-value, Hammer Efficiency Factor = Annual Calibration Value, N₆₀ = SPT N-uncorrected corrected for hammer efficiency, N₆₀ = (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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.0 - 0.4 0.4 - 0.5 0.5 - 2.5	19/10/11/10	-		SSA	10.3	0-0.4 ft. Asphalt wearing course. 0.4-0.5 ft Asphalt base course 3 inch spoon: Top 3": Weathered asphalt base course		
	2D	1/0	2.5 - 2.6	50(1")			HW _a	8.5	Middle 8": Light brown, damp, sandy fine to coarse GRAVEL, little silt (Roadway Base). Bottom 2": Gray, damp, coarse GRAVEL, litte fine to coarse sand, little silt (Rock Fill).	0.8 2.5	
5	3D	24/0	5.0 - 7.0	7/10/8/1	-				3 inch spoon: No recovery. Rock fill based on drill behavior. HW _a =Advanced HW casing with spinning shoe. No blows performed. Lost water at 4 ft, loose based on drill behavior. 3 inch spoon: No recovery, final blow drove spoon to 7.5 ft., hole stayed open for 4D.		
	4D	24/3	7.5 - 9.5	14/12/6/10	-				6.5-7.5 ft: Loose based on drill behavior, lost water return. Gray, wet, fine to coarse GRAVEL, trace silt (Rock Fill). Dense at 8.5, lost water return. Gray and dark green, wet, fine to coarse GRAVEL, trace fine to coarse sand (Rock Fill).		
10	5D	13/4	10.0 - 11.1	10/26/50(1")				-0.1	Top of Bedrock at Elev. ____ ft. Roller cone ahead to 11.5 ft. Water returned with casing at 11.5 ft. R1: Dark gray to greenish-back, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality: Poor R1: Core Times (min:sec) 11.5-12.5': (2:15) 12.5-13.5': (1:50) 13.5-14.5': (2:10) 14.5-15.5': (2:30) 15.5-16.5': (6:25) Recovery = 83%	11.1	
	R1	60/50	11.5 - 16.5	RQD = 45%			NQ-2				
15											
	R2	40.4/39	16.5 - 19.9	RQD = 33%							
20											
	R3	20/17	19.8 - 21.5	RQD = 68%				-10.5	R2: Dark gray to greenish-back, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Calcite veins up to 1/4 inch thick common. Highly fractured zone 17.5 to 18 ft. deep. Rock Mass Quality = Poor R2: Core Times (min:sec) 16.5-17.5': (1:50)		
25											

Remarks:

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

D-5/33

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: BB-TMB-102 PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS	
Date Start/Finish: 5/15/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.8	
Hammer Efficiency Factor: 0.6	Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathod <input type="checkbox"/>		

Definitions
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample attempt
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Insitu Vane Shear Test attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. hammer
 WOR/C = Weight of rods or casing
 WIP/W2P = Weight of one/two person(s)
 S_u = Insitu Field Vane Shear Strength (psf)
 T_v = Pocket Torvane Shear Strength (psf)
 q_u = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw field SPT N-value
 Hammer Efficiency Factor = Annual Calibration Value
 N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 N₆₀ = (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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in) Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
25											17.5-18.5': (3:35) 18.5-19.5': (2:50) 19.5-19.8': (0:30) Recovery = 98% Core barrel plugged at 19.8 ft, picked up 7" of R1. R3: Dark gray to greenish-back, very fine grained METAVOLCANICS, hard, moderately to slightly weathered, discontinuities close to very close, subhorizontal to steep, smooth to very rough, planar to curved to irregular, contain calcium carbonate, green talc, chlorite and fine grained pyrite. Rock Mass Quality = Fair R3: Core Times (min:sec) 19.8-21.5': (4:15)	
30											Bottom of Exploration at 21.5 feet below ground surface.	
35												
40												
45												
50												

Remarks:

D-6/33

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Clark Bridge Replacement	Boring No.: BB-TMB-103
		Location: Tremont, Maine	PIN: 19305.00
Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch	
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon	
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16.3" SS	
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2	
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9	
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>	

Definitions: R = Rock Core Sample, SSA = Solid Stem Auger, HSA = Hollow Stem Auger, RC = Roller Cone, WOH = Weight of 140lb hammer, WOR/C = Weight of rods or casing, WIP/W2P = Weight of one/two person(s)
 S_u = Insitu Field Vane Shear Strength (psf), T_v = Pocket Torvane Shear Strength (psf), q_u = Unconfined Compressive Strength (ksf), N-uncorrected = Raw field SPT N-value, Hammer Efficiency Factor = Annual Calibration Value, N₆₀ = SPT N-uncorrected corrected for hammer efficiency, N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
 D = Split Spoon Sample, MD = Unsuccessful Split Spoon Sample attempt, U = Thin Wall Tube Sample, MU = Unsuccessful Thin Wall Tube Sample attempt, V = Insitu Vane Shear Test, PP = Pocket Penetrometer, MV = Unsuccessful Insitu Vane Shear Test attempt
 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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/12	0.0 - 0.3 0.3 - 0.5 0.5 - 2.5	26/12/11/22	-		SSA	10.3		Asphalt wearing course. Asphalt base course. 3 inch spoon: Top 3": Weathered asphalt base course.	
								8.5		Bottom 8": Light brown, moist, fine to coarse SAND, little fine gravel, little silt (Roadway Fill).	
5	2D	24/3	5.0 - 7.0	6/10/28/11	-			2.5		Gravel and cobbles based on drill behavior. Lost water return at 3 ft. a = Advanced HW casing with spinning shoe. 3 inch spoon: Gray, damp, single 3" cobble, trace silt. Spoon bent during driving (Rock Fill). No return, cobbles based on drill behavior.	
								0.8			
10								2.0		Wood in wash at approximately 9 to 10.3 ft. No return at 10.4 ft.	
								0.6			
	3D	24/10	11.0 - 13.0	9/12/6/13	-			0.0		3 inch spoon: Gray, wet, fine to coarse GRAVEL, little fine to coarse sand, little silt, trace clay (Emerged Marine Sediments). 3 inch spoon: Gray, wet, fine to coarse GRAVEL, some fine to coarse sand, little silt (Emerged Marine Sediments). b = Drove NW casing Wash turns red-gray at approximately 14.5 ft., alternates to blue-gray. 3 inch spoon: Blue-gray, wet, fine sandy SILT, little fine gravel (Emerged Marine Sediments). HW Casing bent at bottom, rods not fitting through NW casing. Driller removed NW casing and resumed spinning HW.	
	4D	24/7	13.0 - 15.0	48/39/50(5")	-		44 _b 9 ₆	-7.3			
15	5D	6/6	15.0 - 15.5	110(6")						Top of Bedrock at Elev. ____ ft. Roller cone ahead to 20 ft. Casing not seating well at 19 ft. R1: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered, highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs. Rock Mass Quality = Poor R1: Core Times (min:sec) 20.0-21.0': (2:05)	
20	R1	60/36	20.0 - 25.0	RQD = 32%							
25											

Remarks:

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

D-7/33

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Clark Bridge Replacement Location: Tremont, Maine	Boring No.: <u>BB-TMB-103</u> PIN: <u>19305.00</u>
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Driller: Maine Test Borings	Elevation (ft.): 11	Auger ID/OD: 4.5 inch
Operator: M. Porter	Datum: NAVD88	Sampler: 3" Split Spoon
Logged By: M. Henrick	Rig Type: Mobile B-53 Truck	Hammer Wt./Fall: 300/16 3" SS
Date Start/Finish: 5/15-16/12	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: Not Surveyed	Casing ID/OD: HW	Water Level*: 8.9
Hammer Efficiency Factor: 0.6	Hammer Type: Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input checked="" type="checkbox"/>	

Definitions D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. hammer WOR/C = Weight of rods or casing WIP/W2P = Weight of one/two person(s)	S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
		S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Sample Information										Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (16 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log				
25	R2	60/27	25.0 - 30.0	RQD = 7%			V	-19.0	-	21.0-22.0': (2:20) 22.0-23.0': (2:10) 23.0-24.0': (2:40) 24.0-25.0': (5:30) Recovery = 60% R2: Dark gray to greenish black, fine grained, METAVOLCANICS, moderately hard, slightly to moderately weathered highly fractured, discontinuities close to very close, horizontal to steep, rough to very rough, planar to irregular, some calcite coating, highly to completely weathered zone from 24 to 24.5 ft-bgs. Rock Mass Quality = Very Poor R2: Core Times (min:sec) 25.0-26.0': (2:10) 26.0-27.0': (2:05) 27.0-28.0': (2:35) 28.0-29.0': (2:15) 29.0-30.0': (1:40) Recovery = 45% Bottom of Exploration at 30.0 feet below ground surface.			
30													
35													
40													
45													
50													

Remarks:

D-8/33

TABLE 1
DISCONTINUITY SUMMARY
CLARK BRIDGE REPLACEMENT PROJECT
TREMONT, MAINE

Discontinuity		Dip [degrees]	Dip Direction uncorrected [degrees]	Dip Direction State Plane Grid North ⁽¹⁾ [degrees]	Persistence [ft]	Aperture [in]	Infilling	Strength ⁽²⁾	Surface Roughness	Fracture Shape	Water	Fracture Spacing [ft]
ID	Type											
1	joint	85	161	144	6+	0.1	none	medium	smooth	planar	dry	0.2
2	joint	73	205	188	10+	0.1	none	medium	smooth	planar	dry	0.8
3	joint	89	85	68	50+	0.1	chlorite(?)	medium	smooth	planar	dry	0.4
4	joint	87	227	210	10+	0.2	none	medium	smooth	planar	dry	1
5	joint	47	210	193	<1	--	none	medium	smooth	planar	dry	5
6	joint	84	284	267	50+	0.25	none	medium	smooth	planar	dry	1
7	joint	89	47	30	5	<0.1	none	medium	smooth	planar	wet	0.5
8	joint	82	322	305	15+	0.1	none	medium	smooth	planar	dry	0.2
9	joint	89	215	198	5	<0.1	none	medium	smooth	planar	dry	0.1
10	joint	7	359	342	20	<0.1	none	high	very rough	irregular-planar	damp	2
11	joint	71	201	184	5	<0.1	none	medium	smooth	planar	damp	2
12	joint	83	151	134	4	<0.1	none	medium	smooth	planar	dry	0.5
13	joint	82	335	318	5	<0.1	none	medium	smooth	planar	dry	1
14	joint	80	194	177	4	<0.1	none	medium	smooth	planar	dry	4
15	joint	66	152	135	3	<0.1	none	medium	smooth	planar	dry	0.2
16	joint	86	198	181	5	<0.1	none	medium	smooth	planar	dry	1
17	joint	73	332	315	10	<0.1	none	medium	smooth	planar	dry	1
18	joint	89	234	217	1	<0.1	none	low-medium	smooth	planar	dry	1.5
19	joint	85	168	151	10	<0.1	none	medium	smooth	planar	wet	0.5
20	joint	73	179	162	5	<0.1	none	medium	smooth	planar	dry	0.25
21	joint	71	275	258	25	<0.1	none	medium	smooth	planar	wet	0.5
22	joint	83	33	16	15	<0.1	none	high	rough	planar	dry	1
23	joint	78	292	275	3	<0.1	none	medium	smooth	planar	dry	1
24	joint	81	320	303	15	0.5	sand/fine gravel	low	smooth	planar	wet	2
25	joint	74	265	248	3	<0.1	none	medium	smooth	planar	wet	1
26	joint	74	275	258	6	<0.1	none	medium	smooth	planar	wet	1
27	joint	85	232	215	20	0.1 - 1	quartz	medium	smooth	planar	dry	0.1
28	joint	8	142	125	3	<0.1	quartz	medium	rough	irregular-planar	dry	2
29	joint	81	320	303	10	<0.1	none	medium	rough	irregular-planar	damp	0.75
30	joint	85	331	314	15	<0.1	none	medium	smooth	planar	dry	3
31	joint	55	285	268	3	<0.1	none	medium	smooth	planar	dry	1.5
32	joint	88	215	198	20	<0.1	none	medium	smooth	curved-planar	dry	0.3
33	joint	80	214	197	20	<0.1	quartz	medium	smooth	planar	dry	0.4
34	joint	89	338	321	2	0.1	none	medium	rough	irregular-planar	dry	1.5
35	joint	87	45	28	5	0.2	quartz	medium	smooth	planar	dry	0.4

Prepared by: JES
Checked by: WAD

Notes:
(1) Magnetic declination for site is 17 degrees west of true north.
(2) Shear strength of infilling materials as follows:
Low = friction angle < 20 degrees
Medium = 20 degrees < friction angle < 30 degrees
High = 30 degrees < friction angle
ft = feet; in= inches

D-9/33

5/17/12 Thruses Ses 123-674 to vhs document

TO	TYPE	DIP DIR (PT)	PRESIST (PT)	HYPERMUNE (PT)	SPLIT	SPECTRY	GR	SPTM	SHAPE	SHAPE SURF
1	3	85 161	6+	0.1	none	mod	dry	2"	PL	SM
2	3	73 205	10+	0.1	none	mod	dry	9"	PL	SM
3	3	90 085	50+	0.1	CH1?	mod	dry	5"	PL	SM
4	3	87 127	10+	0.2	none	mod	dry	1'	PL	SM
5	3	47 210	7'	-	none	mod	dry	5'	PL	SM
6	3	84 284	50+	0.25"	none	mod	wet	1'	PL	SM
7	3	90 047	5'	<0.1"	none	mod	dry	0.0'	PL	SM
8	3	82 322	15+	0.1"	none	mod	dry	0.12'	PL	SM
9	3	70 215	5'	<0.1"	none	mod	dry	0.1'	PL	SM
10	3	07 359	20'	<0.1"	none	high	dmp 2-3'	ERILE VR. ←	PL	SM
11	3	71 201	5'	<0.1"	none	mod	dmp 2'	ERILE VR. ←	PL	SM

5/17/12 Thruses Ses 123-874 to vhs document

TO	TYPE	DIP DIR (PT)	PRESIST (PT)	HYPERMUNE (PT)	SPLIT	SPECTRY	GR	SPTM	SHAPE	SHAPE SURF
12	3	83 151	4'	<0.1"	none	mod	dry	0.5'	PL	SM
13	3	82 375	5'	<0.1"	none	mod	dry	1.0'	PL	SM
14	3	80 194	4'	<0.1"	none	mod	dry	4'	PL	SM
15	3	76 152	3'	<0.1"	none	mod	dry	2"	PL	SM
16	3	86 198	5'	<0.1"	none	mod	dry	1'	PL	SM
17	3	73 332	10'	<0.1"	none	mod	dry	1'	PL	SM
18	3	70 234	1'	<0.1"	none	mod-low	wet 1.5'	PL	SM	
19	3	85 168	10'	<0.1"	none	mod	dry	0.5'	PL	SM
20	3	73 179	5'	<0.1"	none	mod	wet 0.25'	PL	SM	
21	3	71 275	25'	<0.1"	none	high	dry 0.5'	PL	R	
22	3	83 033	15'	<0.1"	none	mod	dry	1'	PL	SM
23	3	78 292	3'	<0.1"	none	mod	dry	1'	PL	SM
24	3	81 320	15'	0.5"	none	mod	wet 2'	PL	SM	

5/17/12

5/17/12 Thurs 50g 123-87440 VHS Treatment

ID	Type	OID	OID (CFT)	Appr (CFT)	Profile	Connect	Con	Sprink	Sixers	Sheet
28	J	74 265	3'	<0.1"	nom	mod	under water	1'	PL	SM
26	J	74 276	1'	<0.1"	nom	mod	"	1'	PL	SM
27	J	85 232	20'	0.1-1"	2ft	mod	dry	0.1'	IR-PL	SM
28	J	08 142	3'	<0.1"	nom	hard	dry	2'	IR-PL	R
29	J	81 320	10'	<0.1"	nom	mod	dry	0.75'	PL	R
30	J	85 331	15'	<0.1"	nom	mod	dry	3'	PL	SM
31	J	55 285	3'	0.1"	nom	mod	dry	1.5'	PL-CR	SM
32	J	88 245	20'	<0.1"	nom	mod	dry	0.3'	PL	SM
33	J	80 214	20'	0.3"	4ft	mod	dry	0.4'	PL-CR	R
34	J	84 338	2'	0.1"	nom	mod	dry	1.5'	PL	SM
35	J	87 045	5'	0.2"	3ft	mod	dry	0.4'	PL	SM

5/17/12 Thurs 50g 123-87440 VHS Treatment

- Dry. At sil e 1245, drive to MH (home)

mil = 126057

folks: ME 125 NH. 0.75

1.75

2.00

mil = 126360



Search NGDC Search NOAA Go

Navigation menu: Data, Declination, FAQ, SPIDR, Geomagnetism home, Models & Software, Space Weather, WMM, Web Links. Breadcrumbs: NOAA > NESDIS > NGDC > Geomagnetism. Links: comments | privacy policy

Estimated Value of Magnetic Declination

To compute the magnetic declination, you must enter the location and date of interest.

Checkout our new online calculators!
This calculator will be phased out May 2012.

If you are unsure about your city's latitude and longitude, look it up online! In the USA try entering your zip code in the box below or visit the [U.S. Gazetteer](#). Outside the USA try the [Getty Thesaurus](#).

Search for a place in the USA by Zip Code:

Enter Location: (latitude 90S to 90N, longitude 180W to 180E). See [Instructions](#) for details.

Latitude: N S Longitude: E W

Enter Date (1900-2015): Year: Month (1-12): Day (1-31):

Declination = 16° 41' W changing by 0° 5' E/year

~ 17° W of N

For more information, visit:

Answers to some [frequently asked questions](#) | [Instructions](#) for use | [Today's Space Weather](#)



0-12/33

Compass shows the approximate bearing of the magnetic north (MN)

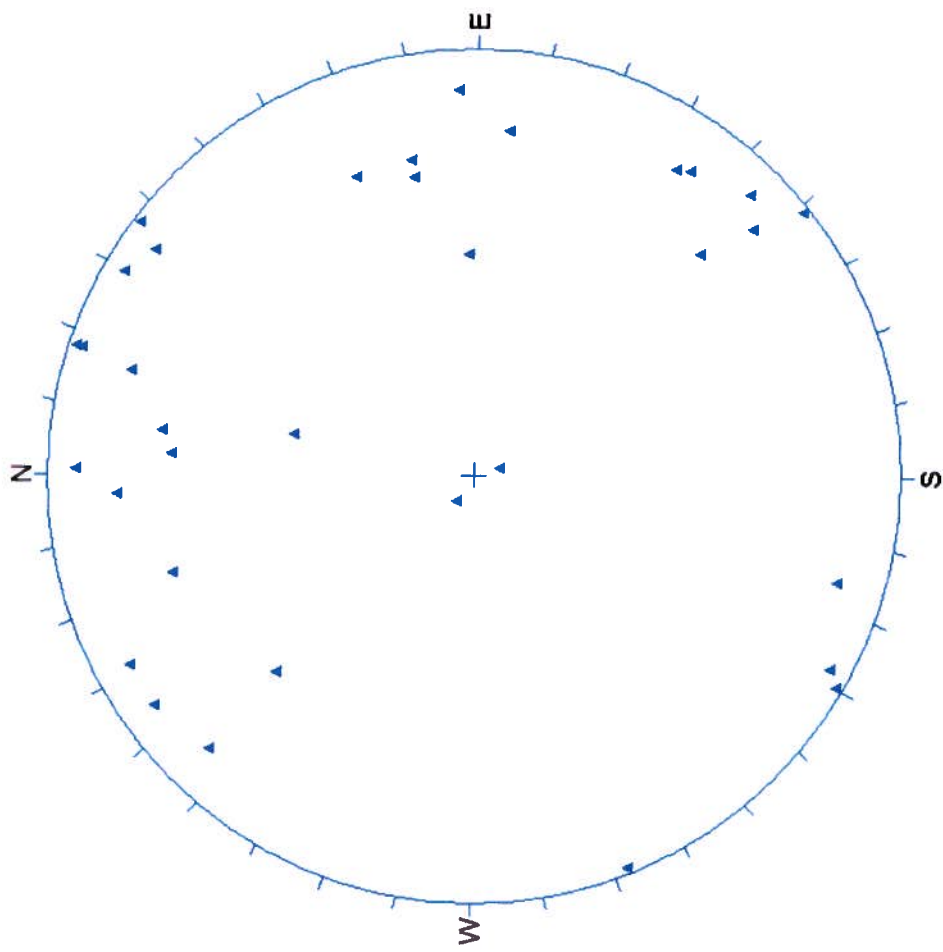
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Questions: geomag.models@noaa.gov

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ID	LAT	LONG	DEC LAT	DEC LONG	LAT DD MM SS.SSSSS	LONG DD MM SS.SSSSS
BBTMB101	N44-15.173	W68-20.949	44.25288	68.34915	10.38000 44 15 10.38	20.94900 56.94000 068 20 56.94
BBTMB102	N44-15.172	W68-20.952	44.25287	68.34920	10.32000 44 15 10.32	20.95200 57.12000 068 20 57.12
BBTMB103	N44-15.177	W68-20.958	44.25295	68.34930	10.62000 44 15 10.62	20.95800 57.48000 068 20 57.48
BBTMB104	N44-15.176	W68-20.962	44.25293	68.34937	10.56000 44 15 10.56	20.96200 57.72000 068 20 57.72
BBTMB105	N44-15.170	W68-20.936	44.25283	68.34893	10.20000 44 15 10.20	20.93600 56.16000 068 20 56.16
BBTMB106	N44-15.166	W68-20.932	44.25277	68.34887	9.96000 44 15 09.96	20.93200 55.92000 068 20 55.92
D1	N44-15.146	W68-20.961	44.25767	68.34935	27.60000 44 15 27.60	20.96100 57.66000 068 20 57.66
D2	N44-15.171	W68-20.927	44.25285	68.34878	10.26000 44 15 10.26	20.92700 55.62000 068 20 55.62
D3	N44-15.186	W68-20.915	44.25310	68.34858	11.16000 44 15 11.16	20.91500 54.90000 068 20 54.90
Seis Ln E	N44-15.164	W68-20.926	44.25273	68.34877	9.84000 44 15 09.84	20.92600 55.56000 068 20 55.56
Seis Ln W	N44-15.182	W68-20.971	44.25303	68.34952	10.92000 44 15 10.92	20.97100 58.26000 068 20 58.26

ID	LAT	LONG	NORTHING(EASTING(X) METER	NORTHING(Y) FEET	EASTING(X) FEET
BBTMB101	44 15 10.38	068 20 56.94	65139.94	312046.39	213724.15 1023824.21
BBTMB102	44 15 10.32	068 20 57.12	65138.08	312042.40	213718.05 1023811.12
BBTMB103	44 15 10.62	068 20 57.48	65147.33	312034.40	213748.38 1023784.87
BBTMB104	44 15 10.56	068 20 57.72	65145.46	312029.08	213742.27 1023767.41
BBTMB105	44 15 10.20	068 20 56.16	65134.42	312063.70	213706.02 1023881.01
BBTMB106	44 15 09.96	068 20 55.92	65127.02	312069.04	213681.75 1023898.53
D1	44 15 27.60	068 20 57.66	65671.37	312029.45	215467.76 1023768.61
D2	44 15 10.26	068 20 55.62	65136.29	312075.68	213712.17 1023920.30
D3	44 15 11.16	068 20 54.90	65164.10	312091.60	213803.40 1023972.54
Seis Ln E	44 15 09.84	068 20 55.56	65123.33	312077.03	213669.65 1023924.75
Seis Ln W	44 15 10.92	068 20 58.26	65156.55	312017.08	213778.65 1023728.04

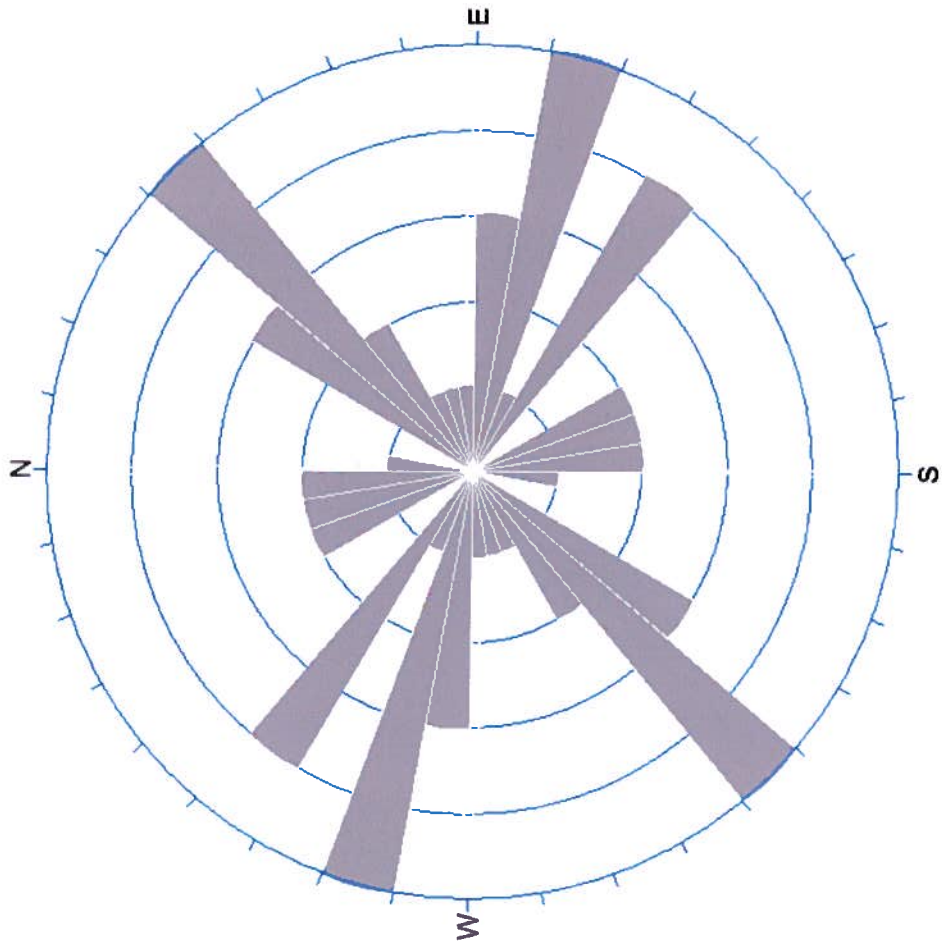


TYPE

joint [35]



Equal Angle
Lower Hemisphere
35 Poles
35 Entries

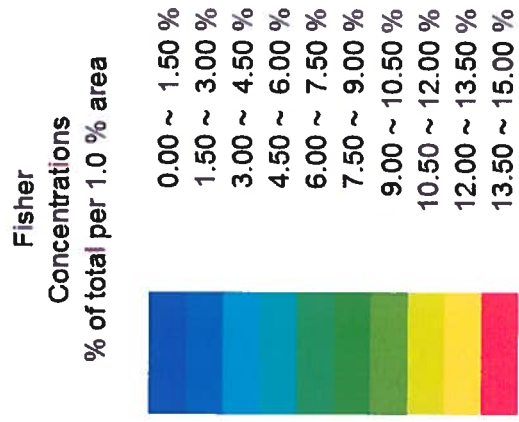
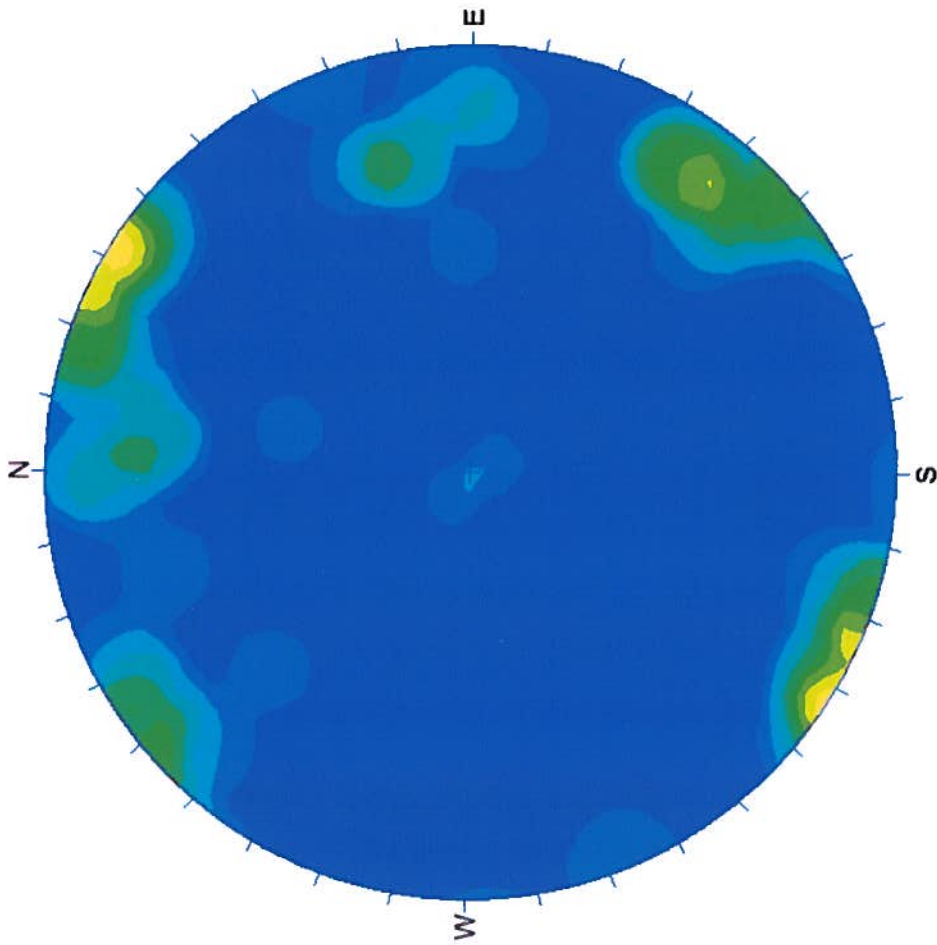


Apparent Strike
5 max planes / arc
at outer circle

Trend / Plunge of
Face Normal = 0, 90
(directed away from viewer)

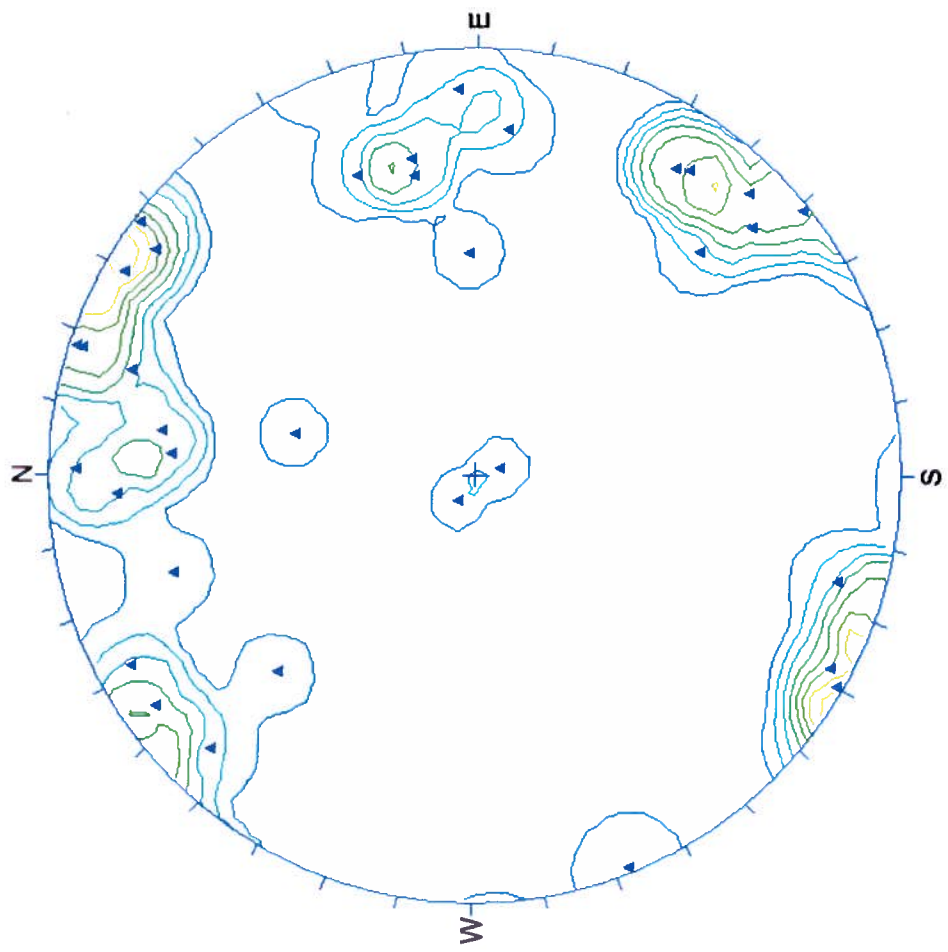
No Bias Correction

33 Planes Plotted
Within 45 and 90
Degrees of Viewing
Face



No Bias Correction
Max. Conc. = 13.0342%

Equal Angle
Lower Hemisphere
35 Poles
35 Entries

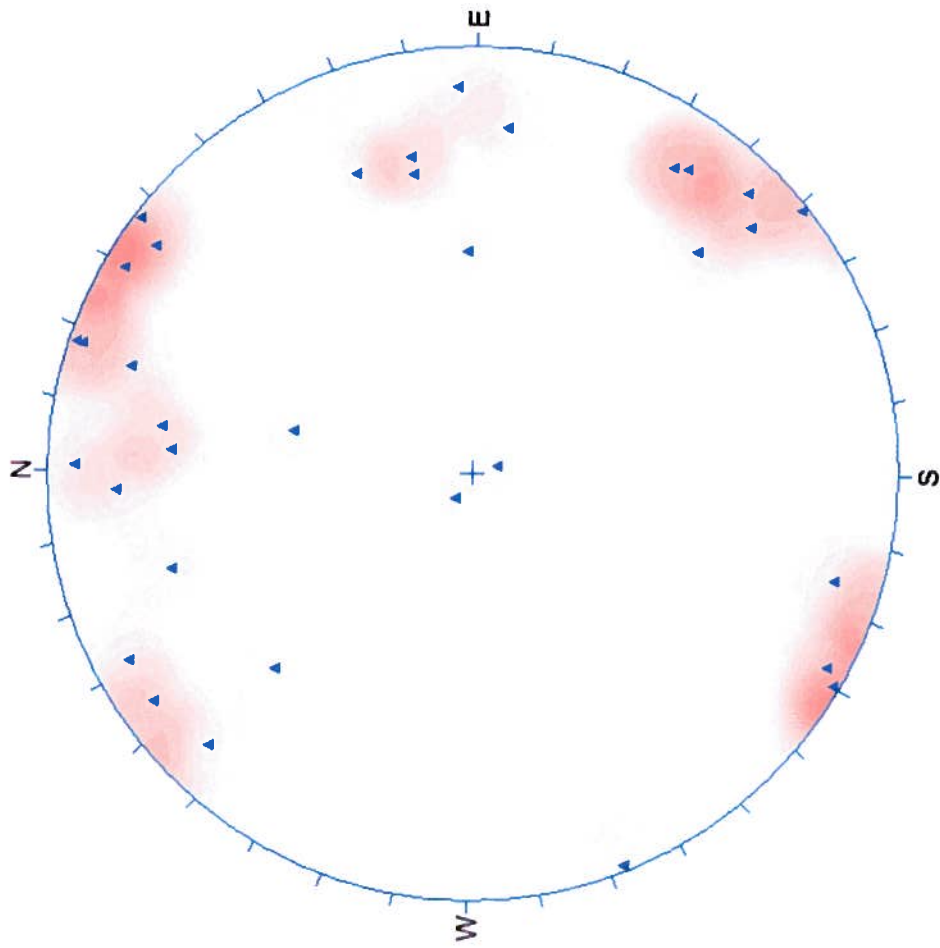


TYPE

joint [35]



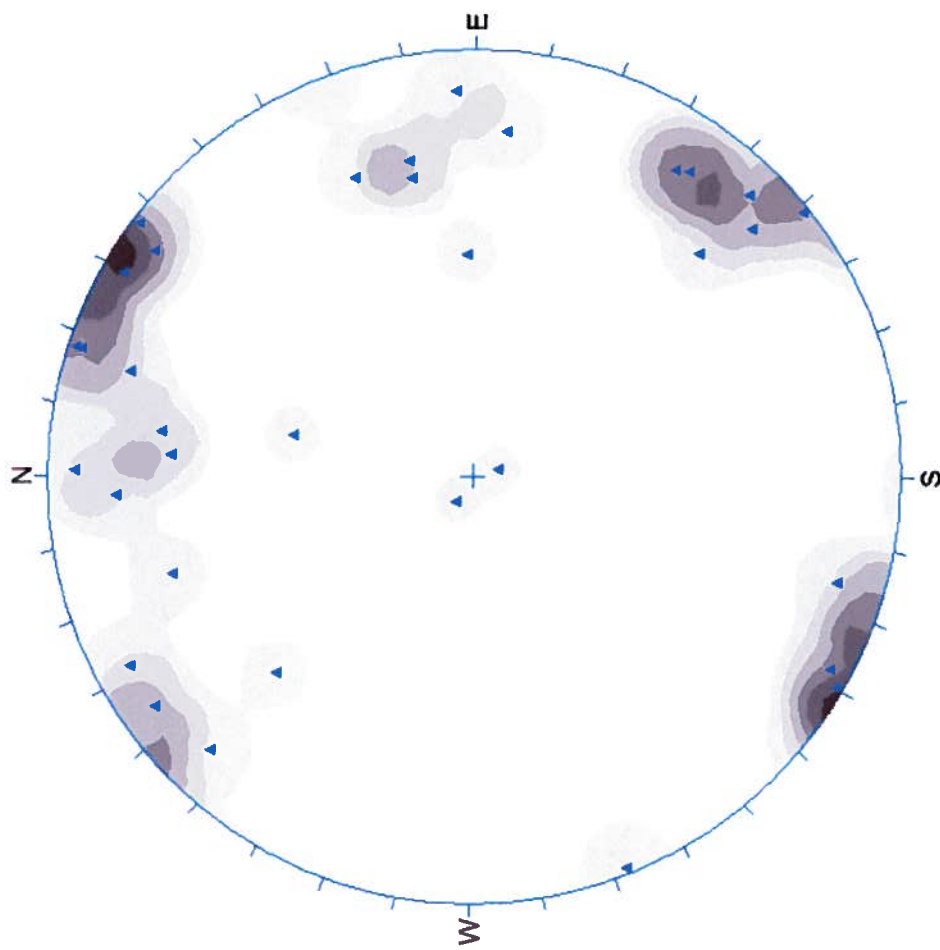
Equal Angle
 Lower Hemisphere
 35 Poles
 35 Entries



TYPE

joint [35]

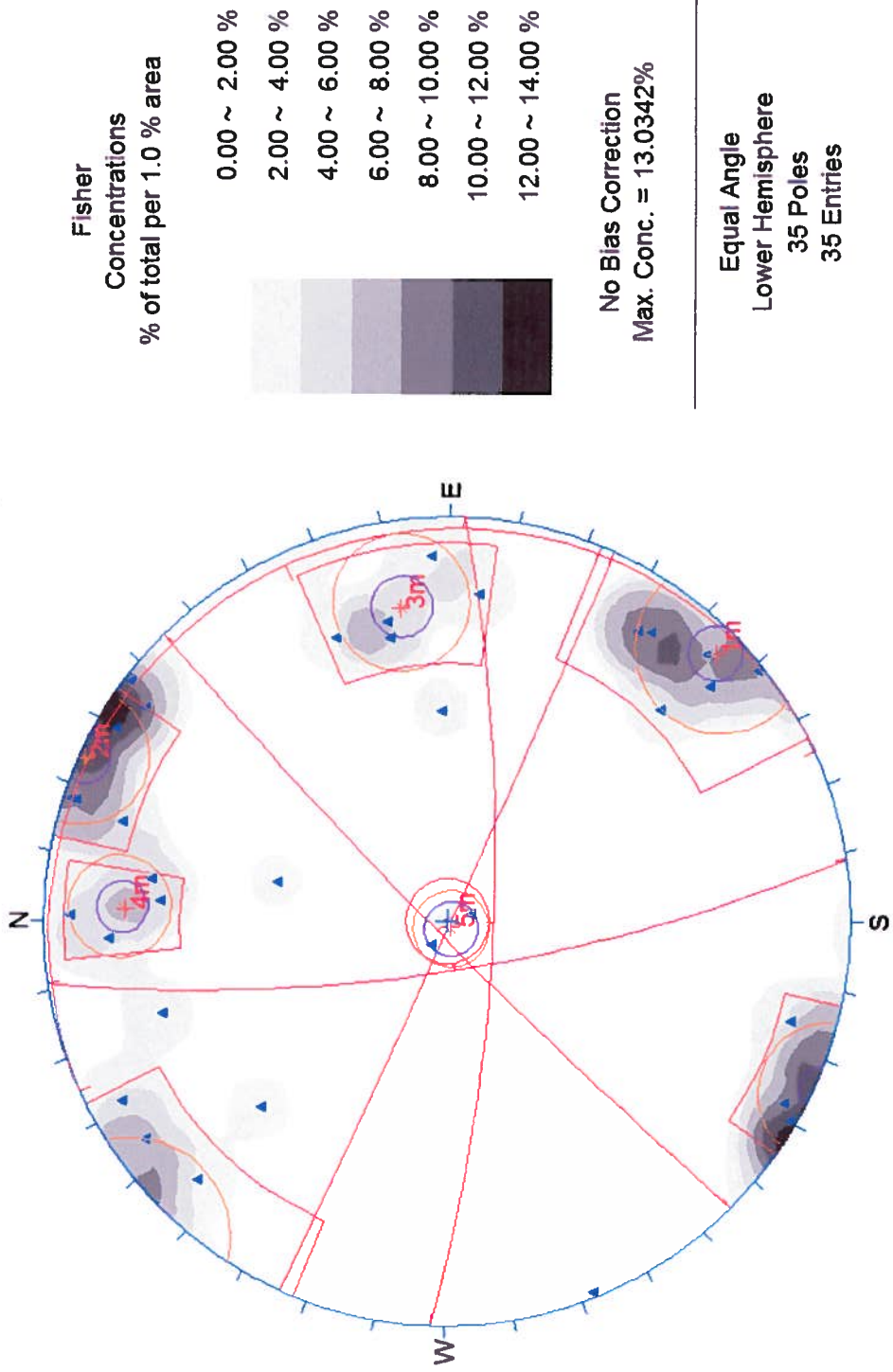
Equal Angle
 Lower Hemisphere
 35 Poles
 35 Entries

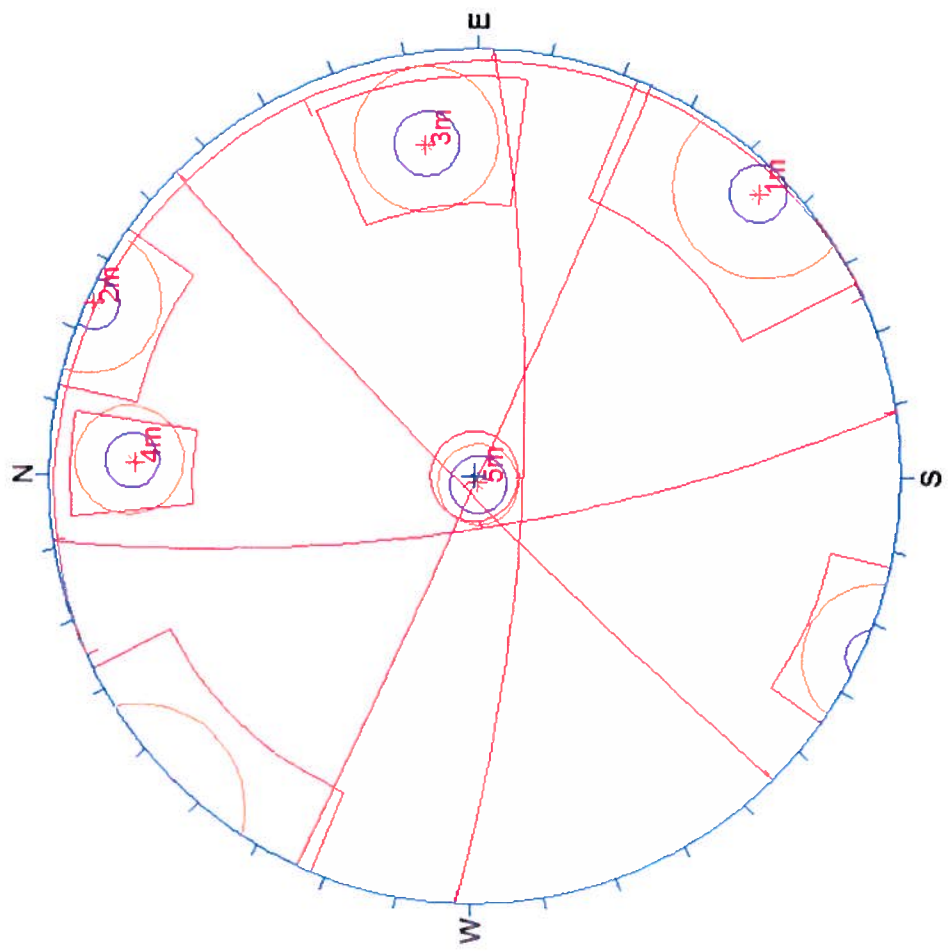


TYPE

▲ joint [35]

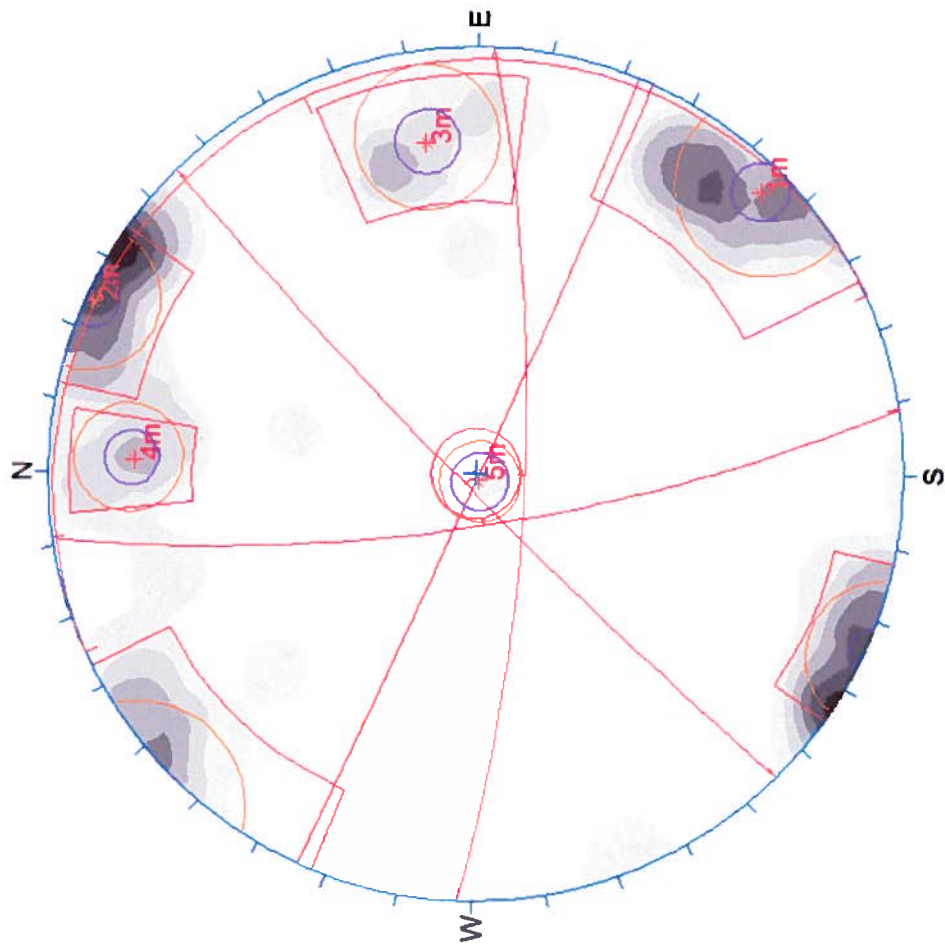
Equal Angle
Lower Hemisphere
35 Poles
35 Entries





Orientations	
ID	Dip / Direction
1	m 86 / 315
2	m 89 / 204
3	m 76 / 261
4	m 77 / 182
5	m 02 / 065

Equal Angle
Lower Hemisphere
35 Poles
35 Entries



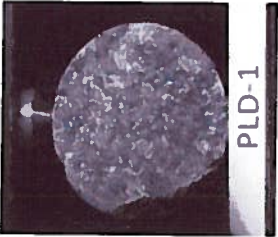

Orientations	
ID	Dip / Direction
1	m 86 / 315
2	m 89 / 204
3	m 76 / 261
4	m 77 / 182
5	m 02 / 065

Equal Angle
Lower Hemisphere
35 Poles
35 Entries

Client:	Golder Associates	Test Date:	06/12/12
Project Name:	VHB Tremont	Tested By:	jsc
Project Location:	Tremont, ME	Checked By:	mpd
GTX #:	11871	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Specimen Diameter (D), in.	Specimen Length (L), in.	Failure Load (P), lbs.	D_e^2 , in ²	D_e , in.	I_s , psi	F	$I_{s(50mm)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-101	R1	13.30-13.90	PLD - 1	Diametral	2.00	6.09	4640	3.98	2.00	1166	1.006	1173	23	26,800

<p>PLD - 1 before</p> 	<p>PLD - 1 after</p>  <p style="text-align: center;">Intact material failure</p>
---	--

Notes:

- Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
- D_e = the equivalent core diameter
- I_s = the uncorrected point load strength index
- F = the size correction factor
- $I_{s(50)}$ = the size corrected point load strength index

Client: Golden Associates	Test Date: 06/12/12
Project Name: VHB Tremont	Tested By: Jsc
Project Location: Tremont, ME	Checked By: m pd
GTX #: 11871	Sample Type: rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No	Sample No	Depth, ft	Test No	Test Type	Specimen Diameter (D), in	Specimen Length (L), in	Failure Load (P), lbs	D _e ² , in ²	D _e , in	I _c , psi	F	I _{c(50)} , psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-101	R2	18.80-19.20	PLD-2	Diametral	1.99	4.30	6378	3.96	1.99	1609	1.005	1617	23	37,000

PLD - 2 before



PLD - 2 after



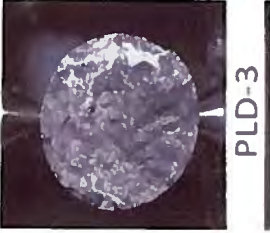

INJECT MATERIAL FAILURE

Notes:

- Generalized Correction Factor, K used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1
- D_e = the equivalent core diameter
- I_c = the uncorrected point load strength index
- F = the size correction factor
- I_{c(50)} = the size corrected point load strength index

Client: Goldier Associates	Test Date: 06/12/12
Project Name: VHB Tremont	Tested By: JAC
Project Location: Tremont, ME	Checked By: mpd
GTX #: 11871	Sample Type: rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No	Sample No	Depth ft	Test No.	Test Type	Specimen Diameter (D), in	Specimen Length (L), in	Failure Load (P), lbs	D _e ² in ²	D _e in	I _c psi	F	I _c (50) psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB TMB 103	R1	20.80-21.20	PLD-3	Diametral	1.99	4.77	2055	3.97	1.99	518	1,005	521	23	11,900
PLD - 3 before														
PLD - 3 after														

Notes:
 Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1
 D_e = the equivalent core diameter
 I_c = the uncorrected point load strength index
 F = the size correction factor
 I_c(50) = the size corrected point load strength index


Client: **Goldier Associates**
 Project Name: **VHB Tremont**
 Project Location: **Tremont, ME**
 GTX #: **11871**

Test Date: **06/12/12**
 Tested By: **JSC**
 Checked By: **MPD**
 Sample Type: **rock core**

Point Load Strength Index of Rock by ASTM D 5731

Boring No	Sample No	Depth, ft	Test No	Test Type	Specimen Diameter (D), in	Specimen Length (L), in	Failure Load (P), lbs	D _s ² , in ²	D _s , in	I _s , psi	F	I _{s(50)} , psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
BB-TMB-103	R2	21.20-21.70	PLD-4	Diametral	2.00	5.65	3248	3.98	2.00	815	1.006	821	23	18,800

PLD - 4
before



PLD - 4
after



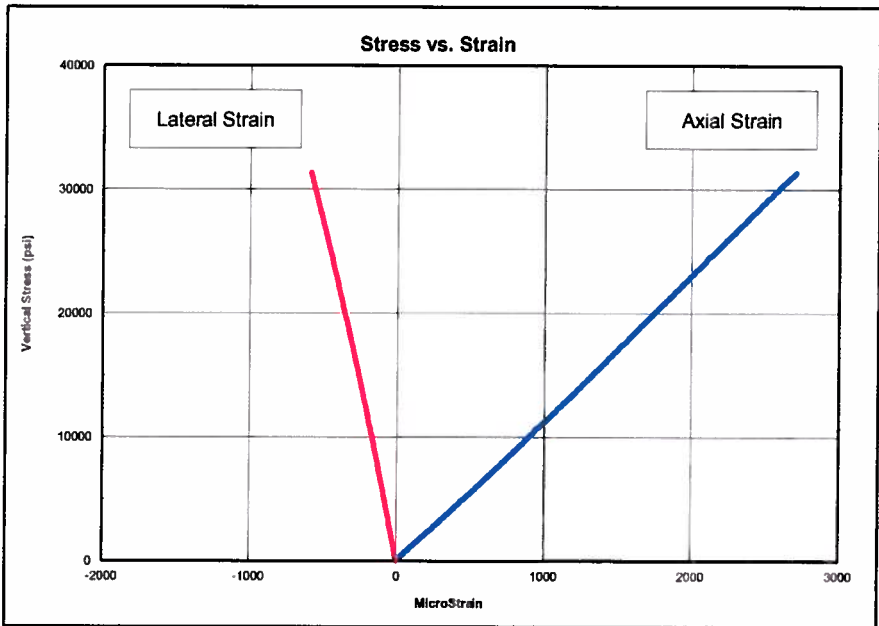
Interaxial Failure

Notes:
 Generalized correction factor, K, used to estimate the compressive strength based on the specimen diameter and ASTM D 5731 Table 1.
 D_s = the equivalent core diameter
 I_s = the uncorrected point load strength index
 F = the size correction factor
 I_{s(50)} = the size corrected point load strength index



Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/8/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-101
Sample ID:	R2
Depth, ft:	18.19-18.56
Sample Type:	rock core
Sample Description:	See photograph Intact material failure

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



Peak Compressive Stress: 31,567 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-10000	11,200,000	0.19
10000-20000	11,800,000	0.22
20000-30000	11,900,000	0.25

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: Golden Associates
 Project Name: VHB Tremont
 Project Location: Tremont, ME
 GTX #: 11871
 Boring ID: BB-TMB-101
 RZ
 Sample ID: 18.19-18.56 R
 Depth: See photographs

Test Date: 6/5/2012
 Tested By: dba
 Checked By: rmpd

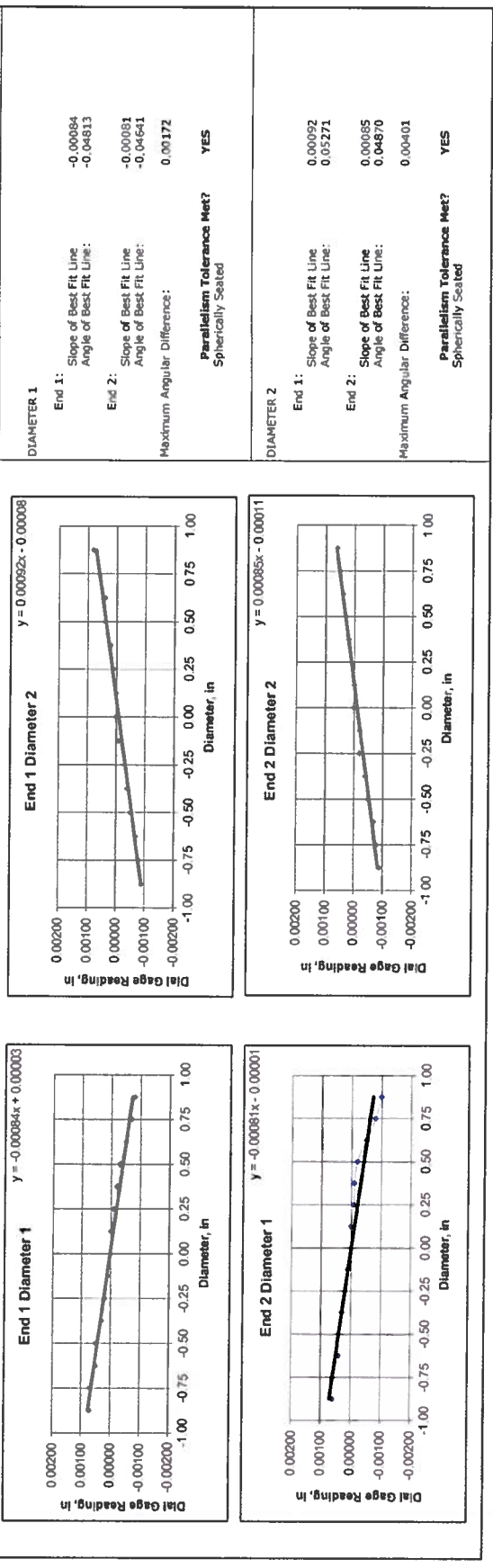
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-08

BULK DENSITY		Average	
1	4.41	2	4.41
Specimen Length, in:	1.99	Specimen Diameter, in:	1.99
Specimen Mass, g:	594.18		
Bulk Density, lb/ft ³ :	165		
Length to Diameter Ratio:	2.2		

Length to Diameter Ratio Tolerance Met?		YES	
END 1	-0.625	-0.500	-0.500
Diameter 1, in	0.00070	0.00050	0.00050
Diameter 2, in (rotated 90°)	-0.00090	-0.00070	-0.00050
END 2	-0.875	-0.750	-0.500
Diameter 1, in	0.00060	0.00060	0.00040
Diameter 2, in (rotated 90°)	-0.00090	-0.00080	-0.00070

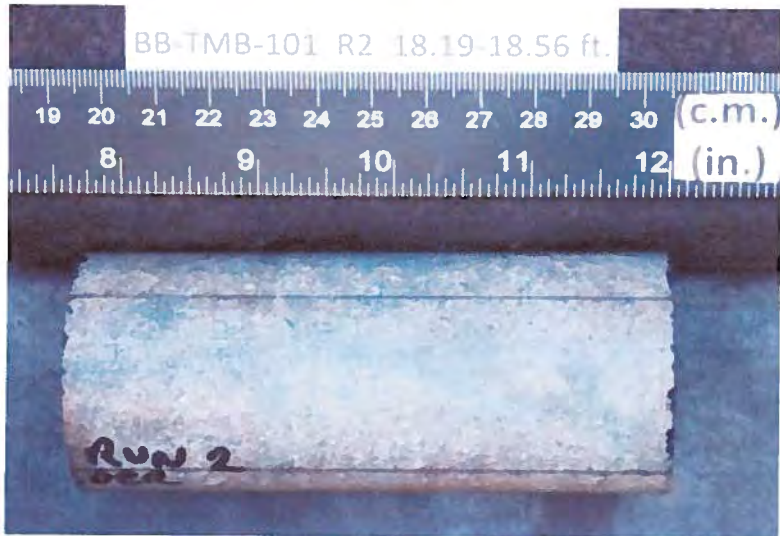
END FLATNESS AND PARALLELISM (Procedure FP1)		Length to Diameter Ratio Tolerance Met?		YES	
Diameter 1, in	0.00070	-0.625	-0.500	-0.375	-0.250
Diameter 2, in (rotated 90°)	-0.00090	0.00050	0.00050	0.00030	0.00020
END 1	0.00060	-0.00080	-0.00070	-0.00040	-0.00030
Diameter 1, in	0.00060	0.00060	0.00040	0.00040	0.00020
Diameter 2, in (rotated 90°)	-0.00090	-0.00080	-0.00070	-0.00040	-0.00020

DEVIATION FROM STRAIGHTNESS (Procedure S1)		Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.?		YES	
0° =	0.00150	0.375	0.250	0.125	0.125
90° =	0.00170	-0.00020	-0.00010	0.00000	0.00000
Difference between max and min readings, in:	0.00040	0.00000	0.00000	0.00000	0.00000
Maximum difference must be $<$ 0.020 in.	0.00150	0.375	0.250	0.125	0.125
0° =	0.00150	0.375	0.250	0.125	0.125
90° =	0.00170	-0.00020	-0.00010	0.00000	0.00000
Difference between max and min readings, in:	0.00040	0.00000	0.00000	0.00000	0.00000
Maximum difference must be $<$ 0.020 in.	0.00150	0.375	0.250	0.125	0.125
0° =	0.00150	0.375	0.250	0.125	0.125
90° =	0.00170	-0.00020	-0.00010	0.00000	0.00000
Difference between max and min readings, in:	0.00040	0.00000	0.00000	0.00000	0.00000
Maximum difference must be \pm 0.00085	0.00150	0.375	0.250	0.125	0.125
0° =	0.00150	0.375	0.250	0.125	0.125
90° =	0.00170	-0.00020	-0.00010	0.00000	0.00000
Difference between max and min readings, in:	0.00040	0.00000	0.00000	0.00000	0.00000
Maximum difference must be \pm 0.00085	0.00150	0.375	0.250	0.125	0.125

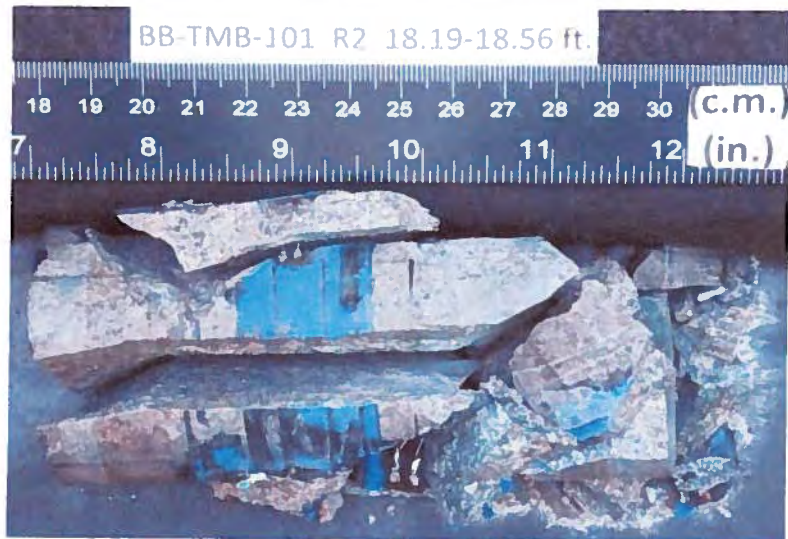


PERPENDICULARITY (Procedure P1)		Maximum angle of departure must be \leq 0.25°	
End 1	0.00150	End 1	0.00160
Diameter 1, in	0.00170	Diameter 1, in	0.00150
Diameter 2, in (rotated 90°)	0.00085	Diameter 2, in (rotated 90°)	0.00150
End 2	0.00160	End 2	0.00160
Diameter 1, in	0.00150	Diameter 1, in	0.00150
Diameter 2, in (rotated 90°)	0.00085	Diameter 2, in (rotated 90°)	0.00150
Perpendicularity Tolerance Met?		Perpendicularity Tolerance Met?	
YES		YES	

Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/8/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-101
Sample ID:	R2
Depth, ft:	18.19-18.56



After cutting and grinding

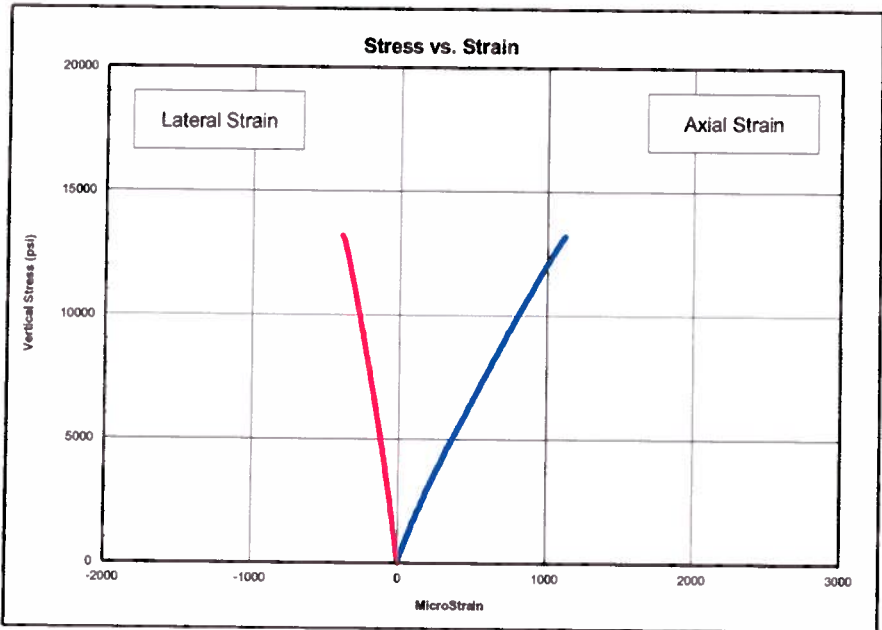


After break



Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/11/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-102
Sample ID:	R2
Depth, ft:	19.20-19.57
Sample Type:	rock core
Sample Description:	See photograph Discontinuity failure

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



Peak Compressive Stress: 13,189 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-4000	13,300,000	0.32
4000-8000	11,600,000	0.33
8000-12000	10,800,000	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.



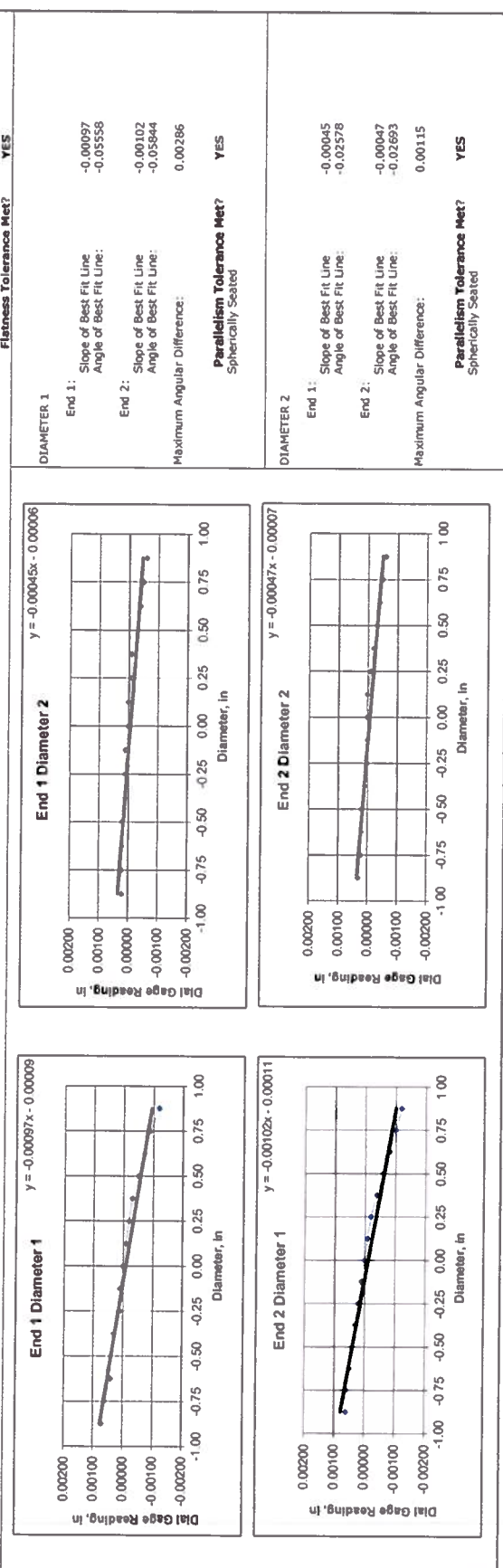
Client: **Golder Associates**
 Project Name: **VHB Tremont**
 Project Location: **Tremont, ME**
 GTX #: **11871**
 Boring ID: **BB-TMB-102**
 Sample ID: **R2**
 Depth: **19.20-19.57 ft**
 Visual Description: **See photographs**

Test Date: **6/8/2012**
 Tested By: **daa**
 Checked By: **mpd**

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-08

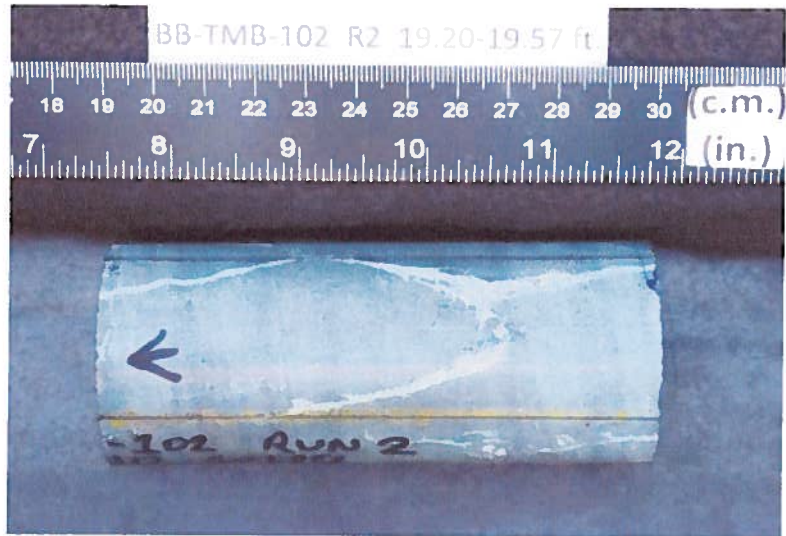
BULK DENSITY		DEVIATION FROM STRAIGHTNESS (Procedure S1)	
Specimen Length, in:	4.36	Average	
Specimen Diameter, in:	1.99	4.36	
Specimen Mass, g:	629.78	1.99	
Bulk Density, lb/ft ³ :	177		
Length to Diameter Ratio:	2.2		

END FLATNESS AND PARALLELISM (Procedure FP1)		Length to Diameter Ratio Tolerance Met?		Straightness Tolerance Met?	
END 1			YES		YES
Diameter 1, in	-0.875	-0.375	-0.250	0.375	0.500
Diameter 2, in (rotated 90°)	0.00070	0.00040	0.00010	-0.00020	-0.00070
	0.00020	0.00020	0.00010	0.00000	-0.00030
					-0.00040
					-0.00060
					-0.00060
END 2			YES		YES
Diameter 1, in	-0.875	-0.375	-0.250	0.375	0.500
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00020	-0.00020	-0.00080
	0.00030	0.00020	0.00010	0.00000	-0.00060
					-0.00040
					-0.00060



PERPENDICULARITY (Procedure P1)		Flatness Tolerance Met?		Parallelism Tolerance Met?	
END 1			YES		YES
Diameter 1, in	0.00190	0.00095	0.055	Slope of Best Fit Line	-0.00097
Diameter 2, in (rotated 90°)	0.00080	0.00040	0.023	Angle of Best Fit Line	-0.05558
				Slope of Best Fit Line	-0.00102
				Angle of Best Fit Line	-0.05844
				Maximum Angular Difference	0.00286
END 2			YES		YES
Diameter 1, in	0.00180	0.00090	0.052	Slope of Best Fit Line	-0.00045
Diameter 2, in (rotated 90°)	0.00090	0.00045	0.026	Angle of Best Fit Line	-0.02578
				Slope of Best Fit Line	-0.00047
				Angle of Best Fit Line	-0.02693
				Maximum Angular Difference	0.00115

Client:	Golder Associates
Project Name:	VHB Tremont
Project Location:	Tremont, ME
GTX #:	11871
Test Date:	6/11/2012
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-TMB-102
Sample ID:	R2
Depth, ft:	19.20-19.57



After cutting and grinding



After break

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY		
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES			
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW Well-graded gravels, gravel-sand mixtures, little or no fines GP Poorly-graded gravels, gravel-sand mixtures, little or no fines	REF 5 ATT E	Descriptive adjective (e.g. silty, sandy, clayey, etc.) Density Cohesionless soils N-value (blows per foot) Very loose 0 - 4 Loose 5 - 10 Medium Dense 11 - 30 Dense 31 - 50 Very Dense > 50	
		GRAVEL WITH FINES (Appreciable amount of fines)	GM Silty gravels, gravel-sand-silt mixtures. GC Clayey gravels, gravel-sand-clay mixtures.			
		SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS			SW Well-graded sands, gravelly sands, little or no fines SP Poorly-graded sands, gravelly sand, little or no fines
			SANDS WITH FINES (Appreciable amount of fines)			SM Silty sands, sand-silt mixtures SC Clayey sands, sand-clay mixtures.
	FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated Consistency of Cohesive soils SPT N-Value blows per foot WOH, WOR, WOP, <2 2 - 4 5 - 8 9 - 15 16 - 30 >30 Approximate Undrained Shear Strength (psf) 0 - 250 250 - 500 500 - 1000 1000 - 2000 2000 - 4000 over 4000 Field Guidelines Fist easily Penetrates Thumb easily penetrates Thumb penetrates with moderate effort Indented by thumb with great effort Indented by thumb nail Indented by thumb nail with difficulty
			CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
OL Organic silts and organic silty clays of low plasticity						
SILTS AND CLAYS (liquid limit greater than 50)		MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
		CH Inorganic clays of high plasticity, fat clays.				
		OH Organic clays of medium to high plasticity, organic silts				
HIGHLY ORGANIC SOILS	PI Peat and other highly organic soils					
Desired Soil Observations: (in this order) Color (Munsell color chart) Moisture (dry, damp, moist, wet, saturated) Density/Consistency (from above right hand side) Name (sand, silty sand, clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc., if applicable) Cementation (weak, moderate, or strong, if applicable, ASTM D 2488) Geologic Origin (till, marine clay, alluvium, etc.) Unified Soil Classification Designation Groundwater level				Rock Quality Designation (RQD): RQD = $\frac{\text{sum of the lengths of intact pieces of core} > 100 \text{ mm}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality Rock Mass Quality RQD Very Poor <25% Poor 26% - 50% Fair 51% - 75% Good 76% - 90% Excellent 91% - 100% Desired Rock Observations: (in this order) Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Lithology (igneous, sedimentary, metamorphic, etc.) Hardness (very hard, hard, mod. hard, etc.) weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90) -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m) -tightness (tight, open or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock mass quality (very poor, poor, etc.) ref. AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A Recovery		
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements: PIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth		



TECHNICAL MEMORANDUM

Date: July 9, 2015
To: Tim Bryant
cc: Meg Melendy, Katy Bouchard
Email: tbryant@vhb.com, kbouchard@vhb.com
From: Mark Peterson
Project No.: 123-87440 01
Company: VHB
**RE: SUPPLEMENTAL PRELIMINARY GEOTECHNICAL CONSIDERATIONS
FINAL PDR SUPPORT
CLARK BRIDGE REPLACEMENT PROJECT
TREMONT, MAINE
MAINEDOT WIN 19305.00**

This memorandum summarizes the findings from supplemental geotechnical assessments for Vanasse Hangen Brustlin, Inc.'s (VHB's) final Preliminary Design Report (PDR) for the Clark Bridge replacement project in Tremont, Maine. Golder completed a preliminary geotechnical investigation for the project in 2012¹ in support of VHB's Preliminary Design Report² submittal in 2012. The purpose of this phase of work was to: 1), revisit bridge foundation support requirements for the west abutment accounting for construction considerations; and 2), reconcile a preferred alternative for a retaining wall system or steep slope supporting the south slope widening of the causeway. Our work was performed in accordance with our revised proposal dated April 2, 2015 and our Agreement dated May 14, 2015.

PROJECT BACKGROUND

Clark Bridge carries State Route 102 over Marshall Brook in Tremont, Maine, and was constructed in 1924 to replace two single span bridges. The bridge is a 15-foot (ft) clear span cast-in-place concrete slab superstructure supported on dry-laid stone masonry abutments. Two narrow causeways with riprap surfaced side slopes support the west and east bridge approaches³, and are about 20 and 100 ft long, respectively. According to the 1924 design drawings, the causeways were constructed of timber cribbing, with field stone used for abutments and retaining walls, and quarried stone rubble masonry for fill materials.

¹ Golder Associates Inc. (2012). "Preliminary Geotechnical Investigation, Clark Bridge Replacement, Route 102, Tremont, Maine, MainDOT WIN 19306.00", December 12, 2012.

² Vanasse Hangen Brustlin, Inc. (2012). "Preliminary Design Report, Clark Bridge #3663, Tremont Road over Marshall Brook, Tremont, ME, WIN 19305.00", October 30, 2012, Revised December 12 & 31, 2012.

³ For purposes of this Memorandum the project North is parallel to the alignment of Marshall Brook and Clark Bridge and the approach causeways are referenced as an east-west orientation.
150709 golder memo for final pdr support r1.docx



Bridge replacement requirements include: 1), maintenance of one-lane of alternating two-way traffic throughout construction; and 2), the new bridge cannot change water elevations or flushing rates in the marsh north of the bridge. Recommendations for the bridge replacement discussed in the 2012 PDR² include a new bridge with the same opening as the existing bridge, and a widened causeway for a sidewalk on both approach roadways. The proposed roadway on the bridge and approaches includes two 11 foot paved lanes with 4 foot shoulders with a 5 foot sidewalk on the widened south side. Current recommendations for the bridge replacement include a 40 ft span prestressed concrete voided slab supported on precast full height cantilever abutments and wingwalls on precast spread footings overlying cast-in-place subfootings bearing on bedrock. Proposed abutments are located about 10 feet behind the existing dry-laid field stone abutments. Widening the south side of the causeway must be completed within a limited right-of-way boundary, and can conceivably be completed with a retaining wall or a steepened reinforced slope. In VHB's PDR², a Prefabricated Concrete Modular Gravity Wall system (PCMG, e.g., T-Wall) was assumed for preliminary cost estimating.

WEST ABUTMENT FOUNDATION SUPPORT CONSIDERATIONS

At the west abutment for the new bridge, cast-in-place concrete subfootings are planned to extend to the bedrock surface at a point starting about 10 ft behind the existing dry-laid field stone abutments. The bedrock surface in this area is interpreted to slope upward to the west (and possibly to the south also) from a low point at about Elev. -5 ft (NAVD88 datum) underlying the existing abutment. The base of the existing west abutment footing is unknown; however, it is possible that the footing may be supported on the layer of very dense emerged marine sediments encountered at the test borings at about Elev 0 ft. Based on soil resistance and frost penetration criteria, a shallower alternative foundation support grade on the very dense gravel and sandy silt sediments at roughly Elev 0 ft was considered for the new west abutment. If considered acceptable, the shallower foundation grade would facilitate the foundation excavation and avoid the risk of undermining the existing abutment foundation during construction.

To assess the feasibility of a shallower foundation subgrade for the west abutment we examined the likely shallowest possible sub-footing subgrade based on soil resistance, reviewed criteria in the MaineDOT Bridge Design Guide regarding requirements for scour protection, and reviewed the 2012 Preliminary Hydrology, Hydraulics and Scout Report⁴. Our conclusions from this assessment include the following:

1. The base of the new sub-footing would need to be founded on soil no higher than (and possibly deeper than) Elev 0 ft based solely on the depth to satisfactory soil resistance. The bedrock surface is estimated to be at about Elev -5 ft at the east edge of the new footing and roughly Elev -2 ft at the west edge.

⁴ Northstar Hydro, Inc. (2012). "Clark Bridge, Route 102, Tremont, Maine, Preliminary Hydrology, Hydraulics and Scout Report", prepared for VHB, Inc. and Maine Dept. of Transportation, Maine DOT WIN # 19305.00, Bridge #3663, October 29, 2012.

2. MaineDOT's Bridge Design Guide contains several references to footing depths on soil at bridge abutments subject to scour. Section 5.3.4.4 indicates spread footings in soil within a stream crossing should be located at least 6 ft below the thalweg of the waterway. Section 2.3.11.1 indicates that footings for new bridges over "waterways" should be placed a minimum of 2 ft below the design flood scour level, and that geotechnical analyses should assume that soils above the design scour level will be removed and not available for support. Section 2.3.11.2 indicates a scour evaluation should be performed if there is a history of scour at the existing bridge. Criteria for designed scour countermeasures are provided, but only for design flows less than 8.8 feet per second (fps). Section 2.3.11.3 regarding riprap indicates that for bridges located immediately on the ocean, or when the average velocity is greater than 12 fps and ice is a concern, heavy riprap 4 ft thick should be used. Although heavy riprap is planned at the channel bed for the new Clark Bridge, it is not clear to us if heavy riprap is considered an acceptable scour countermeasure for supporting the new abutment footing on soil; and even if it is considered acceptable, we assume an analysis as described in FHWA's HEC-23⁵ would be necessary to identify the required footing depth.
3. Scour calculations were not conducted as part of the preliminary hydraulics report⁴ because it was assumed the new abutments would be supported directly on bedrock as discussed in the preliminary geotechnical report¹. Based on modeling using data from a 1978 storm of record, the maximum flow velocity at the existing bridge is estimated to be 13.6 fps.
4. While the design soil scour depth at the new abutment footing has not been analyzed and the foundation configuration is unusual (because the existing footing and abutment wall will remain and velocities at the new footings have not been estimated), we conclude that scour concerns would likely require that the sub-footing for the new abutment be supported on bedrock. In addition, considering that the bedrock surface is interpreted to be only about 2 ft or less below the west side of the sub-footing, practical considerations dictate that the footing should be extended to the bedrock surface to avoid foundation support risk for the new bridge.
5. The sloping rock surface in this area is interpreted to be steeper than 4H:1V and as indicated in the MaineDOT's Bridge Design Guide Section 5.3.4.2, will require benching at the sub-footing footprint to provide a more level bearing surface. Mechanical rock excavation methods are recommended.
6. The temporary excavation support system for the west abutment foundation will need to satisfy several requirements including: installation through timber crib and rockfill materials; satisfactory lateral support for the existing abutment wall foundation; dewatered conditions; phased construction

⁵ Lagasse, P.F., Clopper, P.E., Pagan-Ortiz, J.E., Zevenbergen, L.W., Arneson, L.A., Schall, J.D. and Girard, L.G. (2009). "Bridge Scour and Stream Instability Countermeasures, Volumes 1 and 2", Third Edition, Hydraulic Engineering Circular No. 23, FHWA-NHI-HEC-23, Federal Highway Administration, Washington, DC.

requirements; and accommodation of foundation support requirements for the adjacent return wall for the widened causeway.

WALL SYSTEM ALTERNATIVES FOR CAUSEWAY WIDENING

Expanding the roadway travel lanes and shoulders and adding a sidewalk will widen the guardrail-to-guardrail distance at the top of the causeway by about 11 ft, all of which is planned to be located on the south side of the causeway. To support the added causeway fill and stay within the existing right-of-way (ROW), return retaining walls are planned to extend about 75 ft east of the east abutment and 25 ft west of the west abutment. The PDR² shows a preliminary section for the new wall including a 2 ft high 2H:1V slope from the new guardrail down to the top of the new retaining wall at about Elev 11.5 ft. The location of the new retaining wall is shown on the preliminary plans at about 15 ft south of the crest of the existing southern guardrail, and about 10 ft north of the ROW. The design high tide level at the new wall is Elev. 5.4 ft (MHHW), and the water level modeled for the 1978 storm of record condition⁴ is Elev. 10.2 ft.

The preliminary retaining wall system illustrated in the 2012 PDR² is a PCMG wall (e.g., T-Wall). During the current assessment we considered alternative wall systems and slope configurations that may be applicable for the causeway return walls accounting for stability, the site tidal environment, MaineDOT's Qualified Products List (QPL) of Proprietary Retaining Wall Systems, construction sequencing, and cost. As part of this effort we met with Laura Krusinski of MaineDOT on June 8, 2015 to discuss geotechnical considerations for alternative retaining wall systems and the Department's experience with walls located at coastal sites subject to tidal fluctuations. Conclusions and recommendations from the current assessment are summarized as follows:

1. Based on the interpreted bedrock surface profile at the causeway areas it appears that the bedrock surface is located roughly within 1 to 3 ft of the mudline along the general alignment for the return walls. Accordingly, we assume all return wall foundations will bear directly on rock. An exception might be the eastern end of the west wall where the bedrock surface appears to dip and is overlain by dense to very dense gravel and sandy silt; however, since the bedrock surface in this area is likely to be within 2 to 3 ft (or less) of a wall footing subgrade on soil (assuming scour considerations do not require deeper footings), it is prudent to assume the wall foundation will be extended to bedrock. At the west end of the west wall the ground surface rises and there may be on the order of 7 ft of overburden soil overlying the bedrock, but we recommend this short wall section also be supported directly on bedrock. We note that the slope of rock (both longitudinal and transverse directions) at the wall alignment is not well defined and likely varies from the rock surface profile shown in the preliminary geotechnical report¹ for the centerline of the causeway.
2. The configuration of the retaining wall sections will vary along the length of the walls due to changes in the foundation subgrade (bedrock surface) and the finished road surface grade at the crest of the

causeway. Our preliminary assessment indicates wall heights (assuming full height wall with no soil back slope) may vary from about 10 ft to 12 ft for the return walls on the east and west causeways. General design criteria for gravity walls and earth reinforced walls in this height range indicate the wall width or width of reinforced soil would need to range from about 7 ft to 12 ft, depending on wall type. The preliminary wall alignment and section included in the 2012 PDR² provides sufficient room to accommodate these wall and backfill dimensions within the causeway section designated for Phase 1 construction; however, the temporary bracing system supporting the north half of the causeway fill will need to be laterally supported for most of its height. While the preliminary wall alignment could be shifted further south and stay within the ROW, it is assumed at least 10 ft is required in front of the wall for a 2H:1V riprap slope against the base of the wall.

3. Our assessment of feasible wall systems focused on proprietary precast wall systems assuming they would be cost effective compared to a cast-in-place cantilever retaining wall. We conclude each of the following wall systems (all of which are on the MaineDOT's QPL) are feasible for application to this project with appropriate provisions for backfill materials, filters to prevent loss of backfill soil from fluctuating currents and tide cycles, and material properties suitable for a saltwater corrosive environment:

- a. T-Wall (The Neel Company): A Prefabricated Concrete Modular Gravity Wall (PCMG). This wall has been the most common proprietary wall system used by the MaineDOT in coastal environments subject to tide fluctuations and has reportedly performed well.
- b. Doublewal (United Concrete Products, Inc.): A PCMG wall that could conceivably be used at this site according to manufacturer's literature, but we are not aware of an application where it was used at a Maine coastal site
- c. Redi-Rock Wall (Redi-Rock International): A Precast Concrete Block Gravity Wall (PCBG). The Redi-Rock gravity wall appears to have an approximate height limit of about 10 ft for application to this site based on a brief review of preliminary design guidelines provided by the manufacturer. For higher walls the wall can be modified to a mechanically stabilized earth (MSE) wall with geogrid reinforcement. We understand the MaineDOT has used Redi-Rock walls (both gravity and MSE) at many sites, but not at a coastal environment with a saturated backfill subject to tidal fluctuations.
- d. Stone Strong Systems (Stone Strong, LLC): A Precast Aggregate-Filled Concrete Block Gravity Wall (PACBG). The Stone Strong gravity wall appears to have an approximate height limit of about 9 ft for application to this site based on a brief review of preliminary design guidelines provided by the manufacturer. For higher walls the wall can be modified to a mechanically stabilized earth (MSE) wall with geogrid reinforcement. We understand the

MaineDOT has used Stone Strong walls, but not at a coastal environment with a saturated backfill subject to tidal fluctuations.

4. The applicability of Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) to this site was assessed and discussed with MaineDOT. The GRS-IBS system is a FHWA sanctioned approach for supporting a bridge abutment on a reinforced soil foundation. The reinforced soil foundation is similar to MSE technology, and where the GRS system is used for wingwalls and return walls it is essentially a MSE wall. MaineDOT has had favorable experience with two GRS-IBS installations to date (a third is currently planned), but these included abutment support in addition to wingwall/return wall applications, the abutments were built entirely in a single phase of construction, and the GRS walls were not located in high water velocity sites. For the Tremont site the use of GRS-IBS does not appear to be well-suited because the phased construction would prevent the use of the system for abutment support. When considered solely for use as a retaining wall for the causeway widening we believe there are uncertainties concerning resistance to backfill erosion/piping through or below the facing elements at this site that apply to any MSE wall structure. We do not rule out the possibility that these concerns could be accommodated in design and with diligent attention to detail during construction; however, related added design/construction features may result in a less cost effective wall system.
5. Reinforced Soil Slopes (RSS) were assessed on a conceptual level as an alternative to a wall system for the causeway widening. Slopes on the order of 0.75H:1V are considered feasible at this site if the design includes hard armor facing elements to resist design water velocities and wave action, as well as design filter elements to prevent erosion/piping of the reinforced soil. Although we do not rule out the possibility that suitable facing elements may be available from a manufacturer and a design could be developed that properly addresses soil slope erosion/piping issues at this site, we consider these concerns as unresolved risks requiring further evaluation.
6. A general comparison of construction prices was made for the wall systems described above based on historical unit price data from 2012 to 2015 provided by MaineDOT and cost data for the 2015 T-Wall system currently under construction for the Schoodic Loop Road Over Frazer Creek project at Acadia National Park provided by VHB. The cost comparisons attempted to include similar items for wall materials, backfill soils, excavation costs, riprap or stone in front of the walls, leveling pad and labor for installation; however, it was not possible to confirm a consistent accounting of these items for all of the projects included in the assessment. The results of the general cost comparison are as follows:

Source	Wall Type	Location	Average Wall Cost/SF of Wall Face
MaineDOT	T-Wall and Doublewal	several (24)	\$67 - \$69
MaineDOT	Redi-Rock (gravity)	several (19)	\$62 - \$69
MaineDOT	Stone Strong (gravity)	several (5)	\$105 - \$112
MaineDOT	GRS-IBS	North Haven Island	\$107
VHB	T-Wall	Schoodic Point	\$67

7. Based on favorable analogous past applications, a higher degree of confidence regarding a resistance to backfill erosion/piping and stability under storm conditions, suitability to the phased construction, and estimated cost, we conclude that the T-Wall system is the preferred alternative. Other QPL wall systems discussed herein as feasible for application to this site should not be considered unacceptable, but if proposed for use certain design issues should be thoroughly assessed during final design and construction.

CLOSING

The geotechnical considerations included in this memorandum are preliminary and provided for the exclusive use of VHB for development of the Preliminary Design Report (PDR) for the replacement of the Clark Bridge in Tremont, Maine. The memorandum was prepared in accordance with generally accepted soil and foundation engineering practices. Our analyses and recommendations are based in part on the subsurface explorations completed. If actual conditions encountered vary from those described or referenced in this memorandum, Golder should be notified so that we may re-evaluate, and if necessary, revise our conclusions or recommendations. Upon completion of the final PDR and selection of a final bridge alignment, profile, span arrangement, and causeway widening retaining wall system, geotechnical evaluations should be conducted to provide final geotechnical design recommendations in a letter report..

Please contact us if you have questions or comments regarding the information summarized herein.



March 9, 2016

Proposal No.: 123-87440 01

Mr. Timothy S. Bryant, P.E.
Vanasse Hangen Brustlin, Inc.
500 Southborough Drive, Suite 105B
South Portland, Maine 04106

**RE: GEOTECHNICAL RECOMMENDATIONS FOR FINAL DESIGN
CLARK BRIDGE REPLACEMENT PROJECT
TREMONT, MAINE
MAINEDOT WIN 19305.00**

Dear Mr. Bryant:

Golder Associates Inc. (Golder) is pleased to submit this letter report to Vanasse Hangen Brustlin, Inc. (VHB) for final geotechnical design recommendations in support of the 85% Design submittal for the Clark Bridge replacement project in Tremont, Maine. Golder completed a preliminary geotechnical investigation for the project¹ in support of VHB's Preliminary Design Report² submittal in 2012 that provided the findings of our subsurface investigation including boring logs, geophysical survey, laboratory test results, and an interpreted subsurface profile, and geotechnical considerations for replacement bridge foundations and causeway widening for use in preliminary design. In addition, Golder submitted a supplemental preliminary geotechnical considerations memo³ in July 2015 that evaluated causeway widening retaining wall alternatives and foundation support requirements. This final design letter report presents recommended geotechnical criteria for final design with regard to foundation bearing resistance, sliding, earth pressure, seismic site class, filter and material requirements, embankment stability and construction considerations and phasing. Subsurface information collected in 2012 and presented in our 2012 report¹ is referenced herein but is not included in this report. Our work was performed in accordance with our revised proposal dated April 2, 2015 and our Agreement dated May 14, 2015.

PROJECT DESCRIPTION

Clark Bridge carries State Route 102 over Marshall Brook in Tremont, Maine, and was constructed in 1924 to replace two single span bridges. The bridge is a 15-foot (ft) clear span cast-in-place concrete slab superstructure supported on dry-laid stone masonry abutments. Two narrow causeways with riprap surfaced side slopes support the west and east bridge approaches⁴, and are about 20 and 100 ft long, respectively. According to the 1924 design drawings, the causeways were constructed of timber cribbing, with field stone used for abutments and retaining walls, and quarried stone rubble masonry for fill materials.

As shown in VHB's 60% Submission Drawings⁵, the design includes a new bridge with the same opening as the existing bridge, and a widened causeway for a sidewalk on both approach roadways. The proposed

¹ Golder Associates Inc. (2012). "Preliminary Geotechnical Investigation, Clark Bridge Replacement, Route 102, Tremont, Maine, MaineDOT WIN 19306.00", December 12, 2012.

² Vanasse Hangen Brustlin, Inc. (2012). "Preliminary Design Report, Clark Bridge #3663, Tremont Road over Marshall Brook, Tremont, ME, WIN 19305.00", October 30, 2012, Revised December 12 & 31, 2012.

³ Golder Associates Inc. (2015). "Supplemental Preliminary Geotechnical Considerations Final PDR Support, Clarke Bridge Replacement Project, Tremont, Maine, MaineDOT WIN 19305.00", July 9, 2015.

⁴ For purposes of this letter the project North is parallel to the alignment of Marshall Brook and Clark Bridge and the approach causeways are referenced as an east-west orientation.

⁵ Vanasse Hangen Brustlin, Inc. (2015). "Tremont Hancock County Clark Bridge Tremont Road over Marshall Brook, Project Length 0.08 mi. Bridge Replacement Bridge No. 3663, Plan Impacts Complete 60% Submission, November 4, 2015.



roadway on the bridge and approaches includes two 11 foot paved lanes with 4 foot shoulders with a 5 foot sidewalk on the widened south side. Current recommendations for the bridge replacement include a 40 ft span concrete slab supported on full height gravity abutments and wingwalls founded on spread footings with underlying cast-in-place subfootings bearing on bedrock. Proposed new abutments are located about 10 feet behind the existing dry-laid field stone abutments. Widening the south side of the causeway must be completed within a limited right-of-way boundary, and is proposed to be completed with a Prefabricated Concrete Modular Gravity Wall system (PCMG, e.g., T-Wall) founded on spread footings and/or subfootings bearing on bedrock. Phased construction is planned to maintain one-lane of alternating two-way traffic throughout construction.

BRIDGE FOUNDATION ALTERNATIVES

Subsurface conditions at the site and described in our preliminary report include the presence of shallow bedrock. Refer to the attached Figures 2 and 3, which are from our preliminary design report and show the exploration locations and interpreted subsurface profile. Due to the presence of shallow bedrock, the most effective foundation type for this site is full height cantilever-type abutments, wingwalls, and retaining walls on cast-in-place spread footings founded directly on bedrock or on cast-in-place subfootings founded directly on bedrock. For the purposes of this report, we assume all of the new abutments and walls will be founded as recommended above and our recommendations and construction considerations apply to cast-in-place spread footings founded directly on bedrock or on cast-in-place subfootings founded directly on bedrock.

BRIDGE ABUTMENT SPREAD FOOTINGS ON BEDROCK

As discussed in our preliminary report and design memo, the bedrock in the area of the west abutment and retaining wall is interpreted to slope upward to the west at an approximate slope of 4H:1V from a low point at about Elev. -5 ft (NAVD88 datum) underlying the existing abutment. The bedrock in the area of the east abutment and retaining wall is interpreted to slope more gradually upward to the east at an approximate slope of 10H:1V from a low point at about Elev. 1 ft underlying the existing abutment.

Bearing Resistance: Based on the rock type, structure, unconfined compressive strength, and interpreted Rock Mass Rating (RMR), we recommend a strength limit state factored bearing resistance of 12 ksf be used for spread footings bearing on sound bedrock assuming the footing is bearing on a level rock surface. In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Sliding Resistance: Assuming the cast-in-place concrete abutment and retaining wall sub footings bear on sound, clean bedrock, a sliding coefficient ($\tan \delta$) of 0.7 is recommended per AASHTO LRFD Table 3.11.5.3-1. A resistance factor of 0.8 is recommended for sliding per MaineDOT Bridge Design Guide Table 5-3.

Settlement: As stated above, we assume that all bridge abutments and retaining walls will be founded on cast-in-place spread footings founded directly on bedrock or on cast-in-place subfootings founded directly on bedrock. As such, any settlement of the bridge abutments or retaining walls will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

Footing Subgrade Preparation: The nature, slope, and degree of fracturing in the bedrock bearing surface will not be evident until the foundation excavations are made. The bedrock surface should be cleared of all loose, fractured, and decomposed bedrock and soil. The final bedrock surface should be approved for cleanliness and condition by the Resident prior to the placement of the footing concrete.

If portions of the sloping rock surface are steeper than 4H:1V, the footing subgrade will require benching to provide a level bearing surface in accordance with MaineDOT's Bridge Design Guide Section 5.3.4.2. Mechanical rock excavation methods are recommended. Steps should be kept to a minimum, have a vertical face, and extend for the full width of the footing. Each step should be a minimum of 2 feet or the

thickness of the footing, whichever is greater. The lower footing should support the upper footing at the step.

FROST PROTECTION

We anticipate that the bridge abutments and retaining walls will be supported on cast-in-place spread footings founded directly on bedrock or on cast-in-place subfootings founded directly on bedrock. As such, heave due to frost is not a design concern and no requirements for minimum depth of embedment are necessary.

SEISMIC DESIGN CONSIDERATIONS

As discussed in our preliminary report, a geophysical survey was performed over the subsurface profile for the Clarke Bridge Replacement. The results of the geophysical seismic survey indicated seismic velocities of the emerged marine sediments overlying bedrock range from 1,600 to 2,000 ft/sec. Bedrock seismic velocities ranged from 12,500 ft/sec to 15,800 ft/sec. Based on the site class definitions presented in AASHTO Table 3.10.3.1.1, the existing subsurface profile (overburden soils and bedrock) would result in a Site Class C. However, since the abutments and retaining walls will be founded directly on bedrock, we recommend that the site class can be increased to Site Class A.

EARTH PRESSURE

Lateral earth pressures will be applied to the backside of the abutment and retaining walls due to the weight of the backfill and surcharge pressures from construction equipment and/or live loads. In addition, since the abutments and retaining walls will be in a tidal environment, additional design and construction measures should be considered. We recommend the earth pressure loads should be calculated based on the following parameters/assumptions.

- Granular Underwater Backfill (MaineDOT Bridge Design Guide, Table 3-3): Unit weight of soil = 125 pcf; angle of internal soil friction $\phi = 32$.
- Active earth pressures will be applied to the T-wall or PCMG retaining walls. For a broken backfill condition, the Coulomb active earth pressure is 0.33 assuming the friction angle δ between the backfill and the assumed vertical back face of the wall is 24 degrees in accordance with the MaineDOT Bridge Design Guide, Table 3-3.
- An at rest earth pressure coefficient of 0.47 should be assumed for the abutments.
- The passive resistance from any fill in front of the walls should be ignored.
- A surcharge load should be considered in accordance with the guidance provided in the Maine Bridge Design Guide Section 3.6.8 and Table 3-4 and AASHTO 3.11.6.4. Based on our review of the 60% design plans, a vehicular surcharge load is likely not required for the retaining wall design since the traffic load will be a greater distance than one-half the wall height away. However, the abutment design should include a surcharge load of at least 288psf (2.3ft*125pcf) for Abutment No. 1 (~17ft) and at least 375psf (3ft*125pcf) for Abutment No. 2 (~10ft). The resulting horizontal pressure from the surcharge should be applied with a uniform distribution over the height of the wall.
- As recommended by the MaineDOT Bridge Design Guide Section 3.6.2, the retaining walls and abutments should be designed for a minimum 3 foot differential in water pressure (above weep holes if present) due to a lag in the tidal influence.
- Based on the guidance provided in the MaineDOT Bridge Design Guide Section 5.6.5.2, for walls located in water, the retaining walls and abutments should be backfilled with an 18-inch thick layer of crushed stone (MDOT 703.31) extending vertically along the inside wall face and separated from the surrounding soils with a Class 1 erosion control geotextile (MDOT 722.0). In addition, due to the potential for overtopping, similar filter criteria (separation geotextile) should be provided for the backfill above the wall.

PCMG WALL

As discussed, VHB's design includes installation of a prefabricated concrete modular gravity (PCMG) wall (i.e., T-Wall) along the southern limit of the approach embankments. Golder understands that the PCMG wall will be designed by the Contractor. The PCMG Wall design should be designed in accordance with MaineDOT Standard Specification 674 for Prefabricated Concrete Modular Gravity Walls and the MaineDOT Bridge Design Guide Chapter 3. Designs should be stamped by a Registered Professional Engineer in the state of Maine and submitted to the Engineer for review. In addition, the PCMG wall design should follow the following design criteria.

- The applied bearing pressure should not exceed 12 ksf for all walls bearing on sound bedrock.
- A minimum 18-inch thick layer of crushed stone (MDOT 703.31) should extend vertically along the inside wall face. The layer of crushed stone shall extend vertically from the bottom course of the wall to the top course of the wall. The crushed stone shall be separated from the surrounding backfill with erosion control geotextile (MaineDOT 722.03). A minimum of 1.5 feet of overlap is required between adjacent lengths of geotextile.
- The Contractor may construct the PCMG using crushed stone (MaineDOT 703.31) as backfill behind and between the the PCMG units. If using crushed stone for backfill, the PCMG may be designed assuming a friction angle of 40 degrees for soil within the units. Crushed stone should be placed and compacted in lifts to achieve a consistent dense course as approved by the Engineer. The crushed stone shall be separated from the surrounding backfill with erosion control geotextile (MaineDOT 722.03).
- Due to the potential for overtopping, similar filter criteria (separation geotextile) should be provided for the backfill above the wall.
- Design the wall for a minimum 3 foot differential in water pressure due to a lag in the tidal influence.
- Based on our review of the 60% design plans, a vehicular surcharge load is likely not required for the retaining wall design since the traffic load will be a greater distance than one-half the wall height away. However, relevance of applied surcharge loading per Maine Bridge Design Guide Section 3.6.8 and Table 3-4 and AASHTO 3.11.6.4., should be re-evaluated during final design.
- The PCMG walls shall consist of Class "LP" concrete and epoxy coated rebar.
- The precast concrete units shall contain calcium nitrate solution or equivalent, as approved by the Engineer, to inhibit corrosion.

EMBANKMENT STABILITY

The current design includes widening the crest of both approach embankments to the north. This will be accomplished by placing a layer of plain riprap over the existing embankment with new slopes ranging between 1.5H:1V and 2H:1V. The riprap should be placed in accordance with the MaineDOT Standard Detail 610(02) – Stone Scour Protection, including the installation of Class 1 non-woven erosion control geotextile (standard specification 722.03) and a minimum 12 inch thick protective aggregate cushion meeting the requirements of 703.29 granular borrow – material for underwater backfill.

It is assumed that the riprap shown along the southern edge of the approach embankments will be placed directly in front of the proposed retaining wall or directly on top of bedrock. In the case that riprap will be placed over soil, the MaineDOT Standard Detail for Stone Scour should be followed including a geotextile and protective cushion. The plain riprap proposed along the southern limit will be placed at a slope of 2H:1V.

All riprap should be founded on a level bearing surface (presumably bedrock) and placed with the appropriate erosion control measure detailed above. Based on an assumed internal friction angle of 41°

for riprap and a maximum slope of 1.5H:1V, the proposed riprap embankment widening will have an infinite slope stability factor of safety of 1.3.

CONSTRUCTION CONSIDERATIONS AND PHASING

Construction of the Clark Bridge replacement will require a phased approach that maintains one-lane of alternating two-way traffic throughout construction. Construction activities will likely include construction of cofferdams and earth support systems to support the existing abutment foundations and approach fills during construction and to control water flow during construction of subfootings and footings for the abutments and walls. According to the 1924 design drawings, the causeways were constructed of timber cribbing, with field stone used for abutments and retaining walls, and quarried stone rubble masonry for fill materials. These materials may present difficulties for cofferdam and excavation support system installations.

The base of the existing west abutment footing is unknown; however, it is possible that the footing may be supported on the layer of very dense emerged marine sediments encountered at the test borings at about El. 0 ft. The design requires that the existing abutments remain in place. The contractor should take measures to avoid the risk of loss of soil support and undermining the existing abutment foundation during construction.

The contractor should maintain the abutment and retaining wall excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the engineer prior to placing concrete. Where foundations are constructed in the dry, the final bearing surface should be washed with high pressure water and air prior to concrete being placed for the footing.

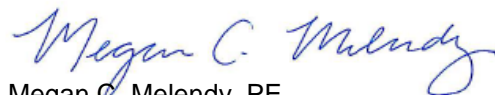
CLOSING

The geotechnical considerations included in this letter report are provided for the exclusive use of VHB for development of the 85% Design Report for the replacement of the Clark Bridge in Tremont, Maine. Once the 85% Design Drawings are completed, please provide a copy to Golder so that we can review the design and provide comment as to the compliance with our design recommendations prior to development of the 100% drawings. This letter report was prepared in accordance with generally accepted soil and foundation engineering practices. Design calculations supporting the recommendations provided in this letter report are included as attachments. Our analyses and recommendations are based in part on the subsurface explorations completed. If actual conditions encountered vary from those described or referenced in this letter report, Golder should be notified so that we may re-evaluate, and if necessary, revise our conclusions or recommendations.

Please contact us if you have questions regarding the information summarized herein.

Sincerely,

GOLDER ASSOCIATES INC.



Megan C. Melendy, PE
Senior Engineer

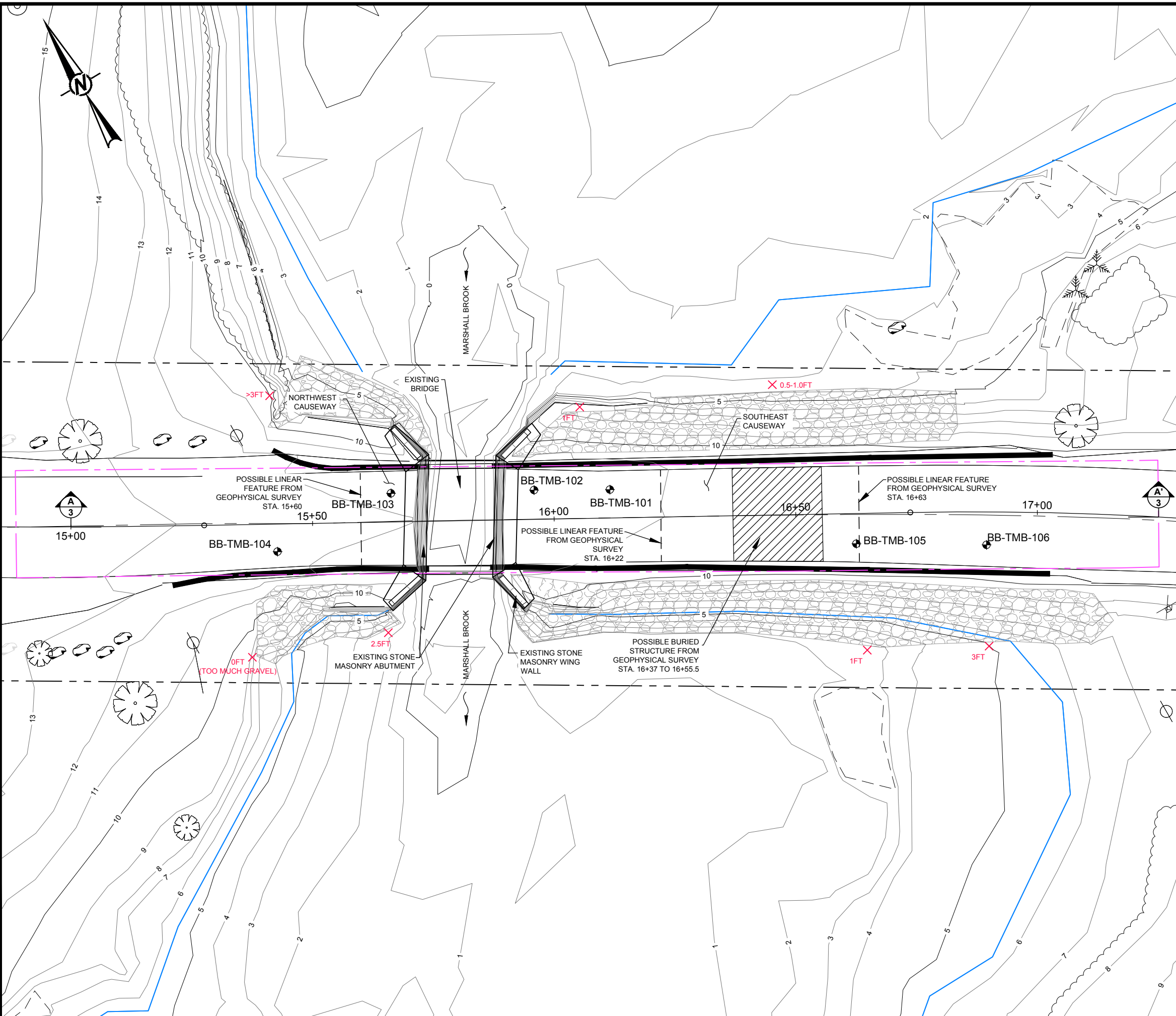


Mark S. Peterson, PE
Principal

Attachments Figure 2 – Subsurface Exploration Locations
 Figure 3 – Interpreted Subsurface Profile A-A
 Supporting Design Calculations

MCM/drj

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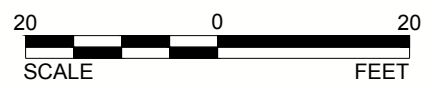


LEGEND

- 5 FT INDEX CONTOUR (FT MSL)
- 1 FT INDEX CONTOUR (FT MSL)
- EDGE OF DRAINAGE WATERWAY
- GUARDRAIL BEAM
- LEDGE OUTCROP
- RIGHT OF WAY
- STATIONING ALONG CENTERLINE
- GEOPHYSICAL SURVEY LOCATION
- TREE LINE
- BOULDER
- DECIDUOUS TREES
- BORING LOCATION AND DESIGNATION
- RIP RAP ARMOR
- CROSS SECTION LOCATION AND DIRECTION
- ROCK PROBE LOCATION (APPROXIMATE) AND REFUSAL DEPTH

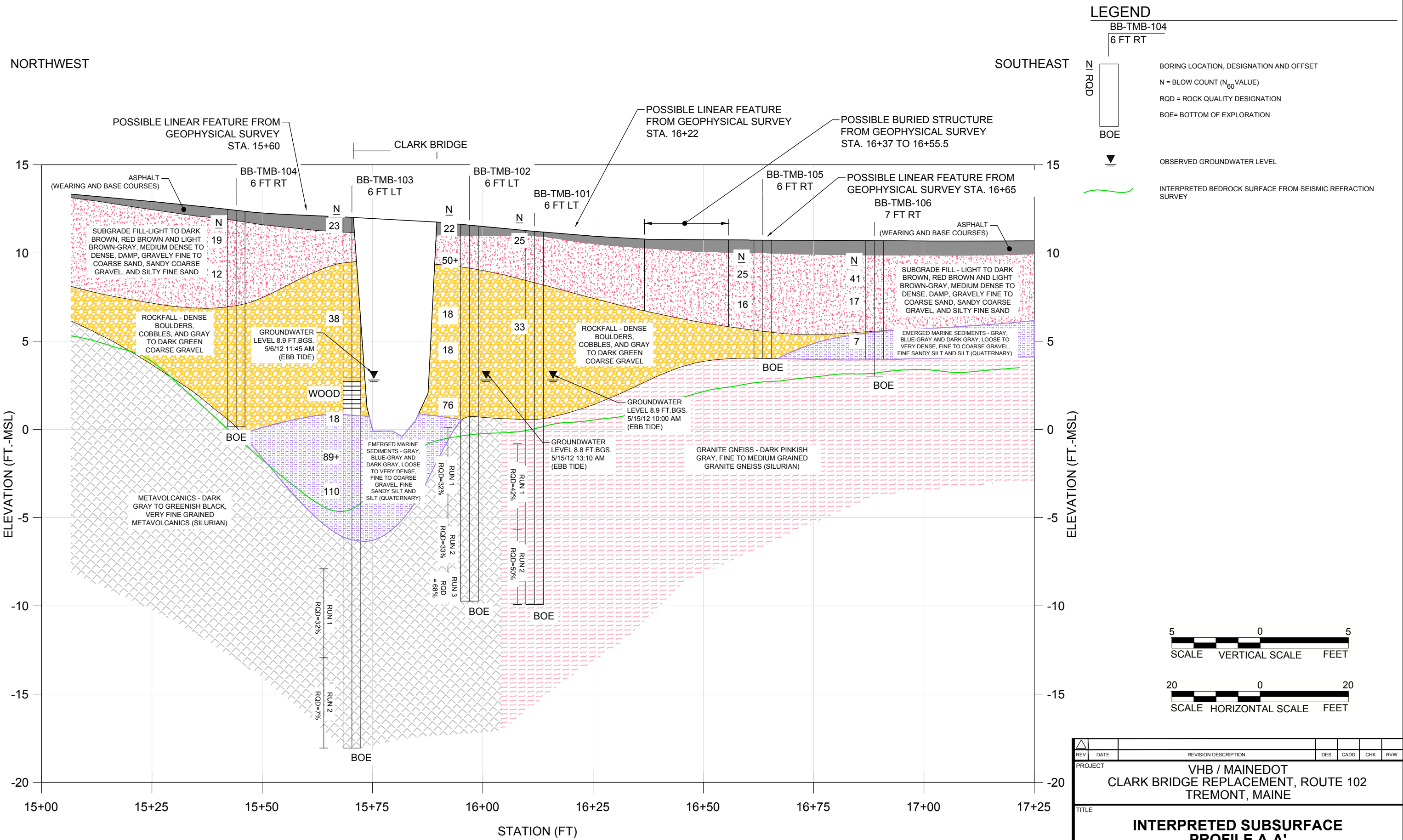
- ### REFERENCES & NOTES
- 1.) BASEMAP ELEMENTS FROM VHB, INC. DRAWING TITLED 3DPTOP_13MAR12.dwg. DATED MAY 10, 2012.
 - 2.) BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY MAINE TEST BORINGS, INC.
 - 3.) BASIS OF BEARING IS MAINE GRID NORTH. HORIZONTAL DATUM IS MAINE 2000 EAST ZONE NAD83 (1996).
 - 4.) ELEVATIONS ARE IN FEET AND ARE REFERENCED TO NAVD88 (GEOID03).
 - 5.) BRIDGE ABUTMENT AND WINGWALL EXTENTS FROM 1924 BRIDGE DRAWINGS (SEE APPENDIX A).

BORING ID	NORTHING (FT)	EASTING (FT)	ELEVATION (FT-MSL)
BB-TMB-101	153296.02	2172345.66	11.19
BB-TMB-102	153303.70	2172331.97	11.42
BB-TMB-103	153317.75	2172305.90	11.89
BB-TMB-104	153318.89	2172279.54	12.27
BB-TMB-105	153261.04	2172384.42	10.45
BB-TMB-106	153247.51	2172407.71	10.46



REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW
PROJECT						
VHB / MAINE DOT CLARK BRIDGE REPLACEMENT, ROUTE 102 TREMONT, MAINE						
TITLE						
SUBSURFACE EXPLORATION LOCATIONS						
PROJECT No.		123-87440	FILE No.		12387440A002	
DESIGN	JRS	12/07/12	SCALE		AS SHOWN	
CADD	MPB	12/07/12				
CHECK	JRS	12/07/12				
REVIEW	MSP	12/07/12				
FIGURE 2						

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LEGEND

BB-TMB-104
6 FT RT

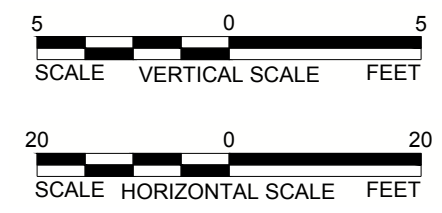
N
ROD
BOE

BOE

▼ OBSERVED GROUNDWATER LEVEL

INTERPRETED BEDROCK SURFACE FROM SEISMIC REFRACTION SURVEY

BORING LOCATION, DESIGNATION AND OFFSET
 N = BLOW COUNT (N₆₀ VALUE)
 RQD = ROCK QUALITY DESIGNATION
 BOE = BOTTOM OF EXPLORATION



REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVV
PROJECT						
VHB / MAINEDOT CLARK BRIDGE REPLACEMENT, ROUTE 102 TREMONT, MAINE						
TITLE						
INTERPRETED SUBSURFACE PROFILE A-A'						
PROJECT No.		123-87440	FILE No.		12387440A003	
DESIGN	JRS	12/07/12	SCALE		AS SHOWN	
CADD	MPB	12/07/12				
CHECK	JRS	12/07/12				
REVIEW	MSP	12/07/12				



FIGURE 3

OBJECTIVE: EVALUATE SEISMIC SITE CLASS

CALCULATIONS:

FOLLOW PROCEDURE OUTLINED IN AASHTO Table C3.10.3.1-1

FOR EXISTING SUBSURFACE PROFILE: USING BLOW COUNTS

$$D = 0-15' \quad \bar{N} = \frac{19+12+23+38+22+15+18+25+33+25+16+41+17+7}{14} = 22$$

(overburden)

$$D = 15-20' \quad \bar{N} = \frac{89+76+100}{3} = 88$$

(H1)

$$D = 20-100' \quad \bar{N} = 100$$

(rock)

$$\bar{N} = \frac{(15 + 5 + 80)}{\left(\frac{15}{22} + \frac{5}{88} + \frac{80}{100}\right)} = 65$$

∴ $\bar{N} > 50$ blows/ft = SITE CLASS C

TABLE: 3.10.3.1-1

EXISTING WITH GEOPHYSICS DATA:

$V_s = 1600 - 2000$ ft/sec (overlying sediments)

$V_s = 12,500 - 15,800$ ft/sec (bedrock)

$\therefore D = 0-20' = 1,600 \text{ ft/sec}$
(overburden)

$D = 20-100 \rightarrow V_s = 12,500 \text{ ft/sec}$
(Rock)

$$\bar{V}_s = \frac{(20 + 80)}{\left(\frac{20}{1600} + \frac{80}{12,500}\right)} \Rightarrow 5,291 \text{ ft/sec}$$

\therefore site class B. (Table 3.10.3-1-1)

HOWEVER :

PROPOSED ABUTMENTS & RETAINING WALLS WILL BE FOUNDED DIRECTLY ON BEDROCK. NO OVER BURDEN

THEREFORE SITE CLASS CAN BE INCREASED TO REFLECT V_s FOR BEDROCK ONLY

$\therefore V_{s \text{ BEDROCK}} = 12,500 \text{ ft/sec}$

SITE CLASS A (Table 3.10.3.1-1)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5,000$ ft/s
B	Rock with $2,500$ ft/sec $< \bar{v}_s < 5,000$ ft/s
C	Very dense soil and soil rock with $1,200$ ft/sec $< \bar{v}_s < 2,500$ ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s $< \bar{v}_s < 1,200$ ft/s, or with either $15 < \bar{N} < 50$ blows/ft, or $1.0 < \bar{s}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s < 600$ ft/s or with either $\bar{N} < 15$ blows/ft or $\bar{s}_u < 1.0$ ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u < 0.5$ ksf
F	Soils requiring site-specific evaluations, such as: <ul style="list-style-type: none"> • Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

- \bar{v}_s = average shear wave velocity for the upper 100 ft of the soil profile
- \bar{N} = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
- \bar{s}_u = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
- PI = plasticity index (ASTM D4318)
- w = moisture content (ASTM D2216)

OBJECTIVE: ESTIMATE BEARING RESISTANCE ON ROCK.

CALCULATIONS:

BEARING RESISTANCE (Carter & Kulhawy, 1988
from NCHRP Report 651
eqn. 8.26)

$$q_{ult} = q_u (\sqrt{s} + \sqrt{(m\sqrt{s} + s)})$$

q_u = uniaxial compressive strength of rock
 m, s = strength parameters for rock

q_u → FROM LAB DATA:

* two types of rock exist:

METAVOLCANIC: $q_u \Rightarrow 11,900 - 18,800$ psi

GRANITE GNEISS: $q_u \Rightarrow 26,800 - 37,000$ psi

* CONSERVATIVELY USE $q_u = 10,000$ psi FOR DESIGN

* NOTE: MOST OF FOOTINGS WILL BEAR ON METAVOLCANIC *

**Golder
Associates**

SUBJECT Tremont - Clark Bridge ^{Final Design}		
Job No. 123-87440	Made by mcm	Date 2/11/10
Ref. -BEARING-	Checked SMP	Sheet 2 of 3
	Reviewed PSH/msp	

RMR : From Rock Core & Outcrop

GRANITE GNEISS : RMR → 46-49 Avg = 47.5 (core)
RMR → 50 (outcrop)

METAVOLCANIC : RMR → 13-31 Avg = 23.6 (core)
RMR → 26 (outcrop)

* USE RMR = 26 FOR DESIGN

NOTE : MOST OF FOUNDATION WILL BEAR ON METAVOLCANIC

m, s : From HOEK, BROWN - 1988

DISTURBED ROCK MASS : * assume mechanical methods of rock removal but conservatively assume disturbed rock mass

$$\frac{m}{m_i} = \exp\left(\frac{RMR - 100}{14}\right) \quad S = \exp\left(\frac{RMR - 100}{6}\right)$$

$m_i = m$ for INTACT ROCK

m, s = rock mass constants

**Golder
Associates**

SUBJECT Tremont - Clark Bridge - Final Design		
Job No.	Made by mem	Date 2/11/16
Ref. BEARING	Checked SMP	Sheet 3 of 3
	Reviewed P&E for WSP	

$$m_i = \underline{25}$$

H-B TABLE 1 - ROCK TYPE E

$$\therefore m = \exp\left(\frac{26-100}{14}\right) 25 \Rightarrow \underline{m = 0.126}$$

$$\therefore s = \exp\left(\frac{26-100}{6}\right) \Rightarrow \underline{s = 0.0000044}$$

4.4×10^{-6}

$$q_{ult} = 19000 \text{ psi} \left(\sqrt{4.4 \times 10^{-6}} + (0.126 \sqrt{4.4 \times 10^{-6}} + 4.4 \times 10^{-6})^{0.5} \right)$$

$$= 184 \frac{\text{lb}}{\text{in}^2} \cdot \frac{144 \text{ in}^2}{\text{ft}^2} \cdot \frac{\text{kip}}{1000 \text{ lb}}$$

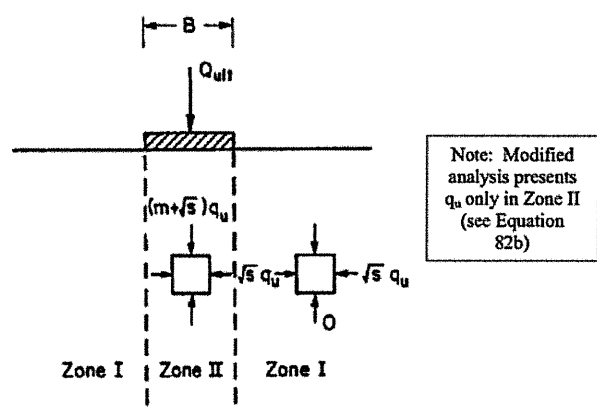
$$q_{ult} = 26.6 \text{ Ksf}$$

FACTORED BEARING RESISTANCE :

$$\phi_b = 0.45 \quad (\text{Table 5-2, Maine DOT Bridge Design})$$

$$q_n = q_u \phi_b = 26.6 \text{ Ksf} (0.45)$$

$$= \underline{12 \text{ Ksf}}$$



Rock Mass Failure Criterion: $\sigma_1 = \sigma_3 + \sqrt{(mq_u \sigma_3 + sq_u^2)}$

Figure 39. Lower bound solution for bearing capacity (Carter and Kulhawy, 1988).

across the interface must be maintained and therefore the bearing capacity of the strip footing may be evaluated from Equation 81 (with $\sigma_3 = s^{0.5}q_u$) as

$$q_{ult} = (m + \sqrt{s})q_u \tag{82a}$$

In an errata to Carter and Kulhawy (1988), Equation (82a) was modified to the following:

$$q_{ult} = (\sqrt{s} + (m\sqrt{s} + s)^{0.5})q_u \tag{82b}$$

A similar approach to the bearing capacity analysis of a strip footing was proposed by Carter and Kulhawy (1988) to be used for a circular foundation with an interface between the two zones that was a cylindrical surface of the same diameter as the foundation. In this axisymmetric case, the radial stress transmitted across the cylindrical surface at the point of collapse of the foundation may be greater than $q_u\sqrt{s}$, without necessarily violating either radial equilibrium or the failure criterion. However, because of the uncertainty of this value, the radial stress at the interface is also assumed to be $q_u\sqrt{s}$ for the case of a circular foundation. Therefore, the predicted (lower bound) bearing capacity is given by Equations 82a and 82b. The m and s constants are determined by the rock type and the conditions of the rock mass, and selecting an appropriate category is easier if either the Rock Mass Rating (RMR) system or the Geological Strength Index (GSI) classification data are available as outlined below. Both bearing capacity formulations expressed in Equations 82a and 82b were investigated in this study.

1.8 Rock Classification and Properties

1.8.1 Overview

A rock mass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, and faults. Table 8 provides a summary of rock mass discontinuity definitions and characteristics. These naturally formed discontinuities create weakness surfaces within the rock mass, thereby reducing the material strength. As previously discussed, the influence of the discontinuities upon the material strength depends upon the scale of the foundation relative to the position and frequency of the discontinuities (Canadian Foundation Geotechnical Society, 2006).

This section provides a short review of rock mass classification/characterization systems and rock properties that are relevant to the methods selected for bearing capacity evaluation. Methods allowing engineering classification of rock mass are reviewed including the Rock Mass index (RMI) system, RMR system and the Hoek-Brown GSI.

1.8.2 Engineering Rock Mass Classification

1.8.2.1 Classification Methods

A number of classification systems have been developed to provide the basis for engineering characterization of rock masses. A comprehensive overview of this subject is provided by Hoek et al. (1995). Most of the classification systems incorporating various parameters were derived from civil engineering case histories in which all components of the engineering geological parameters of the rock mass were considered (Wickham et al., 1972; Bieniawski, 1973, 1979, 1989; Barton et al., 1974). More recently, the systems have been modified to account for the conditions affecting rock mass stability in underground mining. While no single classification system has been developed for or applied to foundation design, the type of information collected for the two more common civil engineering classification schemes—the Q system (Barton et al., 1974), used in tunnel design, and RMR (Bieniawski, 1989), used in tunnel and foundation design—are often considered. These techniques have been applied to empirical design situations, where previous experience greatly affects the design of the excavation in the rock mass. Table 9 outlines the many classification systems and their uses. Detailed descriptions of the different systems and the engineering properties associated with them are beyond the scope of this work and are restricted to the methods relevant to the current research.

The two most commonly used rock mass classification systems today are RMR, developed by Bieniawski (1973) and

order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s .

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s . Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{14}\right) \quad (18)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right) \quad (19)$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{28}\right) \quad (20)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{9}\right) \quad (21)$$

where

m and s are the rock mass constants and m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\text{RMR} = 9 \text{Log}_e Q + 44 \quad (22)$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass <i>m</i> and <i>s</i> values		undisturbed rock mass <i>m</i> and <i>s</i> values				
		A	B	C	D	E
EMPIRICAL FAILURE CRITERION $\sigma'_1 = \sigma'_3 + \sqrt{m\sigma'_c\sigma'_3 + s\sigma_c^2}$ σ'_1 = major principal effective stress σ'_3 = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and <i>m</i> and <i>s</i> are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE <i>dolomite, limestone and marble</i>	LITHIFIED ARGILLACEOUS ROCKS <i>mudstone, siltstone, shale and slate (normal to cleavage)</i>	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE <i>sandstone and quartzite</i>	FINE GRAINED POLYMINERALIC IGNEOUS CRYSTALLINE ROCKS <i>andesite, dolerite, diabase and rhyolite</i>	COARSE GRAINED POLYMINERALIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS - <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>
INTACT ROCK SAMPLES <i>Laboratory size specimens free from discontinuities</i> CSIR rating: RMR = 100 NGI rating: Q = 500		<i>m</i> 7.00 <i>s</i> 1.00 <i>m</i> 7.00 <i>s</i> 1.00	<i>m</i> 10.00 <i>s</i> 1.00 <i>m</i> 10.00 <i>s</i> 1.00	<i>m</i> 15.00 <i>s</i> 1.00 <i>m</i> 15.00 <i>s</i> 1.00	<i>m</i> 17.00 <i>s</i> 1.00 <i>m</i> 17.00 <i>s</i> 1.00	<i>m</i> 25.00 <i>s</i> 1.00 <i>m</i> 25.00 <i>s</i> 1.00
VERY GOOD QUALITY ROCK MASS <i>Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.</i> CSIR rating: RMR = 85 NGI rating: Q = 100		<i>m</i> 2.40 <i>s</i> 0.082 <i>m</i> 4.10 <i>s</i> 0.189	<i>m</i> 3.43 <i>s</i> 0.082 <i>m</i> 5.85 <i>s</i> 0.189	<i>m</i> 5.14 <i>s</i> 0.082 <i>m</i> 8.78 <i>s</i> 0.189	<i>m</i> 5.82 <i>s</i> 0.082 <i>m</i> 9.95 <i>s</i> 0.189	<i>m</i> 8.56 <i>s</i> 0.082 <i>m</i> 14.63 <i>s</i> 0.189
GOOD QUALITY ROCK MASS <i>Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.</i> CSIR rating: RMR = 65 NGI rating: Q = 10		<i>m</i> 0.575 <i>s</i> 0.00293 <i>m</i> 2.006 <i>s</i> 0.0205	<i>m</i> 0.821 <i>s</i> 0.00293 <i>m</i> 2.865 <i>s</i> 0.0205	<i>m</i> 1.231 <i>s</i> 0.00293 <i>m</i> 4.298 <i>s</i> 0.0205	<i>m</i> 1.395 <i>s</i> 0.00293 <i>m</i> 4.871 <i>s</i> 0.0205	<i>m</i> 2.052 <i>s</i> 0.00293 <i>m</i> 7.163 <i>s</i> 0.0205
FAIR QUALITY ROCK MASS <i>Several sets of moderately weathered joints spaced at 0.3 to 1m.</i> CSIR rating: RMR = 44 NGI rating: Q = 1		<i>m</i> 0.128 <i>s</i> 0.00009 <i>m</i> 0.947 <i>s</i> 0.00198	<i>m</i> 0.183 <i>s</i> 0.00009 <i>m</i> 1.353 <i>s</i> 0.00198	<i>m</i> 0.275 <i>s</i> 0.00009 <i>m</i> 2.030 <i>s</i> 0.00198	<i>m</i> 0.311 <i>s</i> 0.00009 <i>m</i> 2.301 <i>s</i> 0.00198	<i>m</i> 0.458 <i>s</i> 0.00009 <i>m</i> 3.383 <i>s</i> 0.00198
POOR QUALITY ROCK MASS <i>Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock</i> CSIR rating: RMR = 23 NGI rating: Q = 0.1		<i>m</i> 0.029 <i>s</i> 0.000003 <i>m</i> 0.447 <i>s</i> 0.00019	<i>m</i> 0.041 <i>s</i> 0.000003 <i>m</i> 0.639 <i>s</i> 0.00019	<i>m</i> 0.061 <i>s</i> 0.000003 <i>m</i> 0.959 <i>s</i> 0.00019	<i>m</i> 0.069 <i>s</i> 0.000003 <i>m</i> 1.087 <i>s</i> 0.00019	RMR = 23 <i>m</i> 0.102 <i>s</i> 0.000003 <i>m</i> 1.598 <i>s</i> 0.00019
VERY POOR QUALITY ROCK MASS <i>Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.</i> CSIR rating: RMR = 3 NGI rating: Q = 0.01		<i>m</i> 0.007 <i>s</i> 0.0000001 <i>m</i> 0.219 <i>s</i> 0.00002	<i>m</i> 0.010 <i>s</i> 0.0000001 <i>m</i> 0.313 <i>s</i> 0.00002	<i>m</i> 0.015 <i>s</i> 0.0000001 <i>m</i> 0.469 <i>s</i> 0.00002	<i>m</i> 0.017 <i>s</i> 0.0000001 <i>m</i> 0.532 <i>s</i> 0.00002	<i>m</i> 0.025 <i>s</i> 0.0000001 <i>m</i> 0.782 <i>s</i> 0.00002

m

E
 DISTURBED
 RMR = 26
m = 0.126
s = 0.000044

Table 5-2 Bearing Resistance Factors

Method/Soil/Condition	Bearing Resistance Factor, ϕ_B
Theoretical method (Munfakh et al. 2001) in clay	0.50
Theoretical method (Munfakh et al. 2001) in sand using SPT.	0.45
Semi-empirical methods (Meyerhof, 1957, Terzaghi, Vesic) all soils	0.45
Footings on rock	0.45
Plate Load Test	0.50



5.3.6 Settlement

The design of spread footings is frequently controlled by settlement at the service limit state. It is advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

Total and differential settlement should be evaluated. The total settlement includes elastic settlement, primary consolidation, and secondary compression. Elastic settlement results from the compression of the material supporting the foundation or from reduction in pore space in nonsaturated soils. Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. Some soils, after experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression, or "creep".

Immediate or elastic settlement should be determined using the Service I Load Combination, specified as unfactored dead load, plus the unfactored component of live loads assumed to extend to the footing level. Time-dependent settlements, i.e., primary consolidation and secondary compression settlement may be determined using the unfactored dead load only. Other factors that can affect settlement, such as embankment loading, lateral and/or eccentric loading, and dynamic or earthquake loads should also be considered, where applicable.

Differential settlement occurs when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Transportation structures, especially bridges, are not exceptionally tolerant of differential settlements. Deformation limitations will form the upper bound of allowable differential settlements used to design shallow foundations.

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

10.5.5.2.2—Spread Footings

C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

		Method/Soil/Condition	Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	ϕ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

C_{wq}, C_{wy} = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

D_f = footing embedment depth taken to the bottom of the footing (ft)

The nominal bearing resistance, in ksf, for footings on cohesionless soils based on *CPT* results may be taken as:

$$q_n = \frac{\bar{q}_c B}{40} \left(C_{wq} \frac{D_f}{B} + C_{wy} \right) \quad (10.6.3.1.3-2)$$

where:

\bar{q}_c = average cone tip resistance within a depth range B below the bottom of the footing (ksf)

B = footing width (ft)

C_{wq}, C_{wy} = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

D_f = footing embedment depth taken to the bottom of the footing (ft)

10.6.3.1.4—Plate Load Tests

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. Where plate load tests are conducted, they should be conducted in accordance with AASHTO T 235 and ASTM D1194.

The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

C10.6.3.1.4

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils.

Scale effects should be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification, i.e., layer thicknesses, depths, and properties. Plate load test results should be applied only within a sub-area of the project site for which the subsurface conditions, i.e., stratification, geologic history, and properties, are relatively uniform.

10.6.3.2—Bearing Resistance of Rock

10.6.3.2.1—General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and *RQD* may be applicable. For footings on less competent rock, more detailed investigations and

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

10.6.3.2.2—Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geomechanics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures.

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

10.6.3.2.3—Analytic Method

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered.

10.6.3.2.4—Load Test

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, B or L , for footings on soils, or 0.45 of the corresponding footing dimensions B or L , for footings on rock.

The design procedures for foundations in rock have been developed using the RMR rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

C10.6.3.2.3

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in Kulhawy and Goodman (1987), Goodman (1989), and Sowers (1979).

C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.

**Golder
Associates**

SUBJECT Tremont - Clark Bridge - ^{Final} Design		
Job No. 123-87440	Made by MCM	Date 2/11/16
Ref. -SLIDING-	Checked SMP	Sheet 1 of 2
	Reviewed PER for MSP	

OBJECTIVE: ESTIMATE COEFFICIENT OF FRICTION FOR PROPOSED SPREAD FOOTING ON ROCK.

PROVIDE RECOMMENDED RESISTANCE FACTORS FOR SLIDING.

REFERENCE:

1. AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, SEVENTH EDITION, 2014.
2. Maine DOT BRIDGE DESIGN GUIDE, AUGUST 2003 WITH 2014 UPDATES.

CALCULATIONS:

PROPOSED DESIGN INCLUDES ABUTMENT AND RETAINING WALLS WILL BE FOUNDED ON BEDROCK. VHB DESIGN INCLUDES CAST-IN-PLACE SUBFOOTING BEARING ON BEDROCK.

COEFFICIENT OF FRICTION: $\tan \delta = 0.7$ (AASHTO, Table 3.11.5.3-1)

**Golder
Associates**

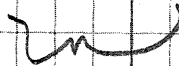
SUBJECT Tremont-Clark Bridge - <i>Final Design</i>		
Job No. 123-87440	Made by MCM	Date 2/11/16
Ref. -SLIDING-	Checked SMP	Sheet 2 of 2
	Reviewed PGP for MGP	

STRENGTH LIMIT STATE RESISTANCE FACTORS:

$$\phi_s = 0.8 \quad (\text{Table 5-3, Maine DOT Bridge Design})$$

∴ FACTORED RESISTANCE

$$R_r = \phi_s R_f + \phi_{ep} R_{ep}$$



∴ PASSIVE RESISTANCE

* Recommend ignore passive resistance due to scour.

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, $\tan \delta$ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

*

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

C3.11.5.4

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f .

For cohesive soils, passive pressures may be estimated by:

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

5.3.7 Overall Stability

The overall global stability of spread footings on or near an earth slope should be investigated using Service I Load Combination and an appropriate resistance factor. Where a slope supports or contains a structural element, such as a spread footing supporting a wall or abutment, the resistance factor, ϕ , shall be taken as of 0.65

For foundations on spread footings constructed along rivers and streams, scour of foundation materials is evaluated as specified in LRFD 2.6.4.4.2. Extreme limit state design should check that the nominal resistance of the footing and slope remaining after the scour due to the check flood for scour can support the unfactored strength limit state loads with a resistance factor, ϕ , of 1.0

The overall stability of retaining wall spread footings on or near a slope should be evaluated using limiting equilibrium methods of analysis, which employ the Modified Bishop, simplified Janbu, Spencer, or other generally accepted methods of slope stability analysis.

5.3.8 Sliding

Failure by sliding should be investigated for all spread footings bearing on soil or bedrock. Passive earth pressure exerted by fill in front of the footing should be neglected in consideration that the soil may be removed as the result of scour or during future construction, and in consideration that soils in front of the footing will be subject to freeze-thaw weakening over time. If passive pressure is included as part of shear resistance to sliding, consideration should be made to possible removal of the soil in front of the foundation in the future. If passive resistance is included in the resistance, its magnitude is commonly taken as 50% of the maximum passive pressure resistance computed using Rankine Passive resistance. This is the basis of a resistance factor for passive resistance of ϕ_{ep} of 0.50.

The factored resistance against failure by sliding is taken as:

$$R_r = \phi R_n = \phi_s R_f + \phi_{ep} R_{ep}$$

where:

R_n = nominal sliding resistance

ϕ_s = resistance factor for shear resistance between soil and foundation specified in Table 5-3.

R_f = nominal sliding resistance between soil and foundation

ϕ_{ep} = resistance factor for passive resistance = 0.50

R_{ep} = nominal passive resistance of the soil available throughout the design life of the structure.

Table 5-3 Resistance Factors for Sliding of Spread Footings at the Strength Limit State

Soil/Condition	Sliding Resistance Factor, ϕ_s
Precast concrete on sand	0.90
Cast-in-place concrete on sand	0.80
Cast-in-place or precast concrete on clay	0.85
Soil on soil	0.90
Cast-in-place concrete on rock (based on reliability theory analysis of footings on sand)	0.80
Cast-in-place concrete on rock (calibrated to ASD Factor of Safety of 1.5)	0.90

Spread footings should be designed such that the factored resistance to sliding, R_f , is greater than the factored force effects due to the horizontal components of loads. Load factors selected should produce the extreme force effect. The live load surcharge is not included over the heel. Specific guidance for selection of load factors for sliding are provided in LRFD Figure C11.5.6-2.

The nominal sliding resistance between footings and cohesionless soils is taken as:

$$R_f = V \times \tan \delta$$

where:

$$\begin{aligned} \tan \delta &= \tan \phi \text{ for cast-in-place footings on soil} \\ \tan \delta &= 0.80 \tan \phi \text{ for precast footings on soil} \\ V &= \text{total vertical force} \end{aligned}$$

The coefficient of friction, $\tan \phi$, for sliding should be as shown in Table 3-3 for the soil type under the footing and LRFD Table 3.11.5.3-1.

The nominal sliding resistance between footings and silt and/or clay soils should be taken to be the lesser of: (1) the undrained shear strength of the silt/clay, or, (2) one-half of the normal stress on soil when the footing is founded on at least 6 inches of compacted granular fill on silt/clay.

For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding. If smooth bedrock is present at the bearing elevation or if the coefficient of sliding is insufficient to resist lateral forces, the bedrock should be doweled to improve stability. When a footing is doweled into rock, the dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the Designer. The spacing of

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

10.5.5.2.2—Spread Footings

C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

		Method/Soil/Condition	Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	ϕ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

**Golder
Associates**

SUBJECT Tremont - Clark Bridge - Final Design
Job No. 23-87440 Made by MCM Date 2/24/16
Ref. STABILITY Checked SMP Sheet 1 of 1
Reviewed PCG for M&S

OBJECTIVE: EVALUATE STABILITY OF PROPOSED RIPRAP EMBANKMENTS

CALCULATIONS: EMBANKMENTS ALONG NORTH SIDE OF CAUSEWAY PLANNED TO EXTEND OVER EXISTING EMBANKMENT
RIPRAP WILL BE PLACED IN FRONT OF RETAINING WALL ON SOUTH SIDE OF CAUSEWAY.

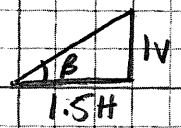
RIPRAP SLOPES RANGE BETWEEN 2H:1V AND 1.5H:1V.

EVALUATE INFINITE SLOPE STABILITY FOR WORST CASE (1.5H:1V). LOOKING FOR FS ≥ 1.3.

$$FS = \frac{\tan \phi}{\tan \beta}$$

ϕ = Internal Friction \neq for Riprap

β = slope \neq



$$\tan \beta = 1/1.5$$
$$\beta = 33.69^\circ$$

ASSUME $\phi_{Riprap} = 41^\circ$

$$FS = \frac{\tan 41^\circ}{\tan 33.69^\circ} = 1.30 \geq 1.3 \checkmark$$

∴ RIPRAP SLOPES ARE STABLE ✖



SUBJECT: Tremont Clark Bridge Replacement - Earth Pressures
Project Number: 123-87440
Project Name: Tremont Clark Bridge Replacement
Prepared by: MCM
Date: 2/17/2016

Checked by: SMP
Reviewed by: PCF for MSP

Objective:

Calculate the earth pressure coefficients.

Method:

Methods described in the MaineDOT Bridge Design Guide, 2003 with 2014 updates; AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014 and NavFac DM 7.02.

References:

1. MaineDOT Bridge Design Guide, 2003 with 2014 updates.
2. AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014.
3. VHB Tremont Hancock County Clark Bridge Tremont Road over Marshall Brook - 60% Submission drawings, November 4, 2015.
4. Golder figure titled "Interpreted Subsurface Profile A-A" included in Preliminary Geotechnical Investigation Report for Clark Bridge Replacement, December 2012.

Calculations:

A.) Determine the at-rest earth pressure coefficient (for use at ABUTMENT WALLS)

Per AASHTO 3.11.5.2, for normally consolidated soils:

$$K_0 = 1 - \sin \phi'_f$$

where:

K_0 at-rest earth pressure coefficient

ϕ'_f angle of internal soil friction, deg

(From MEDOT Bridge Design Guide Table 3-3, Granular underwater backfill)

32

$K_0 =$	0.470
---------	-------

B.) Determine the active earth pressure coefficient using Coulomb theory (FOR T-WALL or PCMG RETAINING WALLS)

Coulomb theory should be used for the design of gravity, semigravity, and prefabricated modular walls, in addition to cantilever walls and abutments with short heels. Interface friction developing along the back face of the wall is considered with Coulomb.

$$K_a = \frac{\sin(\alpha + \phi)^2}{\sin(\alpha)^2 \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}} \right)^2}$$

where:

K_a active earth pressure coefficient

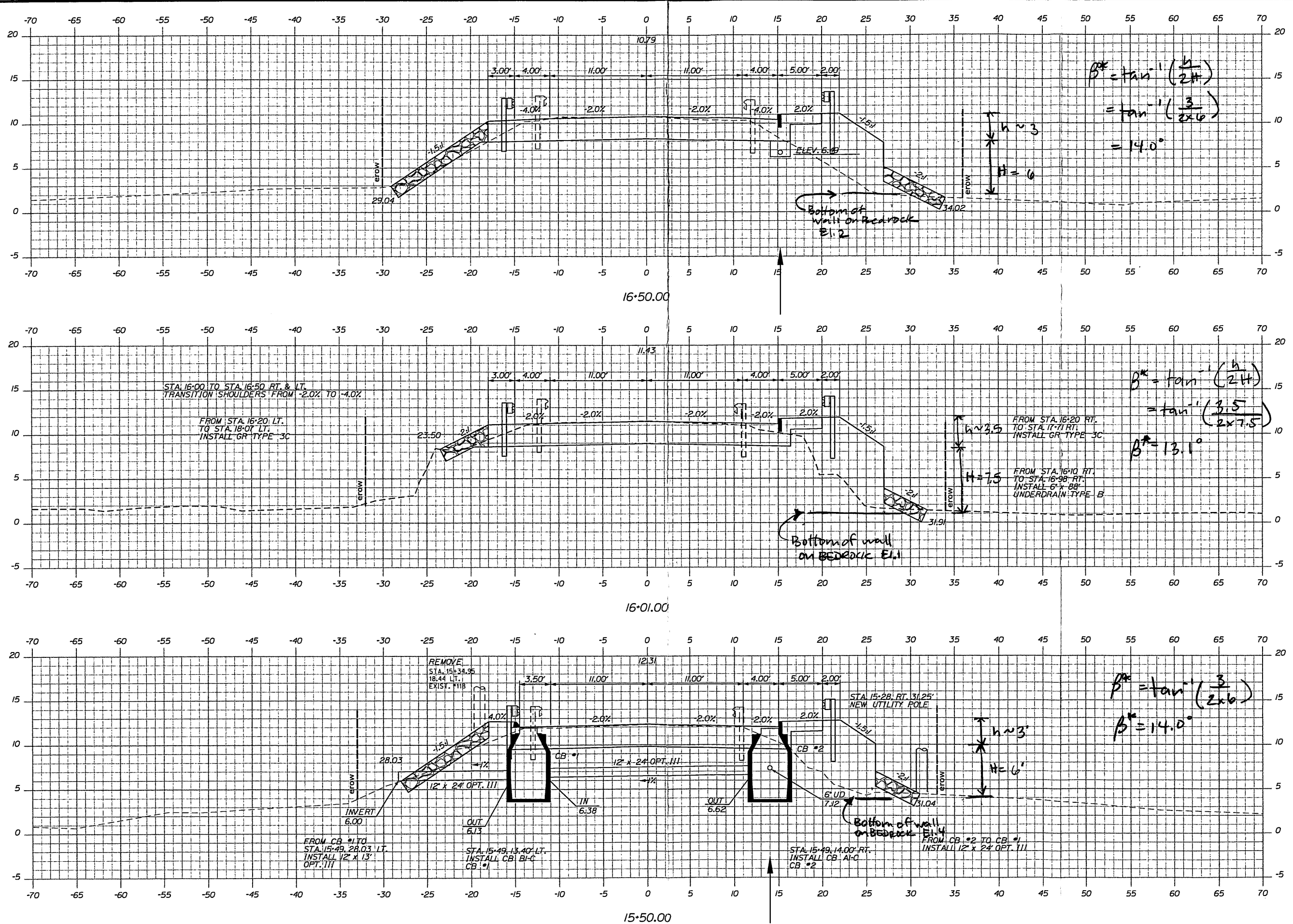
α angle of backface of wall to the horizontal, deg (assume 90 because stepped blocks and earth pressure will be applied to vertical faces) 90

ϕ angle of internal soil friction, deg (From MEDOT Bridge Design Guide Table 3-3, Granular underwater backfill) 32

β angle of backfill to the horizontal, deg (from VHB 60% design drawings, estimate effect of finite sloping backfill) 14

δ friction angle between fill and wall, deg (From MEDOT Bridge Design Guide Table 3-3, Granular underwater backfill) 24

$K_a =$	0.333
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STATE OF MAINE
 DEPARTMENT OF TRANSPORTATION

CLARK BRIDGE
 TREMONT ROAD OVER MARSHALL BROOK
 TREMONT HANCOCK COUNTY

CROSS SECTIONS
 STA. 15+50 TO STA. 16+50

BRIDGE NO. 3663
 WIN 19305.00
 BRIDGE PLANS

PROJ. MANAGER	DESIGN-DETAILED	CHECKED-REVIEWED	DATE
T. BRYANT	JMS	TSB	11/4/2015
DATE	SIGNATURE		
DATE	P.E. NUMBER		
DATE	DATE		

REVISIONS 1
 REVISIONS 2
 REVISIONS 3
 REVISIONS 4
 FIELD CHANGES

SHEET NUMBER
6
 OF 17

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

DESIGN PROPERTIES FOR EARTH PRESSURE

For walls with a total wall height, H, greater than or equal to 5 feet, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

At-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g. rigid frame bridges) or to cantilever walls where the heel is restrained and the base/stem connection prevents rotation of the stem.

3.6.5 Active Earth Pressure Coefficient

3.6.5.1 Coulomb Theory

The Coulomb theory should be used for the design of the following yielding walls:

- Gravity shaped walls and abutments
- Semi gravity walls
- ✱ ▪ Prefabricated modular walls with steep back faces (20° or less measured from the vertical)
- Cantilever walls and abutments with short heels (refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of short heel)

In all of these cases, interface friction (δ) develops along the back face of the wall. For horizontal or sloped backfill surfaces, the value of the coefficient of active lateral earth pressure (Coulomb), K_a , may be taken as:

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \cdot \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}} \right)^2}$$

where:

- α = angle (degrees) of backface of wall to the horizontal, as shown in Figure 3-1.
- ϕ = angle of internal soil friction (degrees), taken from Table 3-3.
- δ = friction angle (degrees) between fill and wall, taken from Table 3-3 for soil against concrete.
- β = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.

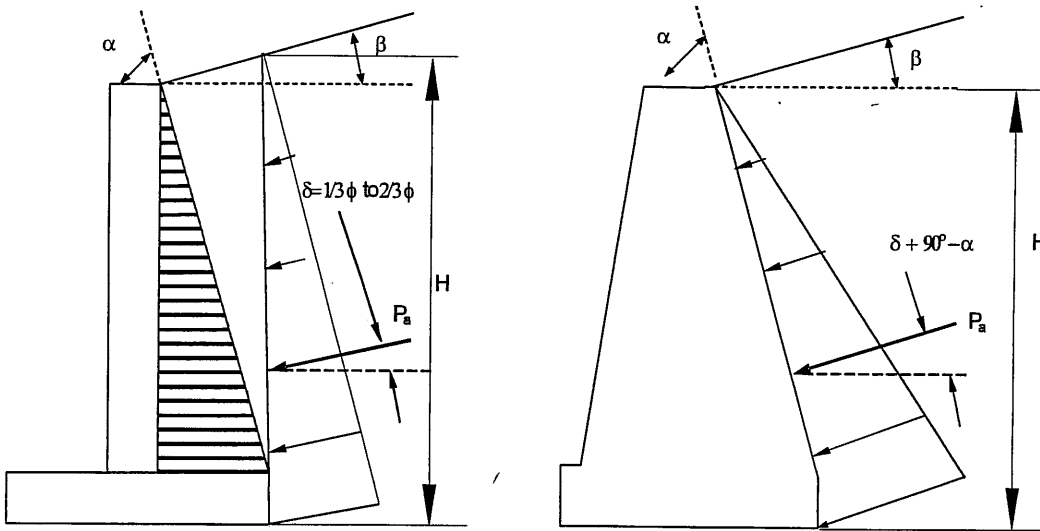
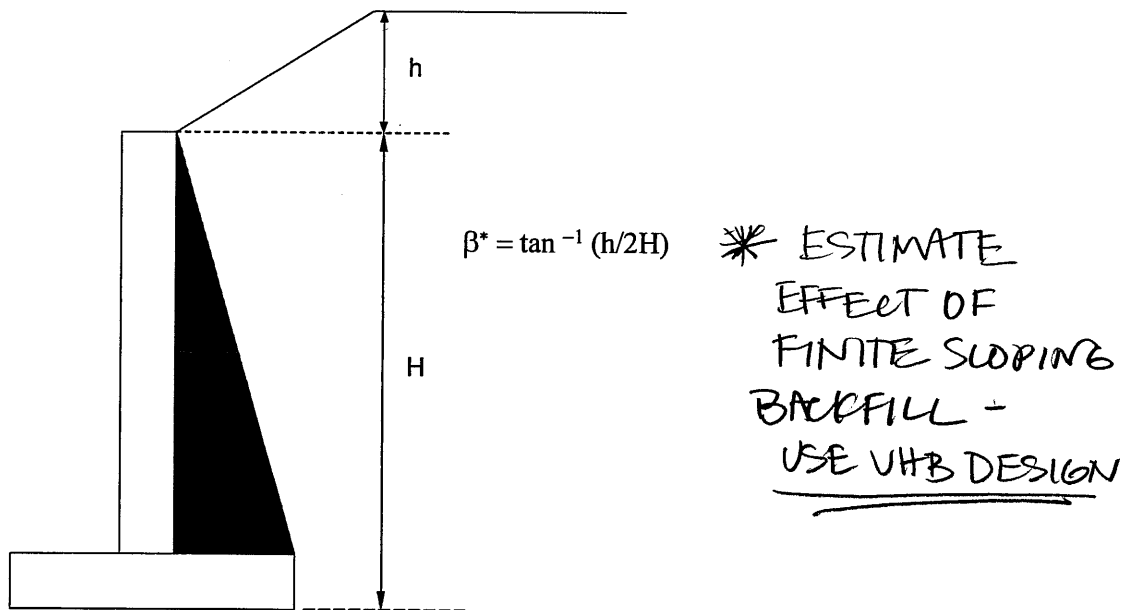


Figure 3-1 Coulomb Theory

The resultant earth pressure force, P_a , is oriented at an angle, either δ or $\delta + 90^\circ - \alpha$, as shown in Figure 3-1. The resultant acts at a distance, $H/3$, from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient, K_a , may be determined using a β value adjusted per AASHTO LRFD Figure 3.11.5.8.1-3 or substituted with β^* , as shown in Figure 3-2.



within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions shall be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions of Section 11. In no case shall highly plastic clay be used for backfill.

3.11.5.3—Active Lateral Earth Pressure

Coefficient, k_a

COULOMB FORT-WALL

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

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

in which:

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

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)

For conditions that deviate from those described in Figure 3.11.5.3-1, the active pressure may be calculated by using a trial procedure based on wedge theory using the Culmann method (e.g., see Terzaghi et al., 1996).

of lateral earth pressures assuming the most unfavorable conditions. See Article C3.11.1 for additional guidance on estimating earth pressures in fine-grained soils. If possible, cohesive or other fine-grained soils should be avoided as backfill.

C3.11.5.3

The values of k_a by Eq. 3.11.5.3-1 are based on the Coulomb earth pressure theories. The Coulomb theory is necessary for design of retaining walls for which the back face of the wall interferes with the development of the full sliding surfaces in the backfill soil assumed in Rankine theory (Figure C3.11.5.3-1 and Article C3.11.5.8). Either Coulomb or Rankine wedge theory may be used for long heeled cantilever walls shown in Figure C3.11.5.3-1a. In general, Coulomb wedge theory applies for gravity, semigravity and prefabricated modular walls with relatively steep back faces, and concrete cantilever walls with short heels.

For the cantilever wall in Figure C3.11.5.3-1b, the earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of soil to the left of the vertical plane is considered as part of the wall weight.

The differences between the Coulomb theory currently specified, and the Rankine theory specified in the past is illustrated in Figure C3.11.5.3-1. The Rankine theory is the basis of the equivalent fluid method of Article 3.11.5.5.

Silt and lean clay should not be used for backfill where free-draining granular materials are available. When using poorly draining silts or cohesive soils, extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions should be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions in Section 11. In no case should highly plastic clay be used for backfill.

3.11.5.8.2—Internal Stability

The load factor γ_p to be applied to the maximum load carried by the reinforcement T_{max} for reinforcement strength, connection strength, and pullout calculations (see Article 11.10.6.2) shall be EV , for vertical earth pressure.

For MSE walls, η_i shall be taken as 1.

C3.11.5.8.2

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The calculation method for T_{max} is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement modulus, from full scale walls at working stress conditions. The load factor EV , on the other hand, was determined in consideration of vertical earth pressure exerted by a soil mass without inclusions, and was calibrated to address uncertainties implied by allowable stress design for external stability for walls. EV is not directly applicable to internal reinforcement loads in MSE walls, since the calibration of EV was not performed with internal stability of a reinforced system in mind.

The use of EV for the load factor in this case should be considered an interim measure until research is completed to quantify load prediction bias and uncertainty.

3.11.5.9—Lateral Earth Pressures for Prefabricated Modular Walls

The magnitude and location of resultant loads and resisting forces for prefabricated modular walls may be determined using the earth pressure distributions presented in Figures 3.11.5.9-1 and 3.11.5.9-2. Where the back of the prefabricated modules forms an irregular, stepped surface, the earth pressure shall be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using

* Coulomb earth pressure theory.

C3.11.5.9

Prefabricated modular walls are gravity walls constructed of prefabricated concrete elements that are infilled with soil. They differ from modular block MSE structures in that they contain no soil reinforcing elements.

- δ = friction angle between fill and wall (degrees), taken from Table 3-3 for soil against concrete.
- β = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.

The resultant passive earth pressure force, P_p , is oriented at an angle, δ , to the normal drawn to the back face of the wall. The resultant passive earth load should be assumed to act at a distance of $H/3$ measured from the bottom of the footing.

3.6.7 Lateral Earth Pressures for Unconventional Retaining Walls

3.6.7.1 Mechanically Stabilized Earth Walls

For mechanically stabilized earth (MSE) walls, the resultant earth pressure, P_a , should be calculated using the active earth pressure coefficient, K_a , as described in Section 3.6.5.1 Coulomb Theory. For sloping and broken backfill surfaces, earth pressures should be calculated per AASHTO LRFD Figures 3.11.5.8 - 1 thru 3.

3.6.7.2 Prefabricated Modular Walls

This category includes prefabricated concrete modular gravity (PCMG) walls, metal bin walls, and gabion walls. Where the back of the prefabricated modules form an irregular stepped surface, the earth pressure should be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using Rankine earth pressure theory. The magnitude and location of the resultant earth loads may be determined using the earth pressure distributions shown in AASHTO LRFD Figures 3.11.5.9 -1 and 2.

Coulomb?
MISTAKE

3.6.8 Surcharge Loads – Live Load Surcharge

A live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height, H , behind the back of the wall. H is defined as the total wall height, measured along a vertical plane extending from the bottom of the footing up to the ground surface at the back of the wall. The additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by a height of soil, H_{eq} , defined in Table 3-3. The surcharge will result in the application of an additional uniform, constant horizontal pressure on the back of the wall having a magnitude P_s , taken as:

$$P_s = H_{eq} \cdot \gamma_s \cdot K$$

where:

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

earth pressure may act as high as $0.4H$ above the base of the wall for a mass concrete gravity retaining wall, where H is the total wall height measured from the top of the backfill to the base of the footing, where the wall deflects laterally, i.e., translates, in response to lateral earth loading. For such structures, the backfill behind the wall must slide down along the back of the wall for the retained soil mass to achieve the active state of stress. Experimental results indicate that the backfill arches against the upper portion of the wall as the wall translates, causing an upward shift in the location at which the resultant of the lateral earth load is transferred to the wall (Terzaghi, 1934; Clausen and Johansen et al., 1972; Sherif et al., 1982). Such walls are not representative of typical gravity walls used in highway applications.

For most gravity walls which are representative of those used in highway construction, nongravity cantilever retaining walls or other flexible walls which tilt or deform laterally in response to lateral loading, e.g., MSE walls, as well as walls which cannot translate or tilt, e.g., integral abutment walls, significant arching of the backfill against the wall does not occur, and the resultant lateral load due to earth pressure acts at a height of $H/3$ above the base of the wall. Furthermore, where wall friction is not considered in the analysis, it is sufficiently conservative to use a resultant location of $H/3$ even if the wall can translate.

3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, k_o

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin \phi'_f \quad (3.11.5.2-1)$$

where:

ϕ'_f = effective friction angle of soil
 k_o = coefficient of at-rest lateral earth pressure

For overconsolidated soils, the coefficient of at-rest lateral earth pressure may be assumed to vary as a function of the overconsolidation ratio or stress history, and may be taken as:

$$k_o = (1 - \sin \phi'_f) (OCR)^{\sin \phi'_f} \quad (3.11.5.2-2)$$

where:

OCR = overconsolidation ratio

Silt and lean clay shall not be used for backfill unless suitable design procedures are followed and construction control measures are incorporated in the construction documents to account for their presence. Consideration must be given for the development of pore water pressure

C3.11.5.2

For typical cantilevered walls over 5.0 ft high with structural grade backfill, calculations indicate that the horizontal movement of the top of the wall due to a combination of structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

In many instances, the OCR may not be known with enough accuracy to calculate k_o using Eq. 3.11.5.2-2. Based on information on this issue provided by Holtz and Kovacs (1981), in general, for lightly overconsolidated sands ($OCR = 1$ to 2), k_o is in the range of 0.4 to 0.6. For highly overconsolidated sand, k_o can be on the order of 1.0.

The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrink-swell, wet-dry and degree of saturation. Tension cracks can form, which considerably alter the assumptions for the estimation of stress. Extreme caution is advised in the determination



TECHNICAL MEMORANDUM

Date: November 30, 2017
To: Carl Ayers, Robert Blunt
cc: Meg Melendy, Mark Peterson
Email: cayers@vhb.com, rblunt@vhb.com
From: Megan Melendy, Mark Peterson
**RE: GEOTECHNICAL RECOMMENDATIONS FOR PHASE III REDESIGN
CLARK BRIDGE REPLACEMENT PROJECT
TREMONT, MAINE
MAINEDOT WIN 19305.00**

This memorandum summarizes our re-evaluation of our geotechnical recommendations for the redesigned Clark Bridge. The Maine Department of Transportation (MaineDOT) has contracted with VHB to provide Phase III Engineering Services for the resign of the bridge carry Tremont Road over Marshall Brook in Tremont, Maine. VHB has retained Golder Associates to provide Geotechnical Engineering Services during the redesign process. Our work was performed in accordance with our revised proposal dated March 2, 2017 and our Agreement dated April 14, 2017.

PROJECT BACKGROUND

Clark Bridge carries State Route 102 over Marshall Brook in Tremont, Maine, and was constructed in 1924 to replace two single span bridges. Clark Bridge is a 15-foot (ft) clear span cast-in-place concrete superstructure supported on dry-laid stone masonry abutments. Two narrow causeways with riprap surface side slopes support the west and east bridge approaches, and are about 20 and 100 ft long, respectively. According to the 1924 design drawings, the causeways were constructed of timber cribbing, with field stone used for abutments and retaining walls, and quarried stone rubble masonry for fill materials.

A replacement bridge design contract package was prepared and bid in June 2016 that included a 40-foot span concrete slab bridge and phased construction to maintain one-lane of alternating two-way traffic throughout construction. Golder prepared a *Geotechnical Recommendations for Final Design, Clark Bridge Replacement Project* design memorandum on March 9, 2016 that included recommendations for incorporation into the June 2016 design contract package. We understand that the project was bid in 2016 and MaineDOT rejected all bids. As part of their Phase III engineering services, VHB has redesigned the project using an upstream offline temporary bridge to maintain two lanes of traffic so the new bridge can be constructed all at once without staged construction. The new redesigned bridge maintains the existing 15 ft clear span over Marshall Brook.

171130 golder memo for clark final design support.docx

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RE-EVALUATION OF GEOTECHNICAL RECOMMENDATIONS

Golder has reviewed the *Plan Impacts Complete Submission Not for Construction* drawing set dated August 4, 2017 (PIC) for Tremont, Maine Clark Bridge prepared by VHB. The partial drawing set included sheets 1, 3-10, and 15-16 and included the preliminary plan view layout for the temporary bridge located upstream of the existing bridge and a plan and profile view of the proposed bridge replacement and approach retaining walls. The redesign bridge abutments are shown as bearing on bedrock, similar to the original June 2016 design. As shown in the redesign, it is likely that the top of bedrock bearing elevation at the redesigned abutment locations will be slightly lower than that shown for the original June 2016 design since the bedrock surface elevation is interpreted to drop toward the center of the channel. Details for the temporary bridge foundation are not shown however subsurface investigations performed during the preliminary design phase (2012) indicated that the top of bedrock elevation will likely be higher at the temporary bridge location than were observed adjacent to the existing bridge structure. Details for the redesigned approach prefabricated concrete modular gravity (PCMG) retaining walls at both approaches were not included in the PIC drawing set provided to Golder, but we assume the details will be similar to the original design since the locations of walls appear to be the same as the June 2016 design. The PCMG walls will need to be slightly longer to account for the shorter bridge span with the redesign, and the walls will need to be supported on a sloping bedrock surface that deepens towards the redesigned abutment locations. In addition, it appears that the precast block mat channel lining included in the June 2016 design was changed to heavy riprap in the PIC drawing set. We assume that issues relating to prevention of soil erosion will be addressed in the final design plan set.

Based on review of VHB's preliminary redesign plan set dated August 4, 2017 as detailed above, it is Golder's opinion that our geotechnical recommendations for the Clarke Bridge Replacement outlined in our March 9, 2016 letter would still apply to the Clark Bridge Redesign. Once a complete set of final design plans and specifications are prepared by VHB, Golder will review for compliance with our recommendations.

CLOSING

The geotechnical considerations included in this memorandum are preliminary and provided for the exclusive use of VHB for development of the Final Design Plans for the Phase III redesign replacement of the Clark Bridge in Tremont, Maine. The memorandum was prepared in accordance with generally accepted soil and foundation engineering practices. Our recommendations are based in part on the subsurface explorations completed. If actual conditions encountered vary from those described or referenced in this memorandum, Golder should be notified so that we may re-evaluate, and if necessary, revise our conclusions or recommendations.

Please contact us if you have questions regarding the information summarized herein.