Geotechnical Investigation

Dick's Brook Bridge Replacement,
Gros Morne National Park, NL

File No: 163567

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1.0 INTRODUCTION

Acting on the request and authorization of Harbourside Engineering Consultants (HEC), on behalf of Parks Canada Agency, Harbourside Geotechnical Consultants (HGC) have completed a geotechnical investigation for the proposed replacement of Dick's Brook Bridge in Gros Morne National Park, Newfoundland and Labrador.

The existing Dick's Brook Bridge is a two-lane, three-span, reinforced concrete T-beam bridge with a 9.2 m wide reinforced concrete deck that carries Newfoundland and Labrador Route 430 over Dick's Brook. The existing structure has two concrete abutments and two concrete piers. The bridge is located at or near the low point of a sag vertical curve with a relatively long tangent alignment along the west approach and a left-turning horizontal curve (looking from the bridge) along the east approach. Overhead power lines run continuously to the north side of the structure and Route 430, these lines run approximately 60 m from the edge of the existing bridge. No side access (e.g. intersections or driveways) were visible on Route 430 in the vicinity of the bridge and approaches.

The purpose of this geotechnical investigation was to determine the subsurface soil and rock conditions at the site and to provide geotechnical recommendations to aid with replacement of the Dick's Brook Bridge.

The scope of work completed for this project includes the following:

- Completion of a geotechnical field investigation, in two phases, consisting of eleven boreholes and four test pits;
- A laboratory testing program; and
- Preparation of this report presenting the findings of the field investigation and laboratory analyses, as well as comments and recommendations to aid with site earthworks and foundation design.

This report has been prepared specifically and solely for the project described herein.

2.0 SITE DESCRIPTION AND GEOLOGY

Dick's Brook Bridge, located between Rocky Harbour and Deer Lake, carries Newfoundland and Labrador Route 430 over Dick's Brook in Gros Morne National Park, NL. The area surrounding the crossing is forested. The existing bridge is located at or near the low point of a sag vertical curve and, further up the curve to both the east and the west, bedrock outcrops are visible in aerial photography and were observed during site reconnaissance.

At the crossing, Dick's Brook flows southwest from the Long-Range Mountains into Dick's Cove which is part of the larger South East Arm of Bonne Bay. The location of the existing bridge is shown on Drawing G1, Borehole Location Plan in Appendix C. The Long-Range Mountains lie to the north and east of the site with Killdevil Mountain prominently visible from the existing bridge.

Geological mapping in the vicinity of the site indicates that the overburden in the area consists principally of marine sediments including clay, silt, sand, gravel, and diamicton. Mapping indicates that these soils are generally moderately to well sorted and commonly stratified but may be massive at some locations.

Bedrock geology in the vicinity of the bridge is mapped as Paleozoic sedimentary rocks of the Labrador Group which includes quartzose sandstone (quartz arenite) of the Hawke Bay Formation and shale, sandstone, mudstone, siltstone, and limestone of the Forteau Formation (Big Hill Member and Mackenzie Mill Member). A fault occurring roughly along Dick's Brook separates these two formations near the crossing with the Hawke Bay Formation to the west and the Forteau Formation to the east.

3.0 INVESTIGATIVE PROCEDURES

3.1 GENERAL

The first phase of the geotechnical field investigation, comprised of seven boreholes (boreholes BH01 to BH06 and BH10), was conducted between December 6, 2016 and December 13, 2016. The second phase, which consisted of four boreholes (boreholes BH11 to BH14) and four test pits (test pits TP01 to TP04), was completed between May 23 and May 28, 2017.

Samples of the soil and bedrock were recovered from the test locations, classified in the field, and taken to our laboratory for final classification and testing. A detailed summary of the soil and bedrock conditions encountered, as well as the sampling and testing carried out, is presented on the borehole records in Appendix A. A document entitled “Symbols and Terms used on Borehole and Test Pit Records”, which clarifies terms used through this report, as well as terms and symbols used on the borehole and test pit records is also included in Appendix A.

3.2 BOREHOLES

Boreholes were advanced using a combination of 100-mm flight augers, HW-sized casing, and NW-sized casing. Soil sampling was carried out at regular intervals using conventional 50-mm diameter split spoon samplers while performing standard penetration testing as described in *ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*.

The standard penetration test (SPT) “N-value” is the number of blows required to advance a 50-mm outer-diameter split-spoon sampler a distance of 300 mm into the soil using a standardized drop height and weight. N-values generally provide an indication of soil consistency or compactness and may also be used to aid in estimation of other soil parameters. Occasionally, a 76-mm split-spoon sampler was used to retrieve samples with relatively large particle sizes. Additionally, relatively undisturbed soil sampling was performed at select locations (those with soft soils) using thin-walled metal Shelby tubes, as described in *ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.*

Field strength testing was performed in some borehole (field vane testing) and on some Shelby tube samples (miniature field vane, penetrometer, and torvane). A record of the field sampling and field strength testing is included on the borehole records in Appendix A; no corrections have been applied to the data presented in this report. Occasional strength testing was performed on split-spoon samples to give an indication of soil consistency but this testing was used in a qualitative manner and has not been reported on the borehole records.

Bedrock was cored using HQ- and NQ-sized diamond coring bits. The recovery and rock quality designation (RQD) of each run of core was recorded.

3.2.1 Phase 1 – Boreholes on Existing Alignment

During the first phase of the investigation, in December 2016, two boreholes were advanced in the vicinity of the west abutment of the existing structure (BH01, BH02), one near the west pier (BH03), one near the east pier (BH04), two near the east abutment (BH05, BH06), and one on the east approach (BH10). Boreholes BH03 and BH04 were advanced through the concrete deck of the bridge to reduce the environmental impact of accessing the locations. Boreholes BH07, BH08, and BH09 were initially planned south of the existing structure but were not advanced as

Parks Canada Agency did not provide approval to access these locations during this phase of the work. Conditions at each test location were observed and logged by experienced geotechnical personnel. Boreholes were drilled to depths ranging from 3.5 to 27.1 m below the ground surface. Upon completion of drilling, standpipe was installed in four boreholes (BH02, BH05, BH06, and BH10). Water levels were measured on December 13, 2016 as indicated on the borehole records in Appendix A. However, at the time of measuring, the standpipe was revealed to be blocked in three of the boreholes (BH02, BH06, and BH10).

3.2.2 Phase 2 – Boreholes South of Existing Alignment

During the second phase of the investigation, in May 2017, two boreholes were advanced south of current alignment and west of the brook (BH11 and BH12) and two east of the brook (BH13 and BH14). The boreholes were drilled to depths ranging from 10.7 to 31.9 m below the ground surface. Upon completion of drilling, standpipe was installed in each of the four boreholes. Water levels were measured on May 28, 2017 as indicated on the borehole records in Appendix A.

3.3 TEST PITS

Two test pits were advanced south of the existing west approach (TP01 and TP02) and two south of the existing east approach (TP03 and TP04). Test pits were excavated to depths of 1.8 to 5.5 m below the ground surface using a track-mounted excavator. The subsurface conditions were visually observed and compactness and consistency of the soils encountered were inferred based on excavator performance. Soil samples were taken from select locations of the various strata encountered.

3.4 LABORATORY TESTING

All samples of soil and rock recovered from the test locations were stored in water-tight containers and taken to our geotechnical laboratory for final classification and testing. Laboratory testing on select soil samples included water content determinations (*ASTM D2216 Standard Test Methods for Laboratory Determination of Water Content of Soil and Rock by Mass*), Atterberg Limits (*ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils)*, particle-size analyses (*ASTM D6913 Standard Test Method for Particle-Size Distribution of Soils Using Sieve Analysis*), isotropically-consolidated undrained (CIU) triaxial compression testing (*ASTM D4767 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils*), and consolidation testing (*ASTM D2435 Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading*).

Testing was performed on select samples of rock core to determine the unconfined compressive strength (*ASTM D7012-14 Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures*).

A summary of the testing performed is presented on the borehole records in Appendix A and in separate figures in Appendix B. Soil descriptions used throughout this report are in general accordance with the Unified Soil Classification System (*ASTM D2487 Standard Practice for Classification of Soils for Engineering purposes / ASTM D2488 Standard Practice for Description and Identification of Soils*).

3.5 SURVEYING

The locations and ground surface elevations for each borehole were surveyed by Yates and Woods LTD. Elevations are referenced to the Canadian Geodetic Vertical Datum of 1928 (CGVD28).

4.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the test locations generally consisted of the following sequence:

- Asphalt and Concrete
- Fill
- Rootmat and Topsoil
- Sand and Gravel
- Clay
- Diamicton
- Sandy Silt
- Bedrock

Not all strata were encountered at all test locations. The subsurface conditions observed in the boreholes are summarized in Table 1 and the following paragraphs and are described in additional detail on the borehole records in Appendix A.

Table 1 Summary of Subsurface Conditions

Location	Ground Elevation[a] (m)	Thickness							Bedrock		Groundwater		Total Depth (m)
		Asphalt/ Concrete (m)	Fill (m)	Rootmat and Topsoil (m)	Sand and Gravel (m)	Clay (m)	Diamicton (m)	Sandy Silt (m)	Depth to Surface (m)	Surface Elevation[a] (m)	Depth (m)	Elevation[a] (m)	
BH01	14.98	0.30	9.00	-	-	3.04	6.76	-	19.10	-4.12	-	-	21.79
BH02	14.99	0.25	9.20	0.38	-	3.81	5.26	3.35	22.25	-7.26	> 9.78	< 5.21	27.08
BH03	6.93	-	2.54	-	-	-	0.30	-	2.84	4.09	-	-	4.90
BH04	6.61	-	1.37	-	-	-	-	-	1.37	5.24	-	-	3.51
BH05	15.39	0.17	2.57	-	-	4.02	1.72	-	8.48	6.91	5.82	9.57	14.65
BH06	15.37	0.20	2.01	-	-	5.03	1.98	-	9.22	6.15	> 4.42	< 10.95	11.91
BH10	16.39	-	0.91	-	-	5.39	> 0.15	-	> 6.45	< 9.94	> 1.37	< 15.02	6.45
BH11	4.28	-	-	0.15	4.57	1.60	3.28	19.58	29.18	-24.90	0.63	3.65	31.90
BH12	5.12	-	-	0.15	-	1.07	4.98	10.82	17.02	-11.90	1.07	4.05	22.76
BH13	7.39	-	-	-	-	2.29	3.75	-	6.04	1.35	1.67	5.72	10.67
BH14	13.67	-	-	0.15	0.76	5.64	3.66	-	10.21	3.46	0.33	13.34	12.67
TP01	16.5	-	-	0.5	0.6	0.4	0.3	-	1.8	14.7	1.5	15.0	1.8
TP02	8.0	-	-	0.2	-	> 4.7	-	-	> 4.9	< 3.1	-	-	4.9
TP03	14.7	-	-	0.2	-	> 5.3	-	-	> 5.5	< 9.2	-	-	5.5
TP04	15.5	-	-	0.2	-	> 5.3	-	-	> 5.5	< 10.0	-	-	5.5

(a) Elevations are referenced to CGVD28.

4.1 SUBSURFACE CONDITIONS

4.1.1 Asphalt and Concrete

A surficial layer of asphalt or concrete was encountered at the top of the four boreholes advanced near the existing abutments (BH01, BH02, BH05, and BH06). Boreholes BH03 and BH04 were advanced through the bridge deck but were only logged from the ground surface below the bridge.

Concrete (Approach Slab)

Boreholes BH01 and BH02 were advanced through the concrete approach slab of the existing structure. At these locations, the concrete was approximately 250 and 300 mm thick.

Asphalt Concrete

A layer of asphalt concrete was encountered at the surface of boreholes BH05 and BH06. Where encountered, this layer was approximately 170 to 200 mm thick.

4.1.2 Fill

Fill was encountered at the surface or below the surficial layer of each of the seven boreholes advanced along the existing alignment. Generally, fill can be divided into two groups: base gravel and sand and gravel.

Base Gravel

A layer of fill comprised of brown to grey gravel to sand with silt and gravel was encountered below the surficial layer in the boreholes advanced through the existing embankment (BH01, BH02, BH05, BH06, and BH10). This layer is the base gravel for the road structure and at the test locations ranged in thickness from 0.9 m (BH10) to 2.6 m (BH05).

The results of particle-size analyses on two samples from this layer are presented in Table 2. Based on our field classification and laboratory testing, this layer may be classified as gravel with silt and sand to silty sand with gravel.

The water content of two samples of this layer were 3 and 4 percent.

Table 2 Particle-Size Analyses – Fill: Base Gravel

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification[a]	Material Composition by Weight (%)		
				Gravel	Sand	Fines[b]
BH01	SS1	0.5 to 1.1	Silty Gravel with Sand	49	40	12
BH06	SS2	0.8 to 1.4	Well-Graded Sand with Silt and Gravel	33	58	9

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

Sand and Gravel

Brown to grey fill comprised primarily of sand, gravel, and silt was encountered below the base gravel in boreholes advanced on the existing alignment, near the west abutment, and boreholes advanced through the bridge deck, near the existing piers.

This layer was 8.1 to 8.2 m thick near the west abutment (BH01 and BH02), 2.5 m thick near the west pier (BH03), and 1.4 m thick near the east pier (BH04). Occasional to frequent cobbles and boulders were encountered throughout this layer. In the boreholes near the east abutment (BH01 and BH02) quartzose sandstone rockfill was encountered in the bottom portion of this layer (below el. 7.0 m).

The results of a particle-size analysis on one sample from the sand and gravel fill is presented in Table 3. Based on our field classification, visual-manual inspection, and laboratory testing, the sand and gravel fill may be classified as grey gravel to gravel with silt and sand. In BH03 the fines were silty to clayey.

Table 3 Particle-Size Analysis – Fill: Sand and Gravel

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification(a)	Material Composition by Weight (%)		
				Gravel	Sand	Fines(b)
BH01	SS8	6.0 to 6.6	Well-Graded Gravel	93	5	1

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

The water content of five samples of the sand and gravel fill ranged from 2 to 11 percent with an average of 6 percent.

4.1.3 Rootmat/Topsoil

Rootmat and topsoil was encountered at the surface of three of the eleven boreholes (BH11, BH12, BH14) and all four test pits advanced south of the existing alignment. At borehole BH13, the rootmat and topsoil had been removed from the area during construction of an access road to the borehole location as a part of the field investigation. Where encountered, this material was approximately 0.2 to 0.5 m thick.

A buried layer of rootmat and topsoil was also encountered in borehole BH02. This material was the rootmat and topsoil at the ground surface before placement of the overlying fill, and contained roots, frequent organic material, and occasional wood. At the borehole location, this layer was approximately 0.4 m thick.

4.1.4 Sand and Gravel

A heterogeneous layer of relatively coarse-grained deposits ranging from silty sand to gravel with silt and sand was encountered below the rootmat in topsoil in two of the boreholes (BH11 and BH14) and one test pit (TP01) advanced south of the existing alignment. At these locations, the thickness of the sand and gravel ranged from 0.6 m (TP01) to 4.6 m (BH11) on the west side of the brook and was 0.8 m thick at the one location it was encountered on the east side of the brook (BH14).

A particle-size analysis was performed on one sample from this layer. The results of these analyses are summarized in Table 4, below.

Table 4 Particle-Size Analysis – Sand and Gravel

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification[(a)]	Material Composition by Weight (%)		
				Gravel	Sand	Fines[(b)]
BH11	SS4	2.2 to 2.8	Silty Sand	14	68	19

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

The natural water content of four samples of the sand and gravel ranged from 13 to 42 percent with an average of 23 percent.

Based on SPT N-values and field observations, this layer can generally be described as loose to compact.

4.1.5 Clay

A layer of brown clay was encountered below the fill or rootmat and topsoil layers in all boreholes advanced near the existing abutments and in borehole BH10 advanced on the east approach. This layer was also encountered in all test locations advanced south of the existing alignment. The clay was not encountered in the boreholes advanced near the existing piers (i.e. borehole BH03 and BH04). The thickness of the clay ranged from 0.4 m to 5.6 m at locations where the full thickness of the later was determined. Test pits TP02, TP03, and TP04 were terminated 4.7 to 5.3 m within this layer.

Two particle-size analyses were performed on samples of the clay; 92 to 99 percent (by weight) of the particles classified as silt- and clay-sized. The results of the particle-size analyses are summarized in Table 5 and on the relevant figures in Appendix B.

The results of Atterberg limit testing on twenty samples of the clay layer are presented in Table 6. West of the bridge, the clay encountered had a liquid limit less than fifty and thus can be classified as lean clay. East of the bridge, in BH06, BH10, and BH14 the top 1.1 to 2.6 m of the clay generally had a liquid limit greater than 50 (and elevated water contents) and thus can be classified as fat clay. Overall, the liquid limit and plasticity index of the clay was observed to reduce with depth.

The natural water content of 28 samples of the lean clay ranged from 21 to 48 percent with an average of 31 percent. The natural water content of five samples of the fat clay were between 45 and 72 percent with an average of 54 percent.

Table 5 Particle-Size Analyses – Clay

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification(a)	Material Composition by Weight (%)		
				Gravel	Sand	Fines(b)
BH01	ST14	10.5	Lean Clay	0	1	99
BH11	ST9	5.6	Lean Clay	1	7	92

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

Table 6 Atterberg Limits Results – Clay

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification(a)	Natural Water Content (%)	Plastic Limit	Liquid Limit	Plasticity Index
BH01	SS13	9.6 to 10.2	Lean Clay	29	18	34	16
BH01	ST14	10.5	Lean Clay	28	16	28	12
BH02	ST15	11.3	Lean Clay	31	17	32	15
BH02	SS16	11.6 to 12.2	Lean Clay	28	16	28	12
BH05	SS06	3.4 to 4.0	Lean Clay	37	20	42	22
BH05	SS09	5.2 to 5.8	Lean Clay	28	16	30	14
BH06	ST06	3.7	Fat Clay	46	21	54	33
BH06	ST09	6.0	Lean Clay	28	17	30	13
BH10	SS03	1.4 to 2.0	Fat Clay	60	25	72	47
BH10	SS05	2.9 to 3.5	Lean Clay	40	21	46	25
BH10	SS08	5.0 to 5.6	Lean Clay	28	17	32	15
BH11	ST09	5.6	Lean Clay	24	16	25	9
BH13	SS02	0.6 to 1.2	Lean Clay	30	19	36	17
BH14	ST03	1.9	Fat Clay	72	27	67	40
BH14	SS06	3.5 to 4.1	Lean Clay	48	20	43	23
BH14	ST07	5.6	Lean Clay	35	19	35	16
BH14	SS10	5.9 to 6.5	Lean Clay	29	16	27	11
TP02	GS01	4.6 to 4.9	Lean Clay	34	18	37	19
TP03	GS01	5.2 to 5.5	Lean Clay	30	16	31	15
TP04	GS01	4.3 to 4.6	Lean Clay	33	18	33	15

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

Based on SPT N-values, visual-manual field observations, and field testing with a pocket penetrometer, torvane, miniature field vane, and field vane, the consistency of the layer can generally be described as firm to stiff west of the bridge and soft to firm, with portions that are firm to stiff, east of the bridge. Testing with the miniature field vane was completed at sixteen locations and resulted in (uncorrected) undrained shear strengths ranging from 20 to 58 kPa; field vane testing at five locations resulted in (uncorrected) undrained shear strengths of 64 to 104 kPa.

Consolidation testing was performed on five samples of the clay. The results of this testing are included in Appendix B and summarized in Table 7, below.

Table 7 Consolidation Testing - Clay

Location	Sample No.	Sample Depth (m)	Compression Index, C_c	Recompression Index, C_r	Preconsolidation Pressure (Casagrande), σ_p' (kPa)
BH05	ST7	4.3	0.12	0.026	305
BH05	ST10	6.2	0.10	0.026	325
BH14	ST3	2.1	0.89	0.180	150
BH14	ST5	3.3	0.63	0.110	210
BH14	ST9	5.5	0.18	0.025	200

Consolidated isotropic undrained triaxial testing was performed on three specimens to estimate the drained strength properties of the clay and the results of this testing is included in Appendix B.

4.1.6 Diamicton

A layer of poorly sorted soils with particle sizes ranging from clay to boulders (i.e. diamicton) was encountered in ten of the eleven boreholes (all borehole except BH04, which was advanced near the east pier) and in test pit TP01. The diamicton was generally encountered below the clay layer and in BH03 was encountered directly below the existing fill.

The diamicton was 5.3 to 6.8 m thick in two boreholes advanced near the west abutment (BH01, BH02), 1.7 to 2.0 m thick in the two boreholes advanced near the east abutment (BH05, BH06), and 0.3 m thick near the existing west pier (BH03). This layer was not encountered in the borehole advanced near the existing east pier (BH04) and the borehole on the east approach (BH10) was terminated approximately 0.2 m into the layer. In the boreholes south of the existing alignment, the thickness of this layer was 3.3 to 5.0 m west of the brook (BH11, BH12) and 3.7 to 3.8 m east of the brook (BH13, BH14). In test pit TP01 this layer was 0.3 m thick. The remaining test pits were terminated in the clay that was generally overlying this layer.

The results of particle-size analyses on seven samples of the diamicton are presented in Table 8 and the results of Atterberg limit testing on one sample is presented in Table 9. Based on our field classification, visual-manual inspection, and the laboratory testing this layer can generally be described as silty sand with gravel with silt and sand. In borehole BH06 there was approximately 1.9 m of clayey sand with gravel overlying the silty gravel.

The natural water content of 18 samples of diamicton ranged from 8 to 14 percent with an average of 10 percent.

Based on N-value and field observations this layer can generally be described as compact to dense.

Table 8 Particle-Size Analyses – Diamicton

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification[a]	Material Composition by Weight (%)		
				Gravel	Sand	Fines[b]
BH01	SS19	14.2 to 14.8	Silty Sand with Gravel	34	41	24
BH02	SS19	14.8 to 15.4	Silty Gravel with Sand	41	38	21
BH06	SS11	7.2 to 7.9	Clayey Sand with Gravel	22	33	45
BH11	SS14	9.0 to 9.6	Silty Sand with Gravel	29	37	34
BH12	SS8A	4.4 to 4.7	Well-Graded Gravel with Silt and Sand	58	37	5
BH14	SS14	8.0 to 8.6	Poorly Graded Sand with Silt and Gravel	34	54	12
TP01	GB01	1.5 to 1.8	Silty Gravel with Sand	43	42	15

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

Table 9 Atterberg Limits Results – Diamicton

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification[a]	Natural Water Content (%)	Plastic Limit	Liquid Limit	Plasticity Index
BH06	SS11	7.5	Clayey Sand with Gravel	12	15	23	8

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

4.1.7 Sandy Silt to Silty Sand

On the west side of the brook, in boreholes BH02, BH11, and BH12, a layer comprised of sandy silt to silty sand was encountered. This layer was 3.4 m thick in BH02, 19.6 m thick in BH10, and 10.8 m thick in BH11. Field observations indicate that this layer is partially cemented.

The results of three particle-size analyses of this material is presented in Table 10. Atterberg limit testing indicated that the layer has non-plastic fines. Based on our field classification, visual-manual inspection, and the laboratory testing the layer can be described as sandy silt to silty sand with gravel.

The natural water content of 14 samples of this layer was between 8 and 27 percent, with an average of 19 percent.

N-values obtained during SPT testing ranged from 6 to 91 with refusal occurring on four occasions. Generally, based on N-value, the layer can be described as compact to very dense.

Table 10 Particle-Size Analyses – Sandy Silt

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification(a)	Material Composition by Weight (%)		
				Gravel	Sand	Fines(b)
BH02	SS22	19.5 to 20.1	Sandy Silt	14	23	63
BH11	SS22	16.0 to 16.6	Silty Sand with Gravel	15	34	50
BH12	SS17	11.6 to 12.2	Silty Sand with Gravel	15	56	29

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For particle-size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

4.1.8 Bedrock

Bedrock was encountered and cored in ten of the eleven boreholes put down as part of this investigation. The bedrock surface varies over the site. For example, bedrock was encountered at a depth of 29.2 m in borehole BH11, which is south of the existing alignment, but outcrops are visible at nearby locations. The bedrock surface was higher along the existing alignment and generally slopes up to the east and to the west from the brook.

Behind the existing west abutment, bedrock was encountered at elevations of -4.1 and -7.3 m in BH01 and BH02, respectively. Behind the existing east abutment, bedrock was encountered at elevations of +6.9 m and +6.2 m in boreholes BH05 and BH06, respectively. Within the span of the existing bridge, in borehole BH03 and BH04, the bedrock surface was encountered at elevations of +4.1 and +5.2 m, respectively.

South of the existing alignment to the west of the brook, in boreholes BH11 and BH12, bedrock was encountered at elevations of -24.9 and -11.9 m, while in boreholes east of the brook, BH13 and BH14, bedrock was encountered elevations +1.4 and +3.5 m. Further west, in test pit TP01, bedrock was encountered at an elevation of +14.7 m.

Bedrock outcrops are visible beside the road along both approaches and the location of some of these outcrops were recorded and are plotted on Drawing G1 in Appendix C.

Bedrock was primarily grey to purple quartzose sandstone (quartz arenite). In borehole BH02, the top 2.4 m of bedrock included interbedded grey shale and reddish-brown siltstone. In boreholes BH11 and BH12, west of the brook, where bedrock was encountered at a lower elevation, bedrock was grey siltstone.

Based on the RQD of the recovered core, the bedrock may generally be classified as very poor quality in boreholes advanced to the west of the brook, and fair to good quality east of the brook.

Five unconfined compressive strength tests were performed on samples of the quartzose sandstone. Unconfined compressive strengths obtained from this testing ranged from 86 to 243 MPa and are summarized in Table 11. Based on these tests, and on field testing, the quartzose sandstone may be classified as strong to very strong.

Samples of the siltstone were generally of very poor quality and were interpreted in the field as weak to very weak. Samples of the siltstone were highly fractured and generally not suitable for unconfined compressive strength testing.

Table 11 Unconfined Compressive Strength Test Results

Borehole	Depth (m)	Rock Type	Unconfined Compressive Strength (MPa)
BH03	4.2	Quartzose Sandstone	86
BH04	2.1	Quartzose Sandstone	61
BH05	9.0	Quartzose Sandstone	175
BH05	12.0	Quartzose Sandstone	243
BH06	11.0	Quartzose Sandstone	128

4.1.9 Groundwater

Groundwater levels were measured in borehole BH05 on December 13, 2016 and in boreholes BH11 to BH14 on May 28, 2017. The water level in borehole BH05, advanced through the existing embankment, was 5.8 m below the ground surface (el. +9.6 m). The water levels in boreholes BH11, BH12, and BH13, which were advanced on lower ground, were 0.6 to 1.7 m below the ground surface (el. +3.7 to +5.7 m). The water level in borehole BH14 was at a depth of 0.3 m (el. +13.3 m). Standpipes were also installed in boreholes BH02, BH06, and BH10 but the standpipes were blocked at depths of 9.8, 4.4, and 1.4 m, respectively.

Water levels may fluctuate with brook level, construction activity, as well as individual weather events and climatic and seasonal weather trends.

5.0 DISCUSSION AND RECOMMENDATIONS

We understand that a single-span bridge is proposed to replace the existing Dick's Brook Bridge and the new structure is to be constructed to the south of the existing bridge with a minimum of about 19.3 m clear between the two structures (at the east abutment). The proposed design of the replacement structure indicates that the bridge superstructure will be founded on fully integral piled abutments supporting a reinforced concrete cap with cantilevered wing walls.

The horizontal alignment of the road will be adjusted as it approaches the bridge from both the east and the west. To reach the proposed grades, which are up to about 15.0 m above the existing grades, fill will be required on the approaches. Due to the presence of soft to firm clays throughout the site, construction of the approach embankments will require the use of stabilizing measures such as counterforce berms and lightweight fill (e.g. expanded polystyrene fill).

As two lanes of traffic are required at all times throughout construction, the existing structure will be used as an on-site detour until traffic is diverted onto the completed new bridge.

Based on our geotechnical investigation, and our understanding of the proposed design, for the bridge foundations we are providing recommendations for piles driven to bedrock. The following subsections provide geotechnical recommendations to support site preparation and foundation design.

5.1 SITE PREPARATION

At locations where the thickness of fill to be placed is less than 3 m, surficial rootmat, topsoil, and other soils containing a significant proportion of organic material should be removed from below the footprint of pile caps, structural fills, and approach fills to expose the in-situ sand and gravel fill, native sand and gravel, native clay, or diamicton.

At locations where the thickness of fill to be placed is in excess of 3 m in thickness, the site should be cleared, tree stumps should be cut near flush with the ground surface, and boulders should be removed. The rootmat and topsoil can remain in place and fill placed directly over it. This approach will limit the amount of exposed clay, which is sensitive, subject to softening, prone to deterioration due to disturbance, and will act as a poor working surface.

The native clay will be very susceptible to deterioration due to trafficking, especially in the presence of water. The clay will soften as it gets disturbed, and there is potential for equipment to get stuck on or in the clay. Therefore, at locations where construction traffic will travel, a stabilizing layer of rock fill should be placed to provide a working surface. This layer should be a minimum of 600 mm thick and the thickness may need to be increased depending on the volume, frequency, and loading of traffic. A woven geotextile should be placed wherever a coarse-grained fill is placed over a fine-grained material (e.g. where rock fill is placed over topsoil or clay). Additionally, consideration should be given to using geogrid below the rock fill to stabilize areas used as haul roads.

5.2 EXCAVATIONS IN SOIL

Due to the new grades being above the existing grades, large-scale excavations are not anticipated near the new structure. However, on the new alignment away from the bridges, cuts are anticipated to meet the design grades.

Excavations in the clay (above or below the groundwater) may temporarily stand near vertical to a depth of several meters. However, the stability of the clay soil is complex and varies over the site with the properties of the clay. Furthermore, due to the freeze-thaw cycles to which the near-surface clay has been subjected, the clay is likely weathered and fissured within the depth of frost penetration (approximately 1.8 m) which will affect its stability.

For planning purposes, excavations of less than 3.0 m in the clay can be assumed to be stable at slopes of 1.5H:1V in the short-term. At locations where the excavation is to exceed 3.0 m, the stability of excavations in the clay should be assessed on a case-by-case basis.

Long-term cut slopes in the clay above the groundwater table should be 3H:1V or flatter. To reduce sloughing and movement, excavations below the groundwater table will have to be flattened further. Slopes excavated in clay should be protected by protective layer of rock fill or sod.

Temporary excavations in the sand and gravel should be no steeper than 1.5H:1V. Flatter slopes will be required when excavating below the groundwater table.

All excavations should follow all applicable safety regulations and should be frequently monitored for any indication of instability. Sloping of excavations deeper than 1.2 m is required to meet provincial regulations wherever personnel will be entering the excavation.

5.3 EXCAVATIONS IN ROCK

Bedrock outcrops are visible near the brook north of the existing bridge and on both sides of the road along the existing approaches to the east and west of the bridge. Furthermore, shallow bedrock was encountered in test pit TP01 (near the proposed west approach) and in boreholes BH03 and BH04 (near the piers of the existing structure).

As the road is going to be re-aligned to the east, and the vertical profile changed to increase the radius of the vertical sag-curve, additional rock cuts are anticipated to meet the design grades.

Based on a review of the outcropping, the bedrock has a steeply-dipping set of joints that have a strike subparallel to the road. Excavation into this rock should follow, or be flatter than, the fracturing in the rock so that this set of joints does not “daylight” on the face of the cut slopes. Based on our preliminary assessment, rock cuts of 1H:4V can be used in design. During construction, the slopes should be assessed by qualified geotechnical or geological personnel during construction to determine if additional excavation or anchoring is appropriate.

A rockfall catchment area should be designed to prevent or limit rock fall originating from the slope above the highway from reaching the highway lanes. Design of the catchment area should consider the ditch height, width, the height of the slope, the steepness of the slope, the type and quality of the bedrock, as well as any other slope stabilization measures used.

Care should be taken during blasting operations to limit the amount of overbreak during blasting as this can act to destabilize the slope. If blasting damages the rock below the intended surface, additional rock removal may be required to ensure the rock cut is stable.

5.4 STRUCTURAL FILL

In the proposed design, the elevation of the underside of the pile caps are above existing grade; structural fill should be used to achieve the proposed subgrade elevation. Structural fill should consist of well-graded rock fill with a maximum particle size of 200 mm and a fines content less than 12 percent. Granular "B" or Granular "C" as specified by the Government of Newfoundland and Labrador's Department of Transportation and Works Specifications Book are examples of suitable materials.

Where placed, structural fill should extend through the full extent of the fills in front of and transversely from the pile cap. Structural fill should extend behind the abutments a distance beyond the outside edge to include a structural splay of 1H:1V (the extents of the zone of influence beneath the pile cap). If fill is placed below the pile caps before the approach fills are placed, shallower slopes will be required to ensure stable slopes during construction (i.e. 1H:1V slopes will not have a sufficient factor of safety against slope instability).

Structural fill should be compacted to 100 percent of the standard Proctor maximum dry density as determined by *ASTM D698 Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort*. For Materials where Proctor densities are not applicable, such as coarse rock fills, material should be compacted to a relative density of at least 80 percent. All structural fill should be placed at a water content that allows compaction to the specified density.

Appropriate lift thicknesses for structural fill will vary with the compaction equipment used to ensure that he required density is achieved over the entire lift. Typically, a rolling pattern of about six slow passes with a 10-ton vibrating roller would be required for a 300 to 500 mm lift. Placement of structural fill should be monitored by experienced geotechnical personnel to ensure that the required density is achieved.

5.5 APPROACH FILL

5.5.1 Slope Stability

A layer of soft to stiff clay was encountered throughout the site and at some locations this layer is more than 5 m thick. The stability of the approach fills will largely be controlled by the thickness and strength properties of the underlying clay which varies throughout the site. In order to maintain a minimum factor of safety of 1.3, the use of stabilizing measures will be required to obtain the design grades that are as much as about 15 m above the existing grades.

The use of counterforce berms and lightweight fill (e.g. expanded polystyrene, EPS) are stabilizing measures that can be used to achieve the design grades.

- Counterforce berms are earthen berms constructed near the toe of the slope placed to provide support against global instability. The elevation of the berm, the width of the berm, and the slopes from the berm will vary over the site depending on the proposed site geometry and subsurface conditions.
- EPS fill (geofoam) is a lightweight soil substitute material used to decrease the weight of fill which will act to increase the stability of the embankment and reduce the amount of induced settlement. The EPS fill is typically manufactured into relatively large lightweight blocks which are hand-placed on site.

The following properties have been used in slope stability analyses:

- **Rock Fill**
  - Total Unit Weight 22.0 kN/m^3
  - Submerged Unit Weight 12.2 kN/m^3
  - Effective Friction Angle 38 degrees
  - Effective Cohesion 0 kPa
- **Approach Fill**
  - Total Unit Weight 21.5 kN/m^3
  - Submerged Unit Weight 11.7 kN/m^3
  - Effective Friction Angle 36 degrees
  - Effective Cohesion 0 kPa
- **Diamicton**
  - Total Unit Weight 22.0 kN/m^3
  - Submerged Unit Weight 12.2 kN/m^3
  - Effective Friction Angle 34 degrees
  - Effective Cohesion 0 kPa
- **Lean Clay**
  - Total Unit Weight 18.5 kN/m^3
  - Submerged Unit Weight 8.7 kN/m^3
  - Undrained Shear Strength 40 kPa
  - Effective Friction Angle 26 degrees
  - Effective Cohesion 0 kPa
- **Fat Clay**
  - Total Unit Weight 17.5 kN/m^3
  - Submerged Unit Weight 7.7 kN/m^3
  - Undrained Shear Strength 40 kPa
  - Effective Friction Angle 22 degrees
  - Effective Cohesion 0 kPa

5.5.2 Settlement

Consolidation of the native clay layer will occur when additional weight is placed on the site to construct the approach embankments. Settlements will be relatively small so long as the effective stress on the clay is less than the preconsolidation pressures. Once the load exceeds this pressure, which will begin to occur with embankments heights on the order of 6 to 10 m above the existing grade, relatively large settlements will occur. Consolidation tests have been performed to assess the compressibility of the clay and to estimate its preconsolidation pressure. This testing indicates that, in particular, the fat clay is highly compressible.

Settlements of the clay layer will continue for years after completion of construction. At some locations on the site, primary consolidation may take up to five years to complete. Once primary consolidation is complete, settlements due to secondary compression (i.e. creep) will continue for decades. However, the rate of secondary settlement will decrease with time.

The use of EPS as lightweight fill will reduce the settlement at all stages, but substantial settlement should be planned for. We have performed settlement analyses that may be summarized as follows:

- Near the west abutment, the finished grade will be raised more than 15 m above the existing grade, from a minimum elevation of about 4.0 m to a finished elevation of 20.6 m at the abutment (and higher along the west approach). If approach fill or structural fill is placed to an elevation of about 15.0 m and EPS is placed above this elevation and underlies the pavement structure detailed herein, total settlements can be estimated as about 250 mm with about 150 mm occurring after completion of construction, assuming a typical construction schedule. Settlements due to secondary compression will continue for the life of the structure and will total approximately 50 to 75 mm over the 75-year design life.

- Near the east abutment, the finished grade will be raised approximately 6.0 m above the existing grade, from a minimum elevation of about 14.0 m to a finished elevation of 19.7 m at the abutment (and higher along the east approach). If approach fill or structural fill is placed to an elevation of about 16.0 m and EPS is placed above this elevation and underlies the pavement structure detailed herein, total settlements can be estimated as about 250 mm with approximately 125 mm occurring after completion of construction, assuming a typical construction schedule. Settlements due to secondary compression will continue for the life of the structure and will total approximately 50 mm.
- Further from the structure, settlements will vary based on fill height and the thickness and properties of the various subsurface strata. Typically, settlements will increase in areas where thicker fills are placed and in areas that are underlain by relatively thick clay deposits. Post-construction settlements will be negligible in areas where the embankment is constructed directly on diamicton or bedrock. Overall, settlements of up to 350 mm are estimated, with approximately 250 mm occurring after completion of construction. Settlements due to secondary compression will vary but may total up to about 100 mm over the 75-year design life of the structure.

Although care has been taken throughout the work, including during the field investigation, laboratory testing, and settlement analyses, estimates of settlement can vary significantly due to the heterogenous nature and inherent variability of the site soils. The potential for settlements larger than those estimated above exists, and the effect of these settlements on the structure or roadway should be carefully considered.

The potential differential settlement is difficult to assess. Wherever there are large anticipated settlements, significant differential settlements are also possible. Generally, the thick embankment fills will help distribute the settlement over a larger area, reducing the abruptness of any differential settlement.

Differential longitudinal settlements at the abutment can be managed through design of an approach slab. Differential settlements will also occur where the EPS transitions to soil fill; this transition should be accomplished using a 2H:1V taper to reduce the abruptness of any changes in settlement.

Differential settlements may also become apparent transversely across the approach slab. In our experience with other bridges that have approach embankments with relatively large settlements, sometimes this relative movement, combined with the stiffness of the slab causes a corner of the slab to lose contact with the underlying soil and results in movement of the slab with each passing vehicle. If this condition occurs, the void under the approach slab should be repaired, typically by grouting the void under the slab and repaving a portion of the approach. It would be prudent to place fills as early in the schedule as possible to allow a larger portion of the settlement to occur prior to completion of the project. It would also be prudent to budget for additional fill volumes to compensate for the anticipated settlement.

5.5.3 Construction

Due to the importance of the embankment fill on the slope stability, and to limit the size of the counterforce berms and EPS volumes, approach fills should consist of well-graded rock fill with a maximum particle size of 450 mm and a fines content less than 12 percent. Granular "C" as specified by the Government of Newfoundland and Labrador's Department of Transportation and Works Specifications Book is an example of a suitable material. Material derived from the cuts in the bedrock would be suitable for use as approach fill provided it meets the requirements above.

All approach fill should be compacted to at least 80 percent relative density. To ensure compaction through the entire depth of the lift, fill should be placed in lifts compatible with the compaction equipment used.

Armour stone should be placed in areas where fills will be subjected to flowing water from the brook. This armour stone should be designed to withstand the velocities anticipated in the brook during high-flow periods.

5.5.4 Monitoring

Monitoring of the fill elevations should be carried out during and after construction to determine the rate of settlement. Geotechnical instrumentation should be installed and monitored to assess the stability of the embankment at various stages throughout construction. This instrumentation should include settlement plates, vibrating wire piezometers, and slope inclinometers.

- Settlement plates and survey points will allow monitoring of the rate of settlement of the fill throughout construction. This monitoring can be completed with GPS or conventional survey equipment (e.g. automatic level or total station) provided that a stable reference point (i.e. outside of the influence of the embankment or any other movement) is available or installed.
- Vibrating wire piezometers are used to provide accurate and reliable readings of water pressures. Vibrating wire piezometers are typically installed by grouting them into a borehole. When installed in a soil, they can be used to measure the porewater pressure which will help provide an understanding of the soil behaviour. Vibrating wire piezometers contain a high-tensile steel wire with a fixed anchor at one end and are attached to a diaphragm in contact with water pressure at the other end. The wire is electronically plucked and its response can be correlated to the water pressure on the diaphragm.
- Slope inclinometers are used to measure subsurface movements and deformations. They allow detection of zones of movement, quantification of movement, and an assessment of whether the rate of movement is constant, slowing, or accelerating. An inclinometer system has two components: an inclinometer casing and an inclinometer measurement system. Inclinometer casing is usually installed in a borehole and a portable inclinometer measurement system is used to assess how the profile of the casing changes with time.

5.6 FOUNDATIONS

5.6.1 General

The design depth of frost penetration should be taken as 1.8 m. The bottom of footings in frost susceptible soils should be located below this depth to prevent heave under frost action. Where this depth is not maintained, an equivalent combination of soil and insulation, or other measures such as excavation and replacement with non-frost susceptible soil, may be used to protect the structure from frost.

5.6.2 Driven Pile Foundations

Steel H-piles driven to practical refusal in bedrock are a practical option to support the bridge abutments. The factored compressive axial resistances of several H-pile sections are provided in Table 12; we would be pleased to review other sections upon your request. In accordance with the Canadian Highway Bridge Design Code (CAN/CSA S6-14, 2014) Clause 6.9.1 this includes a resistance factor of 0.4.

Table 12 Factored Axial Resistance at ULS for Driven Piles

Pile Type	Factored Axial Resistance (Compression)
HP 360 x 174	1600 kN
HP 360 x 152	1350 kN
HP 360 x 132	1200 kN

The compressive resistance will be achieved through a combination of end-bearing and shaft friction. To achieve this resistance, the piles should penetrate the overburden and may also penetrate into bedrock. Precise estimates of pile penetration into bedrock are not possible, however, based on our experience the piles may penetrate approximately 1 to 2 m into the quartzose sandstone encountered near the proposed east abutment and 2 to 4 m into the siltstone encountered near the proposed west abutment.

The bedrock surface changes drastically over relatively small distances on the west side of the river: north of the river, bedrock outcrops are apparent near elevation +3.0 m; near the abutments of the existing bridge, bedrock as encountered at elevations of -4.1 and -7.3 m. In the borehole nearest the proposed abutment (borehole BH11) bedrock was encountered at an elevation of -24.9 m. Therefore, the depths of refusal for piles on bedrock may vary significantly from those encountered in the boreholes advanced on the west side of the river. Furthermore, some piles may meet refusal in the cemented silt, and if so, the dynamic pile monitoring should be used to verify that the required resistances have been obtained.

The resistance of pile groups may be calculated as the sum of the individual pile capacities provided that the centre-to-centre spacing of the piles is at least three pile diameters. The expected settlement of piles driven to refusal on or in bedrock at the serviceability limit state (SLS) loads is negligible.

Piles should be driven with a hammer having a minimum rated energy of 450 Joules/cm^2 of steel cross-sectional area. Practical refusal in bedrock should be taken as a pile penetration of less than 25 mm for 20 blows at the rated energy for four consecutive 25-mm increments. The contractor should provide full details on the method of installation and equipment to the geotechnical engineer prior to starting the work.

Dynamic pile monitoring (e.g. Pile Driving Analyzer System) should be carried out on the initial pile installations to verify that overstressing does not occur, that the hammer is operating within normal efficiencies, and that the estimated resistance provided for design is achieved at the set criteria. As a minimum, dynamic pile monitoring should be performed on 10 percent of the piles at end of initial drive and at the beginning of re-strike at each abutment. Full-time inspection by qualified geotechnical personnel is recommended during pile installation.

To further evaluate the potential for relaxation to occur following initial driving, all piles should be re-tapped a minimum of 24 hours after initial driving refusal. If relaxation occurs, all piles should be re-driven to the refusal criteria and the cycle repeated until the refusal criteria is maintained during subsequent re-taps. If significant relaxation continues to occur, dynamic pile monitoring could be used to determine if the required load capacity is being developed. In particular, piles driven into the siltstone bedrock have a high potential to relax and may require several cycles of re-taps before the driving criteria is maintained.

5.6.3 Down-Drag Loads

When piles are installed through soil subject to settlements (such as the clay encountered in this investigation upon new loading) the resulting downward movement of the soil around the piles, as well as in any soil above the settling layer, induces down-drag forces on the piles and any attached structures (e.g. the pile caps and abutments). The down-drag forces will only occur on the piles to the bottom of the compressible (i.e. clay) layer.

At the abutments, we anticipate settlement in the clay layer and drag loads due to negative skin friction on the piles. Drag loads increase the structural loads in the pile and thus have to be considered in structural design of the piles. In this assessment, it is important to note that drag load and transient live load do not combine and that separate loading cases must be considered:

- Permanent load plus drag load, but no transient live load; and
- Permanent load and transient live load, but no drag load.

The magnitude of down-drag loads may be calculated based on the vertical effective stress and the combined shaft resistance factor, β. Values of the total unit weight, submerged unit weight, and β for use in down-drag analyses are presented in Table 13.

Table 13 Combined Shaft Resistance Factor, β for use in Down-Drag Analyses

Material	Total Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Combined Shaft Resistance Factor, β
New Abutment Fills	22.0	12.0	0.55
Existing Fills	21.0	11.0	0.45
Sand and Gravel	20.5	10.5	0.40
Clay	18.5	8.5	0.35

5.6.4 Lateral Pile Behaviour

For consideration of lateral loads, the depth to fixity for three pile sections are provided in Table 14. The calculations assume that the depth of fixity is obtained within the approach fills at the west abutments and in the native clay at the east abutment.

Table 14 Depth to Fixity

Pile Type	Depth to Fixity (m)			
	Pile Fixity in Approach Fills (West Abutment)		Pile Fixity in Sand and Gravel and Clay (East Abutment)	
	Strong Axis (X-X)	Weak Axis (Y-Y)	Strong Axis (X-X)	Weak Axis (Y-Y)
HP 360 x 174	2.7	2.2	4.5	3.5
HP 360 x 152	2.6	2.1	4.3	3.3
HP 360 x 132	2.5	2.1	4.1	3.2

5.7 BACKFILL

Where backfilled with soil, the abutments for the new bridge and retaining walls should be backfilled with a non-frost susceptible, non-expansive, non-corrosive, free-draining, well-graded material such as Granular ‘C’. The extent of the granular backfill should be in accordance with the wall design requirements.

Alternatively, where EPS is used behind the abutments, the abutments should be designed to resist the force the EPS exerts on the wall. For the case of thermal movements towards the EPS this force may be estimated based on the elastic properties of the EPS and the magnitude of the thermal movement.

It is important that retaining walls are designed to ensure thorough drainage of the backfill material. This may be accomplished with a drainage system such as a longitudinal drain pipe discharging to a positive outlet. Backfill should be placed in lifts and compacted as a minimum to 95 percent of the standard Proctor maximum dry density. Where wall backfill acts as the road subgrade the compaction requirements for the approach fill will govern (i.e. the upper 1.5 m should be compacted to 100 percent of the standard Proctor maximum dry density.

Care should be taken not to damage walls when performing backfilling and compaction operations. To limit compaction-induced stresses, compaction within 1.5 m of retaining structures should be performed with a walk-behind vibratory plate tamper or other lightweight compaction equipment in lieu of a vibratory drum roller.

All drainage materials, including backfill and drainage blankets, must be designed to limit loss of soil according to filter criteria.

The values for the parameters presented in the next section may be used for design of retaining walls. The earth pressure coefficients used for design should be selected or adjusted based on the appropriate finished back-slope angle. Walls that can tolerate little or no movement should be designed for at-rest lateral earth pressures.

5.8 GEOTECHNICAL PARAMETERS FOR RETAINING WALL DESIGN

The following unfactored values for the indicated parameters may be used for retaining wall design (Table 15):

Table 15 Unfactored Geotechnical Parameters

Parameter	Value		
	Compacted Granular “C” (a)(b)	Compacted Quarried Rock Fill(a)	In-Situ Sand and Gravel Fill or Compacted Common Borrow
Effective Angle of Internal Friction, degrees	36	38	32
Effective Cohesion, kPa	0	0	-
Total Unit Weight, kN/m^3	22.0	22.0	21.0
Submerged Unit Weight(c), kN/m^3	12.2	12.2	11.2
Coefficient of Active Earth Pressure(d)	0.26	0.24	0.31
Coefficient of Passive Earth Pressure(d)	3.85	4.20	3.25

Parameter	Value		
	Compacted Granular "C" (a)(b)	Compacted Quarried Rock Fill(a)	In-Situ Sand and Gravel Fill or Compacted Common Borrow
Coefficient of At-Rest Earth Pressure(d)	0.41	0.38	0.47
Friction Factor, Soil/Concrete Interface(e)	0.50	0.50	0.45

(a) Material shall be placed in lifts and suitably compacted as described above.
(b) As per Government of Newfoundland and Labrador Department of Transportation and Works Specifications Book (2011).
(c) For uplift design the groundwater table should be assumed at the ground surface and submerged unit weights should be used.
(d) Coefficients of earth pressure presented in table assume a frictionless wall with a vertical back face and a horizontal back slope.
(e) For mass concrete or masonry, lower values will be required for formed or pre-cast concrete.

5.9 PAVEMENT STRUCTURE

Based on the existing soil conditions, proposed approach fills, and expected traffic loadings, the following (minimum) pavement structure is recommended where asphalt is placed over the existing soils or embankment fill (Table 16):

Table 16 Pavement Structure (Above Native Soils and Embankment Fill)

Materials	Pavement Structure
Asphalt Top	50 mm
Asphalt Base	60 mm
Granular "A"	150 mm
Granular "B"	450 mm

The pavement design is based on the subgrade soils being in a stable condition at the time the granular materials are placed. The subgrade soils may become soft and constructability can be a problem. Where the subgrade is comprised of clay or where the presence of clay below the subgrade is thought to be influencing the subgrade performance, the subgrade should be over-excavated by 600 mm and should be reinstated using engineered rock fill or structural fill (i.e. a subgrade capping layer).

The physical properties and placing of the asphaltic courses, granular 'A', and granular 'B' should be in accordance with the most recent version of Newfoundland and Labrador Department of Transportation and Works Specifications Book.

Where the pavement structure is above EPS geofoam (EPS29), the following pavement structure is recommended:

Table 17 Pavement Structure (Above EPS29)

Materials	Pavement Structure
Asphalt Top	50 mm
Asphalt Base	120 mm
Granular "A"	300 mm
Granular "B"	730 mm

To limit damage to the EPS when installing the granular 'B', the first lift above the EPS should have a thickness of 500 mm. As a minimum, the top 1.0 m of the EPS subgrade should be EPS29 or denser.

5.10 WINTER WEATHER CONDITIONS

Where practical, earthwork during freezing temperatures should be avoided. In the event of winter construction, special measures will be required to ensure that fills and foundations are not placed on frozen ground and that the soils are protected from freezing after placement. Even following careful procedures and precautions experience has shown that earthworks in these types of soils often become impractical at temperatures below approximately -5°C.

5.11 SEISMIC SITE CLASSIFICATION

Based on the findings at the test locations, the site classification for seismic site response in accordance with Clause 4.4.3.2 of the Canadian Highway Bridge Design Code (CAN/CSA-S6-14, 2014) is Seismic Site Class E (soft soil).

6.0 CLOSURE

This report has been prepared to assist in the design and construction of the proposed Dick's Brook Bridge. This report has been prepared for the sole benefit of Harbourside Engineering Consultants and their agents. Any use which a third party makes of this report is the responsibility of such third party.

The recommendations made in this report are in accordance with our present understanding of your project. If any details are included in the final design of the proposed structure that differ from the assumptions outlined in this report, the geotechnical engineer should be consulted.

This report is based on the site conditions encountered by Harbourside Geotechnical Consultants at the time of the work at the specific sampling locations, and can only be extrapolated to a limited extent around these locations. Should any conditions differ from those detailed on the borehole records, the engineer should be notified to allow reassessment of any design assumptions.

If you have any questions or require any additional information, please do not hesitate to contact the undersigned at your convenience.

H a r b o u r s i d e
Geotechnical Consultants

Kind Regards,

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APPENDIX A

Symbols and Terms Used on Borehole and Test Pit Records

Borehole Records BH01 to BH06 and BH10 to BH14

Test Pit Records TP01 to TP04

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:

USCS SOIL CLASSIFICATION SYMBOLS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN 75 µm SIEVE SIZE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON 4.75 mm SIEVE	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES. LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL – SAND – SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL – SAND – CLAY MIXTURES
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING THE 4.75 mm SIEVE	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		SM	SILTY SANDS, SAND – SILT MIXTURES
				SC	CLAYEY SANDS, SAND – CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN 75 µm SIEZE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

OTHER COMMONLY USED SYMBOLS

GLACIAL TILL		UNSTRATIFIED GLACIAL DEPOSIT RANGING FROM CLAY TO BOULDERS
BEDROCK		IGNEOUS BEDROCK
		METAMORPHIC BEDROCK
		SEDIMENTARY BEDROCK
MATERIALS PLACED BY HUMANS		FILL: SUBSURFACE MATERIALS IDENTIFIED AS PLACED BY HUMANS
		ASPHALT
		CONCRETE

SAMPLE TYPE

SS	Split Spoon (obtained by performing SPT)
ST	Shelby Tube (Thin-Walled Tube)
BS	Bulk Sample
PS	Piston Sample
WS	Wash Sample
HQ, NQ, AQ, BQ, etc.	Rock Core Samples Obtained Using Standard Size Diamond Bits

SPT N-VALUE (N-INDEX)

The standard penetration test (SPT) provides a qualitative evaluation of compactness and a qualitative comparison of subsoil stratification. The SPT is performed in in the bottom of a borehole where a split-barrel sampler having an outside diameter of 50.8 mm is impacted using a hammer weighing 623 N falling 0.76 m for each hammer blow. The SPT N-value is the blow count representation of the penetration resistance of the soil. In accordance with ASTM D1586, the N-value, reported in blows per 300 mm, equals the sum of the number of blows (N) required to drive the sampler over the depth interval of 150 to 450 mm. However, when a 600 mm sampler is used the number of blows (N) required to drive the sampler over the interval of 300 to 600 mm may be reported if this value is lower. For samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in mm (e.g. 50/120). Although some methods make use of N-values corrected for various factors (for equipment used, overburden stress, length of drill rod, etc.) no corrections have been applied to the N-values presented on the logs.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests (DCPT) are performed using a standard 60-degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the SPT test. The DCPT value is the number of blows of the hammer required to drive the cone 300 mm. The DCPT provides a qualitative evaluation of compactness and allows for a qualitative comparison of subsurface stratification.

RECOVERY

For soil samples, recovery is recorded as the total length of the soil sample recovered. For rock core, recovery is expressed as a percentage of the total length drilled on a per run basis.

OTHER TESTS

S	Sieve Analysis	CD	Consolidated-Drained Triaxial	C	Consolidation
H	Hydrometer Analysis	CU	Consolidated-Undrained Triaxial	Q_u	Unconfined Compression
γ	Unit Weight	UU	Unconsolidated Undrained Triaxial	I_p	Point Load Index, Ip(50)
G_s	Specific Gravity of Soil Particles	DS	Direct Shear	k	Laboratory Permeability

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	Vegetation, roots, and moss with organic matter and topsoil typically forming a mattress at the ground surface.
Topsoil	Mixture of soil and humus capable of supporting vegetative growth.
Peat	A soil composed of vegetable tissue in various stages of decomposition usually with an organic odor, a dark-brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous.
Till	Non-stratified glacial deposit which may range from clay to boulders
Fill	Artificial (man-made) deposits transported and placed on the natural surface of soil or rock.

Terminology describing soil structure:

Homogeneous	The lack of visible bedding and the same appearance and colour throughout
Desiccated	Having visible signs of weathering by oxidation of clay minerals, shrinking cracks, etc.
Fissured	Having cracks and hence a blocky structure
Stratified	Composed of regular alternating successions of different soil types
Varved	Comprised of regular alternating successions of silt and clay which were transported into freshwater lakes by melt water
Layer	> 75 mm
Seam	2 mm to 75 mm
Parting	< 2 mm
Pocket	Small erratic deposit, usually less than 300 mm
Lens	Lenticular deposit

Terminology describing soil types:

Soils are described in accordance with the Unified Soil Classification System (USCS) as described in ASTM D2487 and ASTM D2488. This system classifies soil into categories representing the results of laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index. Using this system, soils are assigned a group name (e.g. silty sand) and symbol (e.g. SM). The various groupings of this classification system have been devised to correlate in a general way with the engineering behavior of soils. Laboratory tests are performed on the portion of the sample passing the 75 mm sieve.

When laboratory test results indicate that that the soil is close to another classification group, the borderline condition can be indicated with two symbols separated by a slash (e.g. CL/CH).

Terminology describing cobbles, boulders, and non-matrix materials:

Materials outside of the USCS (e.g. particles larger than 75 mm, organic matter, construction debris) are described based on the proportion of these materials by weight using the following terminology:

Trace, or occasional	< 10%
Some	10% to 20%
Frequent	> 20%

Terminology describing the compactness condition of cohesionless soils:

A qualitative term describing the compactness condition of a cohesionless soil is interpreted from the SPT N-value (also known as the N-index). The relationship between the SPT N-value and the compactness condition is shown in the following table.

Compactness Condition	**SPT N-Value (blows per 0.3 m)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Over 50

Terminology describing the compactness condition of cohesive soils:

Cohesive soils can be classified in relation to undrained strength. Undrained strength can be determined by a number of tests including: unconfined compression tests, field and laboratory vane tests, laboratory fall-cone tests, shear-box tests, and triaxial tests. The consistency and undrained shear strength may also be approximately related the SPT N-Value. The relationship between the consistency and the undrained shear strength, as well as a rough correlation with SPT N-Value as shown in the following table.

Consistency	Undrained Shear Strength (kPa)	SPT N-Value (blows per 0.3 m)
Very Soft	< 12	< 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

ROCK DESCRIPTION

Rock is a natural aggregate of minerals that cannot be readily broken by hand and that will not disintegrate on a first wetting and drying cycle. A rockmass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, shears and faults.

Terminology Describing Geological Classification of Rock:

Rock is classified with respect to its geological origin or lithology as follows:

Igneous Rocks	Rocks such as granite, diorite, and basalt, which are formed by the solidification of molten material.
Sedimentary Rocks	Rocks such as sandstone, limestone and shale, which are formed by the lithification of sedimentary soils.
Metamorphic Rocks	Rocks such as quartzite, schist, and gneiss, which have been altered by the application of intense heat and/or pressure.

Terminology Describing the Strength of Intact Rock:

Strength is the maximum stress level that can be carried by a specimen. Rocks may be classified based on their intact strength as shown in the following table.

Term	Unconfined Compressive Strength (MPa)
Extremely Weak	0.25 to 1
Very Weak	1 to 5
Weak	5 to 25
Medium Strong	25 to 50
Strong	50 to 100
Very Strong	100 to 250
Extremely Strong	> 250

Terminology Describing Discontinuity Spacing

The structural integrity of a rockmass will be affected by the presence of discontinuities. The spacing of discontinuities can vary from extremely wide to extremely close as indicated in the table below.

Term	Spacing Width (m)
Extremely Close	< 0.02
Very Close	0.02 to 0.06
Close	0.06 to 0.20
Moderately Close	0.20 to 0.6
Wide	0.6 to 2.0
Very Wide	2.0 to 6.0
Extremely Wide	> 6.0

Rock Quality Designation (RQD)

RQD is an indirect measure of the number of fractures within a rockmass. The method provides a quick and objective technique to estimate rockmass quality during diamond drill core logging. All pieces of intact and sound rock greater than 100 mm long are summed and divided by the total length of the core run in accordance with ASTM D6032.

RQD Classification	RQD (%)
Very Poor Quality	0 to 25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

Terminology to Describe Rock Weathering

The state of weathering significantly alters the geotechnical behaviour of rocks and rockmasses. Weathering of the rockmass may be classified as shown in the following table.

Term	Description
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.
Slightly Weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones
Highly Weathered	More than a half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.

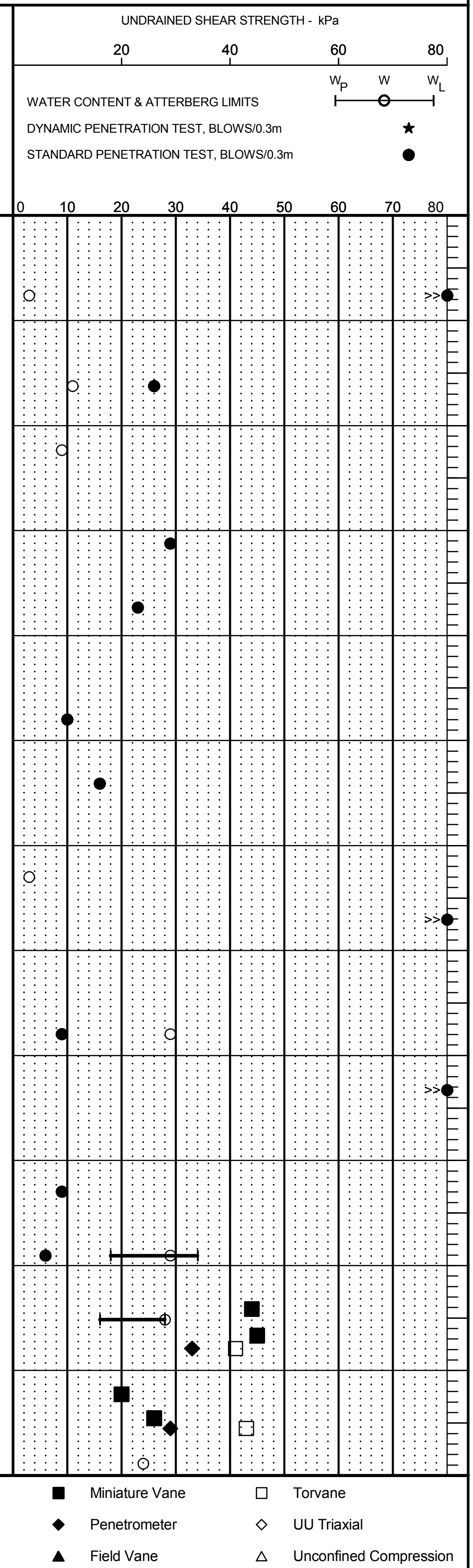
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH01

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-06 TO 2016-12-07 **WATER LEVEL:** N/A
BH SIZE: HW

ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
14.98						
14.68	CONCRETE					
	FILL: light brown silty gravel with sand	SS	1	450	42-46-40-41 (81)	S
13.76	FILL: brown to grey gravel to gravel with silt and sand - with frequent cobbles and occasional boulders	SS	2	125	8-12-14-14 (26)	
		SS	3	75	76-mm Spoon	
		SS	4	50	16-8-21-23 (29)	
		SS	5	125	5-13-12-11 (23)	
		SS	6	0	6-6-6-4 (10)	
		SS	7	0	12-14-7-9 (16)	
		SS	8	150	76-mm Spoon	S
		SS	9	50	20-50 / 50 mm	
	- 330 mm boulder at 7.3 m depth	SS	10	0	3-6-4-5 (9)	
	- with purple quartzose sandstone fragments (rock fill) below 8.0 m depth	SS	11	25	6-6-18-50 / 0 mm	
5.68	Firm to stiff brown lean CLAY	SS	12	250	7-5-4-5 (9)	
		SS	13	400	3-3-3-4 (6)	
		ST	14	600	Push	S
		ST	15	475	Push	
		ST	16	300	Push	



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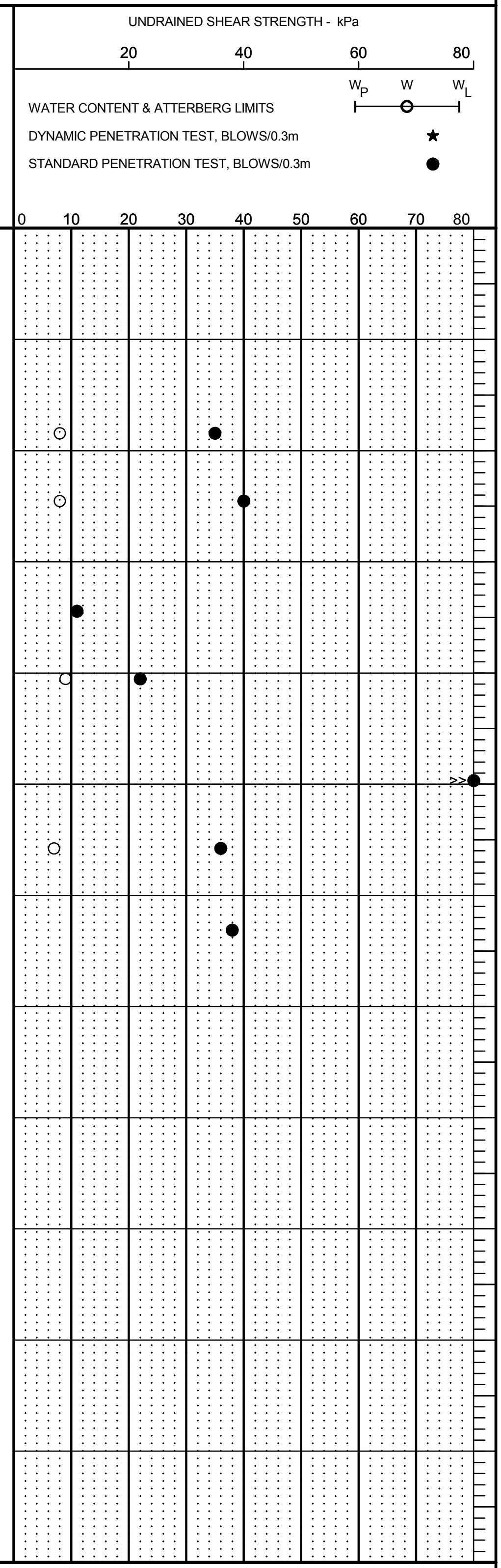
BOREHOLE RECORD

BH01

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2016-12-06 TO 2016-12-07 **WATER LEVEL:** N/A
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	2.64	Firm to stiff brown lean CLAY *(continued)*	ST	17	75	Push	
13		Compact to dense grey silty SAND with gravel to silty GRAVEL with sand (diamicton)					
14			SS	18	125	12-19-21-14 (35)	
			SS	19	300	13-54-21-19 (40)	S
15		- with 350 mm boulder at 14.9 m depth	SS	20	150	9-6-5-7 (11)	
16			SS	21	175	10-11-11-12 (22)	
17			SS	22	0	30-39-57-68 (96)	
			SS	23	250	17-17-19-26 (36)	
18		- brown below 17.9 m depth	SS	24	300	11-17-21-35 (38)	
19	-4.12	Very poor to poor quality, purple QUARTZOSE SANDSTONE	HQ	25	46%	0%	
20		- medium strong to strong - slightly to moderately weathered - staining on fractures	HQ	26	100%	0%	
21		- slighly weathered below 21.0 m	HQ	27	100%	0%	
			HQ	28	82%	45%	
	-6.81	End of borehole					
22							
23							



Miniature Vane
Torvane
Penetrometer
UU Triaxial
Field Vane
Unconfined Compression

BH02

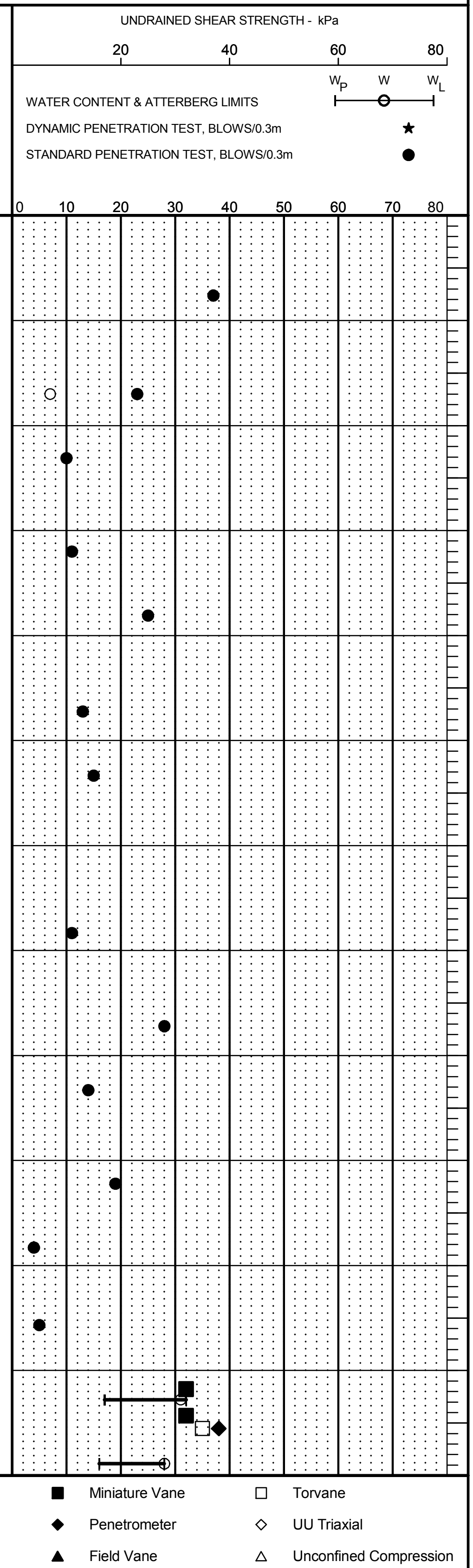
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2016-12-09 TO 2016-12-11 **WATER LEVEL:** 2016-12-13 *

PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	14.99						
	14.74	CONCRETE					
		FILL: light brown silty gravel with sand	SS	1	350	14-16-21-24 (37)	
1	13.77	FILL: brown gravel to gravel with silt and sand	SS	2	125	15-12-11-13 (23)	
2			SS	3	75	4-5-5-5 (10)	
3			SS	4	50	5-5-6-10 (11)	
			SS	5	25	8-12-13-15 (25)	
4			SS	6	0	7-6-7-8 (13)	
5			SS	7	0	7-8-7-9 (15)	
6		- with frequent quartzose sandstone cobbles (rock fill) below 5.8 m depth	SS	8	0	76-mm Spoon	
			SS	9	50	7-6-5-8 (11)	
7			SS	10	50	6-8-21-7 (28)	
8			SS	11	50	8-8-7-7 (14)	
9	5.54		SS	12	50	6-8-11-8 (19)	
	5.16	Very loose black silty sand (ROOTMAT and TOPSOIL) - with frequent organic material and occasional wood	SS	13	100	3-2-2-4 (4)	
10		Firm to stiff brown lean CLAY	SS	14	0	8-6-3-2 (5)	
11			ST	15	300	Push	
			SS	16	600	Push	



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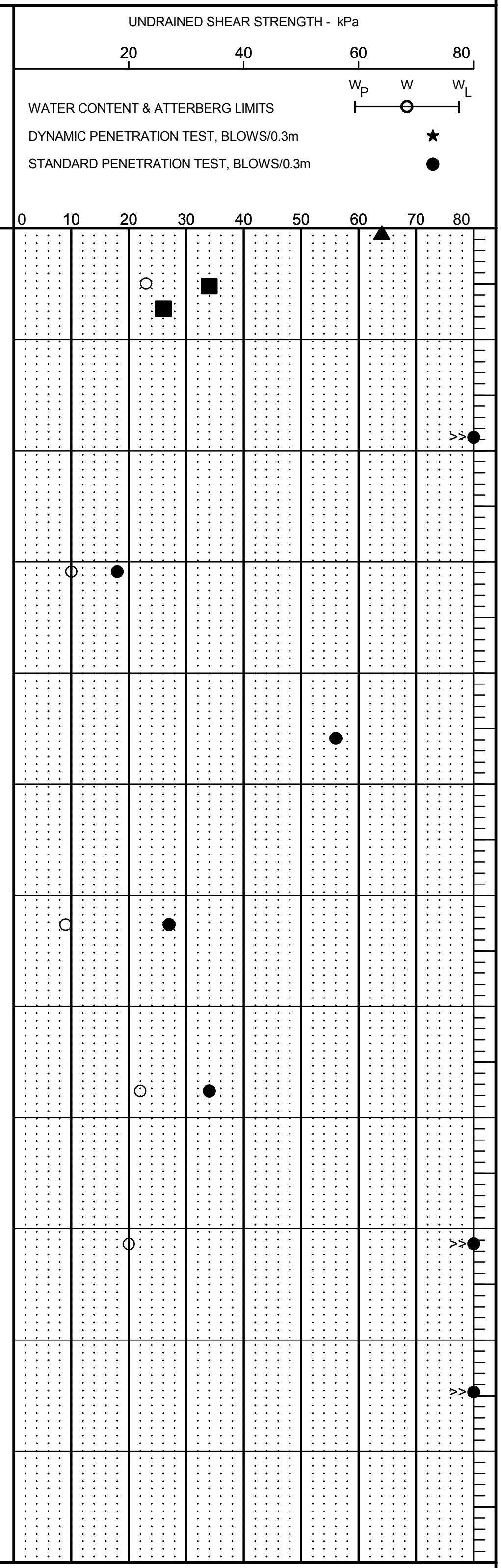
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH02

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2016-12-09 TO 2016-12-11 **WATER LEVEL:** 2016-12-13 *
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Firm to stiff brown lean CLAY *(continued)*	ST	17	600	Push	
13	1.35						
14		Compact to very dense grey silty GRAVEL with sand to silty SAND with gravel (diamicton) - with frequent cobbles and boulders	SS	18	50	21-38-65 / 125 mm	
15			SS	19	175	10-7-11-14 (18)	S
16			SS	20	75	22-24-32-49 (56)	
17		- 430 mm boulder at 17.1 m depth - 330 mm boulder at 17.7 m depth					
18	-3.91		SS	21	150	9-22-14-13 (27)	
19		Dense to very dense brown sandy SILT to silty SAND - partially cemented to cemented					
20			SS	22	375	7-17-17-27 (34)	S
21			SS	23	300	14-31-50 / 50 mm	
22	-7.26						
			SS	24	0	50 / 25 mm	
23			HQ	25	97%	10%	



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

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BOREHOLE RECORD

BH02

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS **PROJECT No.:** 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL **DATUM:** CGVD28
DATES: BORING 2016-12-09 TO 2016-12-11 **WATER LEVEL:** 2016-12-13 * **BH SIZE:** HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	-9.62	Very poor quality purple QUARTZOSE SANDSTONE interbedded with very poor quality grey SHALE and very poor quality reddish-brown SILTSTONE - moderately to highly weathered - shale and siltstone beds very weak - sandstone beds medium strong to strong *(continued)*			HQ	26	83%	0%	
					HQ	27	92%	0%	
					HQ	28	89%	0%	
					HQ	29	86%	0%	
					HQ	30	97%	0%	
27	-12.09	Very poor quality purple QUARTZOSE SANDSTONE - moderately weathered - medium strong to strong - silt seam at 25.8m depth - slightly weathered, strong, with near-vertical silt seam below 25.8 m depth			HQ	31	93%	0%	
		End of borehole - 25-mm PVC standpipe installed *standpipe blocked at 9.78m							

Depth scale: 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

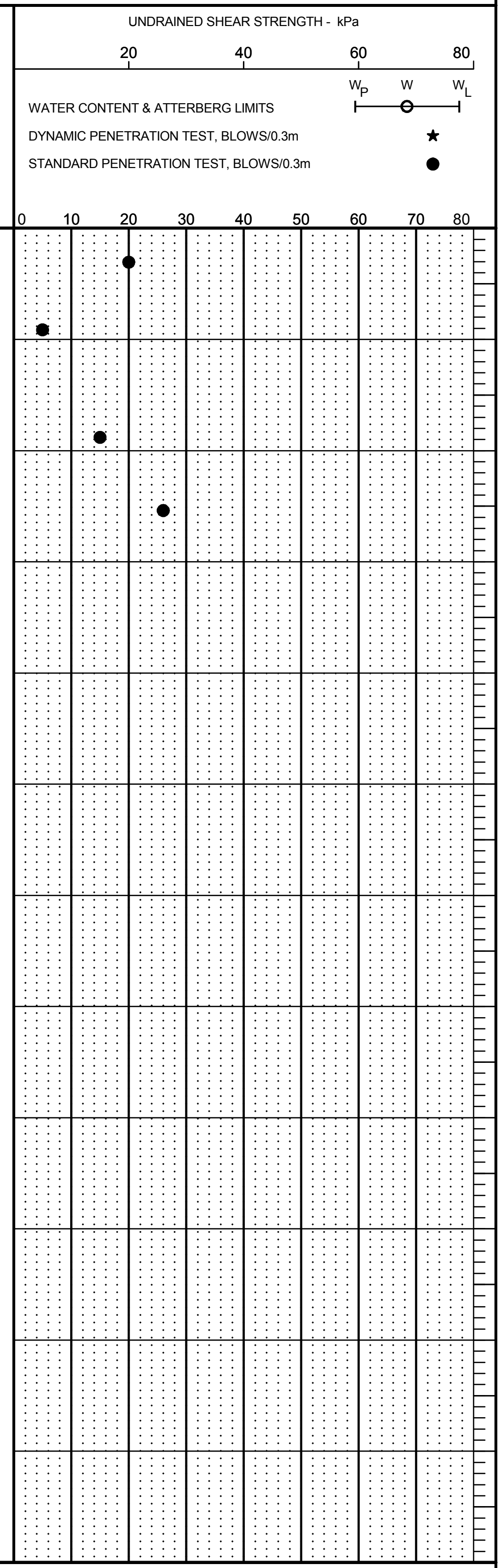
BOREHOLE RECORD

BH03

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-13 **WATER LEVEL:** N/A
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	6.93	FILL: brown silty sand with gravel to clayey gravel with sand - with occasional cobbles			SS	1	125	14-7-13-32 (20)	
					SS	2	75	9-2-3-21 (5)	
					SS	3	25	12-6-10-5 (15)	
	4.39	Dense grey silty GRAVEL with sand (diamicton)			SS	4	150	27-10-16-37 (26)	
	4.09	Very poor quality purple and grey QUARTZOSE SANDSTONE - slightly weathered - strong to very strong - staining on fractures			HQ	6	100%	17%	
					HQ	7	100%	50%	
					HQ	8	100%	20%	Qu
	2.03	End of borehole							



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

BOREHOLE RECORD

BH04

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-12
WATER LEVEL: N/A
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	6.61								
		FILL: brown gravel with silt and sand - with wood at 1.4 m depth			SS	1	100	4-6-6-5 (11)	
					SS	2	75	28-18-16-15 (31)	
	5.24				SS	3	50	21-50 / 0 mm	
		Fair to good quality grey to purple QUARTZOSE SANDSTONE - slightly weathered - strong to very strong - staining on fractures			HQ	4	100%	56%	
					HQ	5	100%	81%	Qu
	3.10	End of borehole							

UNDRAINED SHEAR STRENGTH - kPa: 20 40 60 80

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

>>

Depth scale: 1 2 3 4 5 6 7 8 9 10 11

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

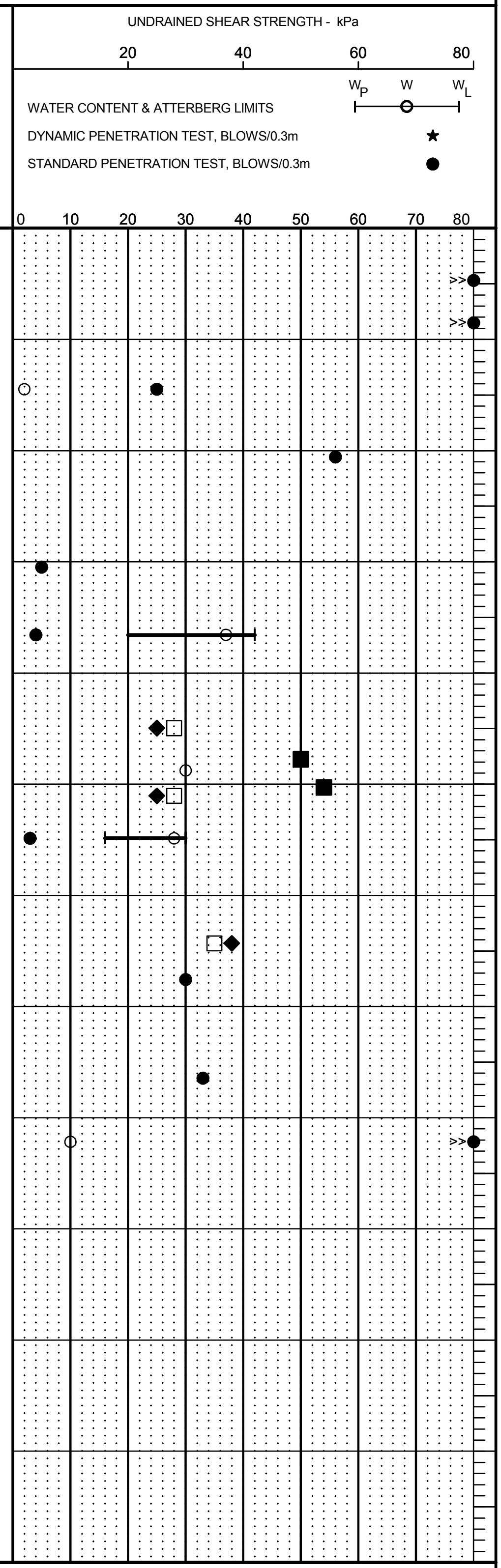
BOREHOLE RECORD

BH05

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2016-12-07 WATER LEVEL: 2016-12-13
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: NW

ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
15.39						
15.22	ASPHALT					
14.93	FILL: grey gravel with silt and sand	SS	1	250	19-21-50 / 25 mm	
		SS	2	100	52-50 / 25 mm	
	FILL: brown silty gravel with sand	SS	3	200	13-19-15-10 (25)	
		SS	4	75	11-19-37-34 (56)	
12.65	Soft to firm brown lean CLAY	SS	5	0	3-3-3-2 (5)	
		SS	6	500	2-2-2-2 (4)	
		ST	7	600	Push	C
		ST	8	600	Push	
		SS	9	600	0-0-3-3 (3)	
		ST	10	600	Push	C
8.63	Compact to dense grey silty GRAVEL with sand (diamicton)	SS	11	375	14-33-13-17 (30)	
		SS	12	75	6-14-19-14 (33)	
		SS	13	150	10-13-11-50 / 75 mm	
6.91	Fair to good quality grey QUARTZOSE SANDSTONE - slightly weathered - strong to very strong	NQ	14	100%	63%	
		NQ	15	100%	70%	Qu
		NQ	16	100%	64%	
	- orange staining on fractures below 10.3 m depth	NQ	17	100%	83%	



Miniature Vane | Torvane | Penetrometer | UU Triaxial | Field Vane | Unconfined Compression

(Continued Next Page)

BH05

HARBOURSIDE
Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-07 **WATER LEVEL:** 2016-12-13
BH SIZE: NW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Fair to good quality grey QUARTZOSE SANDSTONE - slightly weathered - strong to very strong *(continued)* - calcite seam at 12.8 m depth			NQ	18	93%	80%	Qu
13					NQ	19	100%	0%	
14					NQ	20	100%	77%	
	0.74	End of borehole - 25-mm PVC standpipe installed							
15									
16									
17									
18									
19									
20									
21									
22									
23									

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane

◆ Penetrometer ◇ UU Triaxial

▲ Field Vane △ Unconfined Compression

BH06

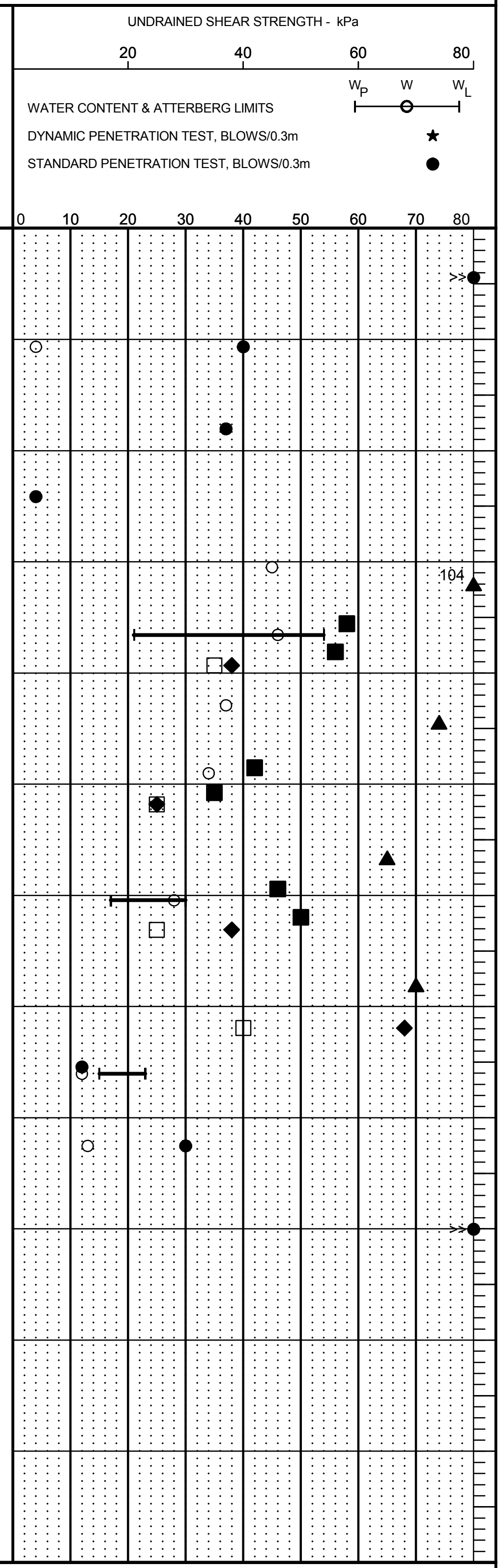
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-08 **WATER LEVEL:** 2016-12-13 *
BH SIZE: HW

ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
15.37						
15.17	ASPHALT					
	FILL: brown sand with silt and gravel - with occasional cobbles	SS	1	125	39-51-50 / 75 mm	
		SS	2	300	15-16-24-25 (40)	S
		SS	3	50	9-27-21-16 (37)	
13.16	Stiff brown fat CLAY	SS	4	0	12-2-2-4 (4)	
		SS	5	425	Push	
		ST	6	600	Push	
11.41	Firm to stiff brown lean CLAY	SS	7	600	Push	
		ST	8	600	Push	
		ST	9	600	Push	
		ST	10	275	Push	
8.13	Compact to very dense brown clayey SAND with gravel (diamicton)	SS	11	450	10-11-7-5 (12)	S
		SS	12	175	14-17-13-23 (30)	
6.28		SS	13	75	6-4-50 / 125 mm	
6.15	Compact grey silty GRAVEL with sand (diamicton)					
	Fair quality grey QUARTZOSE SANDSTONE - slightly weathered - strong	HQ	14	100%	50%	
		HQ	15	93%	69%	Qu
3.46						

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11



Miniature Vane | Torvane | Penetrometer | UU Triaxial | Field Vane | Unconfined Compression

(Continued Next Page)

HARBOURSIDE
Geotechnical Consultants

BOREHOLE RECORD

BH06

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2016-12-08 **WATER LEVEL:** 2016-12-13 *
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		End of borehole - 25-mm PVC standpipe installed *standpipe blocked at 4.42m depth							
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									

UNDRAINED SHEAR STRENGTH - kPa
20 40 60 80

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

BH10

BOREHOLE RECORD

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2016-12-09 **WATER LEVEL:** 2016-12-13 *
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)
0	16.39	FILL: brown gravel with silt and sand	SS	1	500	20-24-13-9 (22)
0.91	15.48	Soft to firm brown fat CLAY	SS	2	200	4-3-2-2 (4)
			SS	3	500	1-1-2-2 (3)
1.96	14.43	Soft to firm brown lean CLAY	SS	4	125	3-2-2-2 (4)
			SS	5	600	0-0-2-2 (2)
			SS	6	450	2-2-2-2 (4)
			SS	7	450	0-0-1-1 (1)
			SS	8	600	0-1-3-3 (4)
	10.09	Dense grey silty GRAVEL with sand	SS	9	500	0-2-6-50 / 75 mm
	9.94	End of borehole - 25-mm PVC standpipe installed *standpipe blocked at 1.37m depth				

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

Scale: 0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

BH11

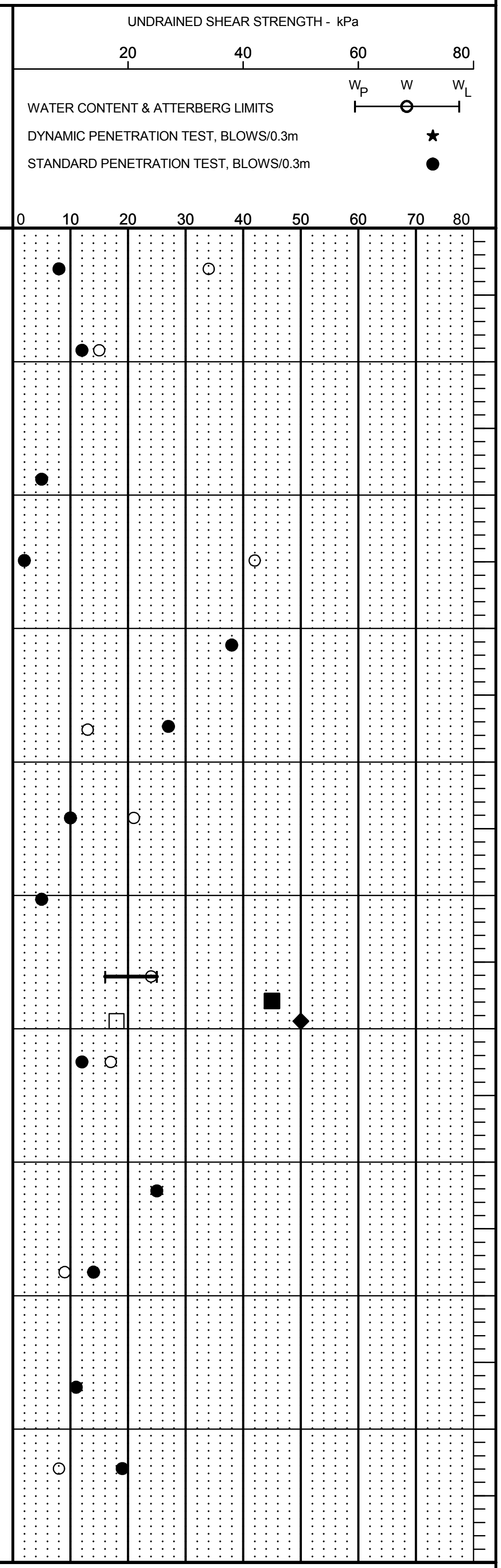
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2017-05-23 TO 2017-05-24 **WATER LEVEL:** 2017-05-28
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

ELEVATION (m)	SOIL/BEDROCK DESCRIPTION
4.28	
4.13	ROOTMAT/TOPSOIL
	Loose to compact brown to black silty SAND with gravel - with trace organics
1.44	Compact to dense grey GRAVEL with silt and sand
-0.44	Soft to firm brown lean CLAY to sandy lean CLAY with gravel
-2.04	Compact grey silty GRAVEL with sand (diamicton)
-3.85	Compact grey silty SAND with gravel (diamicton)
-5.32	

TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
SS	1	200	1-4-4-4 (8)	
SS	2	275	11-12-6-6 (12)	
SS	3	200	4-4-1-10 (5)	
SS	4	200	4-1-1-2 (2)	S
SS	5	50	11-14-24-27 (38)	
SS	6	225	16-10-17-30 (27)	
SS	7	25	12-10-6-4 (10)	
SS	8	25	6-3-3-2 (5)	
ST	9	600	Push	S
SS	10	125	4-6-6-7 (12)	
SS	11	175	10-12-13-12 (25)	
SS	12	275	15-11-6-8 (14)	
SS	13	150	20-6-5-10 (11)	
SS	14	475	7-8-11-10 (19)	S



Miniature Vane
Torvane
Penetrometer
UU Triaxial
Field Vane
Unconfined Compression

(Continued Next Page)

HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH11

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS **PROJECT No.:** 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL **DATUM:** CGVD28
DATES: BORING 2017-05-23 TO 2017-05-24 **WATER LEVEL:** 2017-05-28 **BH SIZE:** HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Compact to very dense light brown sandy SILT to SILT with sand - with occasional gravel - partially cemented to cemented - with occasional grey seams *(continued)*			SS	15	350	7-11-8-10 (18)	
11					SS	16	300	10-13-16-23 (29)	
					SS	17	300	9-10-11-16 (21)	
12					SS	18	450	14-18-17-19 (35)	
13					SS	19	300	8-13-14-14 (27)	
14		- non-plastic fines			SS	20	350	15-20-18-27 (38)	
15					SS	21	425	11-13-16-12 (28)	
16		- non-plastic fines			SS	22	400	9-21-37-35 (58)	S
17									
18					SS	23	525	18-28-27-36 (55)	
19					SS	24	500	12-20-21-29 (41)	

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

(Continued Next Page)

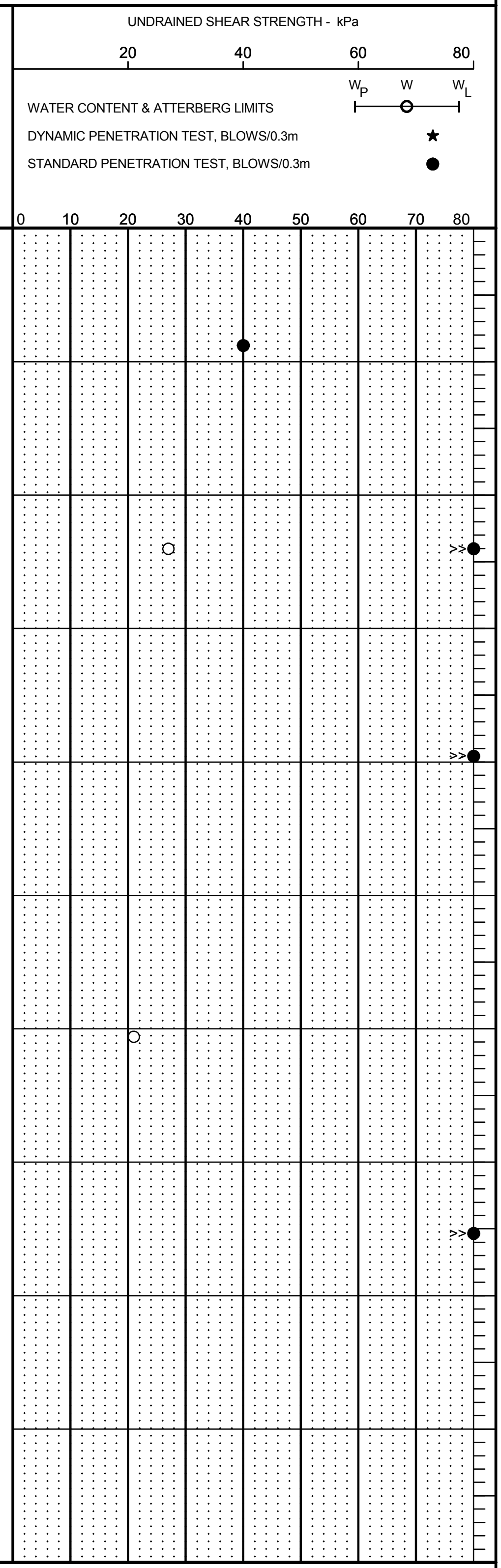
HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH11

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-23 TO 2017-05-24 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Compact to very dense light brown sandy SILT to SILT with sand - with occasional gravel - partially cemented to cemented - with occasional grey seams *(continued)*							
21					SS	25	600	9-14-26-70 (40)	
22					SS	26	500	23-39-58-50 (97)	
24					SS	27	550	22-36-55-75 (91)	
26					SS	28	275	48	
27					SS	29	250	40-50 / 100 mm	
28		- cemented below 27.7m depth			GB	30	375	N/A	
29	-24.90	Poor quality grey SILTSTONE - very weak - occasional silt seams			HQ	31	100%	38%	



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

(Continued Next Page)

BH11

BOREHOLE RECORD

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-23 TO 2017-05-24 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
31		Poor quality grey SILTSTONE - very weak - occasional silt seams *(continued)*			HQ	32	59%	27%	
32	-27.62	End of borehole - 25-mm standpipe installed							
33									
34									
35									
36									
37									
38									
39									

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane

◆ Penetrometer ◇ UU Triaxial

▲ Field Vane △ Unconfined Compression

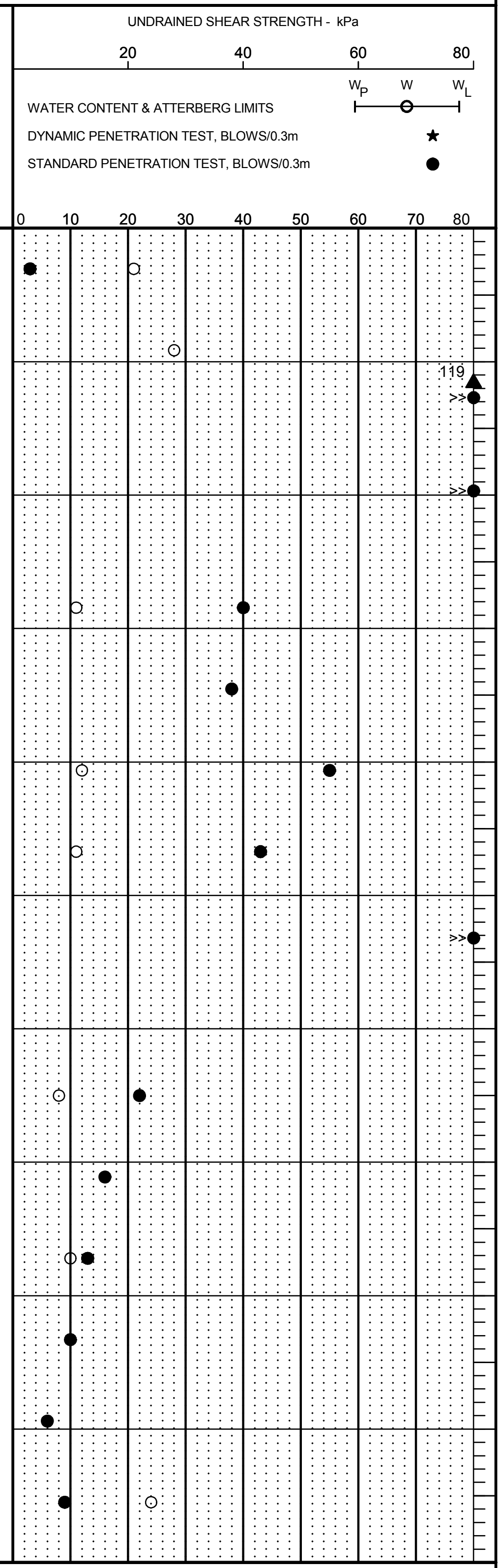
BOREHOLE RECORD

BH12

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-24 TO 2017-05-25 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	5.12								
	4.97	ROOTMAT/TOPSOIL							
		Very soft, brown lean CLAY with sand			SS	1	200	0-1-2-4 (3)	
				∇	ST	2	250	Push	
1	3.90	Compact to very dense brown and grey GRAVEL with silt and sand (diamicton) - with occasional cobbles and boulders			SS	3	50	50 / 100 mm	
2					SS	4	100	12-50 / 125 mm	
3					SS	5	200	24-21-19-30 (40)	
					SS	6	150	18-26-21-17 (38)	
4					SS	7	175	20-44-30-25 (55)	
					SS	8	250	37-32-33-10 (43)	S
5					SS	9	50	22-70 / 75 mm	
6	-1.08	Loose to compact grey sandy SILT to SILT with sand - with trace gravel - partially cemented to cemented			SS	10	200	5-8-14-13 (22)	
7					SS	11	200	12-9-7-9 (16)	
					SS	12	175	8-10-8-5 (13)	
8					SS	13	200	5-4-6-5 (10)	
9					SS	14	150	5-3-3-4 (6)	
					SS	15	0	3-4-6-3 (9)	



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

HARBOURSIDE GEOTECHNICAL CONSULTANTS, BOREHOLE RECORD 17-12-12

(Continued Next Page)

BH12

HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-24 TO 2017-05-25 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Loose to compact grey sandy SILT to SILT with sand - with trace gravel - partially cemented to cemented *(continued)*	SS	16	300	8-25-17-9 (26)	
12			SS	17	300	15-10-12-22 (22)	S
14		Very dense below 14.0 m depth	SS	18	250	16-29-49-70 / 125 mm	
			SS	19	350	89-52-60 / 125 mm	
16			GB	20	725		
17	-11.90		SS	21	75	50 / 75 mm	
		Very poor quality SILTSTONE - very weak to weak - with occasional silt seams	HQ	22	86%	20%	
19			HQ	23	51%	0%	

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

>>

>>

>>

■ Miniature Vane □ Torvane

◆ Penetrometer ◇ UU Triaxial

▲ Field Vane △ Unconfined Compression

(Continued Next Page)

BH12

HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-24 TO 2017-05-25 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
		Very poor quality SILTSTONE - very weak to weak - with occasional silt seams *(continued)*			HQ	24	49%	0%	
21									
22					HQ	25	77%	0%	
	-17.64								
23		End of borehole - 25-mm standpipe installed							
24									
25									
26									
27									
28									
29									

UNDRAINED SHEAR STRENGTH - kPa: 20 40 60 80

W_P W W_L

WATER CONTENT & ATTERBERG LIMITS

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane

◆ Penetrometer ◇ UU Triaxial

▲ Field Vane △ Unconfined Compression

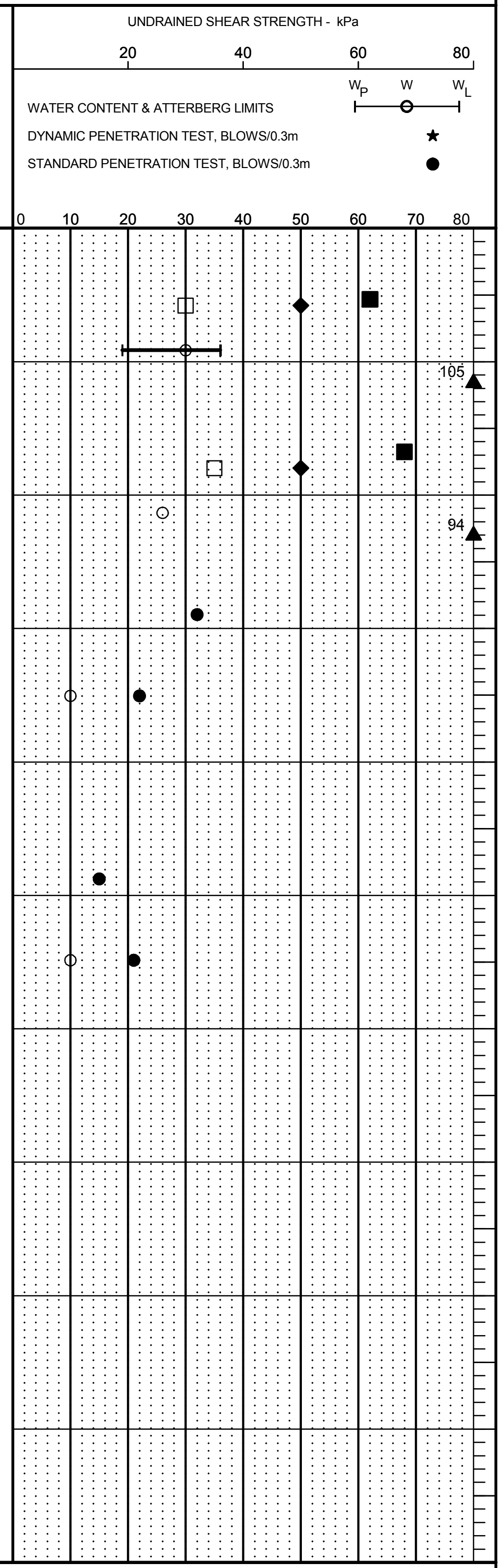
BOREHOLE RECORD

BH13

HARBOURSIDE Geotechnical Consultants

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-25 TO 2017-05-26 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
0	7.39	Soft to firm brown lean CLAY			ST	1	600	Push	
					SS	2	375	Push	
				▽	ST	3	600	Push	
					SS	4	450	Push	
	5.10	Compact grey to brown GRAVEL with silt and sand (diamicton)			SS	5	175	12-26-18-14 (32)	
					SS	6	325	24-9-13-15 (22)	
					SS	7	50	5-6-9-9 (15)	
					SS	8	150	10-11-10-28 (21)	
	1.35	Poor to fair quality light grey QUARTZOSE SANDSTONE - strong to very strong - moderately weathered - occasional white quartz seams - orange staining at fractures			HQ	9	100%	0%	
					HQ	10	100%	57%	
					HQ	11	90%	13%	
					HQ	12	100%	74%	
					HQ	13	100%	41%	



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

HARBOURSIDE GEOTECHNICAL CONSULTANTS, BOREHOLE RECORD 17-12-12

(Continued Next Page)

HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH13

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: BORING 2017-05-25 TO 2017-05-26 **WATER LEVEL:** 2017-05-28
PROJECT No.: 163567
DATUM: CGVD28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
	-3.28								
11		End of borehole - 25-mm standpipe installed							
12									
13									
14									
15									
16									
17									
18									
19									

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

HARBOURSIDE Geotechnical Consultants

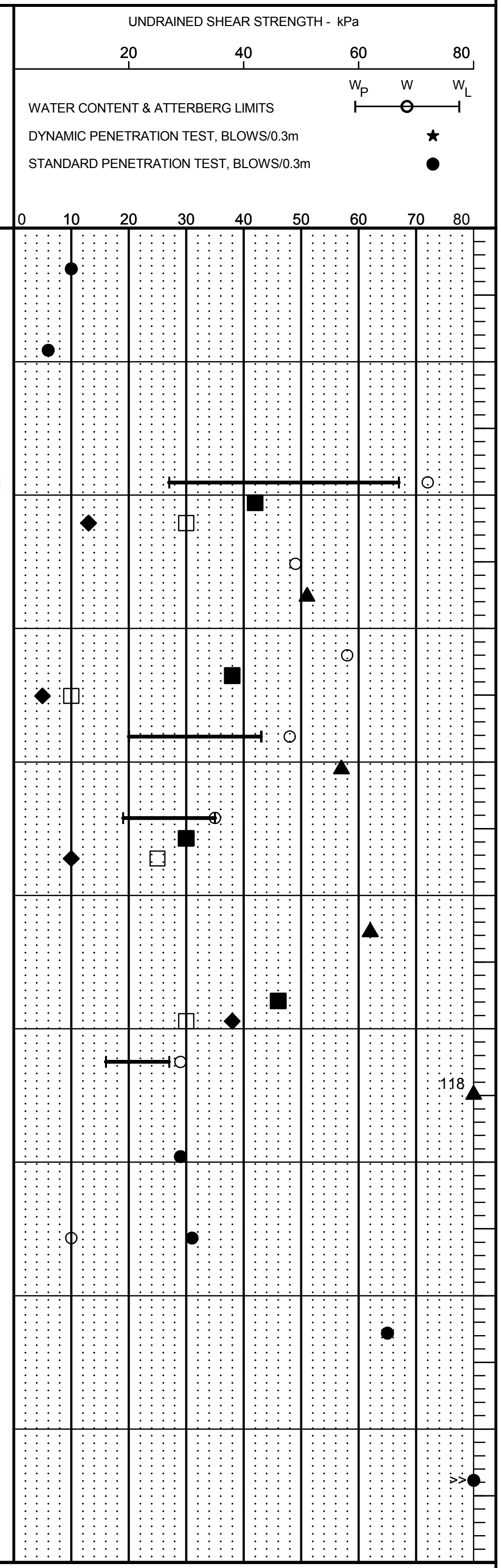
BOREHOLE RECORD

BH14

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-26 TO 2017-05-27 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

ELEVATION (m)	SOIL/BEDROCK DESCRIPTION
13.67	
13.52	ROOTMAT/TOPSOIL
	Loose brown GRAVEL with silt and sand
12.76	Soft to firm brown fat CLAY
10.16	Stiff to firm brown lean CLAY
7.12	Compact to very dense brown GRAVEL with silt and sand (diamicton)

WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
∇	SS	1	400	2-5-5-10 (10)	
	SS	2	400	10-5-3-3 (6)	
	ST	3	600	Push	C, Cu
	SS	4	125	Push	
	ST	5	450	Push	C
	SS	6	600	Push	
	ST	7	600	Push	Cu
	SS	8	50	Push	
	ST	9	600	Push	C
	SS	10	600	Push	
	ST	11	100	Push	
	SS	12	150	20-13-16-32 (29)	
	SS	13	250	41-24-8-23 (31)	
	SS	14	150	12-17-48-17 (65)	S
	SS	15	0	50 / 25 mm	



Miniature Vane
Torvane
Penetrometer
UU Triaxial
Field Vane
Unconfined Compression

(Continued Next Page)

HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH14

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: BORING 2017-05-26 TO 2017-05-27 **WATER LEVEL:** 2017-05-28
BH SIZE: HW

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm) REC. ROCK (%)	BLOWS / 150 mm (N VALUE) RQD %	OTHER TESTS
					SS	16	100	22-42-50 / 25 mm	
	3.46	Very poor to poor quality grey QUARTZOSE SANDSTONE - strong - moderately weathered			HQ	17	100%	35%	
11					HQ	18	90%	29%	
12					HQ	19	92%	0%	
	1.00	End of borehole - 25-mm standpipe installed							
13									
14									
15									
16									
17									
18									
19									

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

Scale: 0, 10, 20, 30, 40, 50, 60, 70, 80

>>●

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

HARBOURSIDE Geotechnical Consultants

TEST PIT RECORD

TP01

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: DUG 2017-05-28 **WATER LEVEL:** -

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

Scale: 0 10 20 30 40 50 60 70 80

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	OTHER TESTS
	16.49						
	16.0	ROOTMAT/TOPSOIL					
1	15.4	Loose brown silty SAND with gravel					
	15.0	Soft brown lean CLAY		▼			
	14.7	Compact grey silty GRAVEL with sand (diamicton)			GB	1	S
2		End of test pit - refusal on inferred bedrock - water seepage observed at 1.5 m depth					
3							
4							
5							
6							
7							
8							
9							

■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

HARBOURSIDE Geotechnical Consultants

TEST PIT RECORD

TP02

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: DUG 2017-05-28 **WATER LEVEL:** -

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	OTHER TESTS
	8.02						
	7.8	ROOTMAT/TOPSOIL					
		Soft brown lean CLAY - with organic material from 1.8 to 2.1 m depth			GB	1	S
	3.1	End of test pit due to limit of excavator reach - no water infiltration observed					

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m
STANDARD PENETRATION TEST, BLOWS/0.3m

Scale: 0 10 20 30 40 50 60 70 80

Depth scale: 1 2 3 4 5 6 7 8 9

- ■ Miniature Vane
- □ Torvane
- ◆ Penetrometer
- ◇ UU Triaxial
- ▲ Field Vane
- △ Unconfined Compression

HARBOURSIDE Geotechnical Consultants

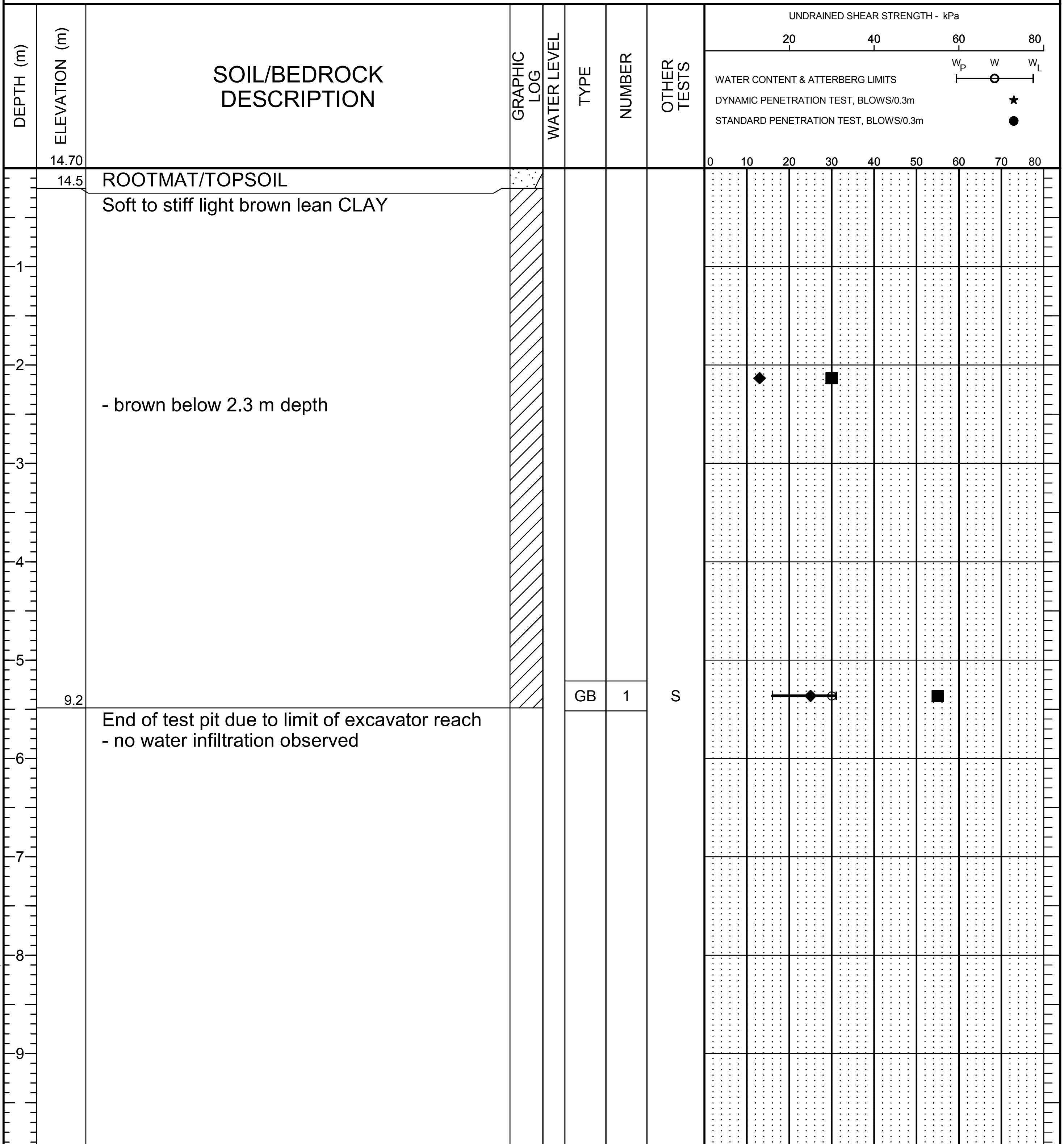
TEST PIT RECORD

TP03

CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATES: DUG 2017-05-28 **WATER LEVEL: -**

PROJECT No.: 163567
DATUM: CGVD28

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	OTHER TESTS
	14.70						
	14.5	ROOTMAT/TOPSOIL					
		Soft to stiff light brown lean CLAY					
		- brown below 2.3 m depth					
	9.2				GB	1	S
		End of test pit due to limit of excavator reach - no water infiltration observed					



Miniature Vane
Torvane
Penetrometer
UU Triaxial
Field Vane
Unconfined Compression

TEST PIT RECORD

TP04

HARBOURSIDE Geotechnical Consultants

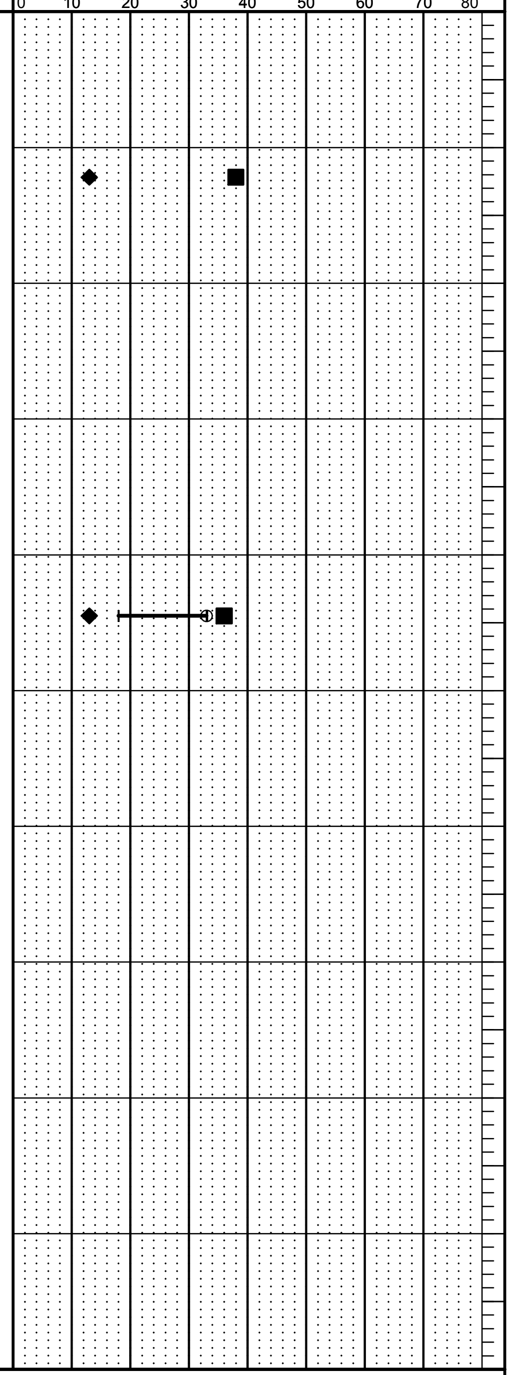
CLIENT: HARBOURSIDE ENGINEERING CONSULTANTS
PROJECT No.: 163567
LOCATION: DICKS BROOK BRIDGE, GROS MORNE NATIONAL PARK, NL
DATUM: CGVD28
DATES: DUG 2017-05-28
WATER LEVEL: -

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	OTHER TESTS
	15.50						
	15.3	ROOTMAT/TOPSOIL					
		Soft to stiff light brown lean CLAY					
2		- dark brown below 1.8 m depth					
					GB	1	S
	10.0	End of test pit due to limit of excavator reach - no water infiltration observed					

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80



■ Miniature Vane □ Torvane
◆ Penetrometer ◇ UU Triaxial
▲ Field Vane △ Unconfined Compression

APPENDIX B

Particle-Size Analyses

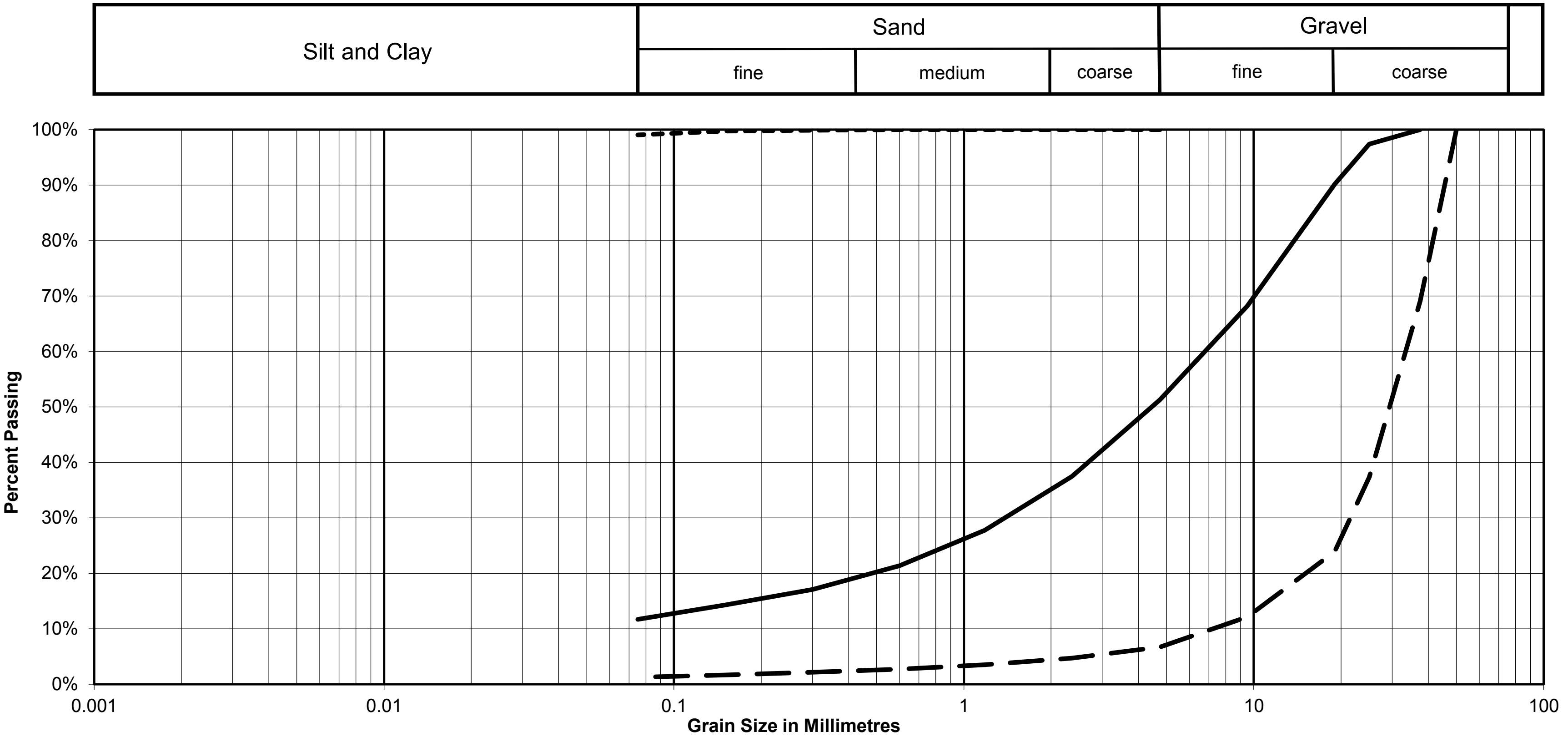
Atterberg Limits Results

Consolidation Test Results

CIU Test Results

Checked: VCG

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH01	SS1	0.5-1.1	49%	40%	12%	Silty Gravel with Sand
— — -	BH01	SS8	6.0-6.6	93%	5%	1%	Well-Graded Gravel
- - - -	BH01	ST14	10.2-10.8		1%	99%	Lean Clay

PROJECT No.: 163567

HGC

t: 1.902.405.4696 | f: 1.902.405.4693

219 Waverley Road, Suite 200

Dartmouth, NS B2X 2C3

http://harboursideengineering.ca

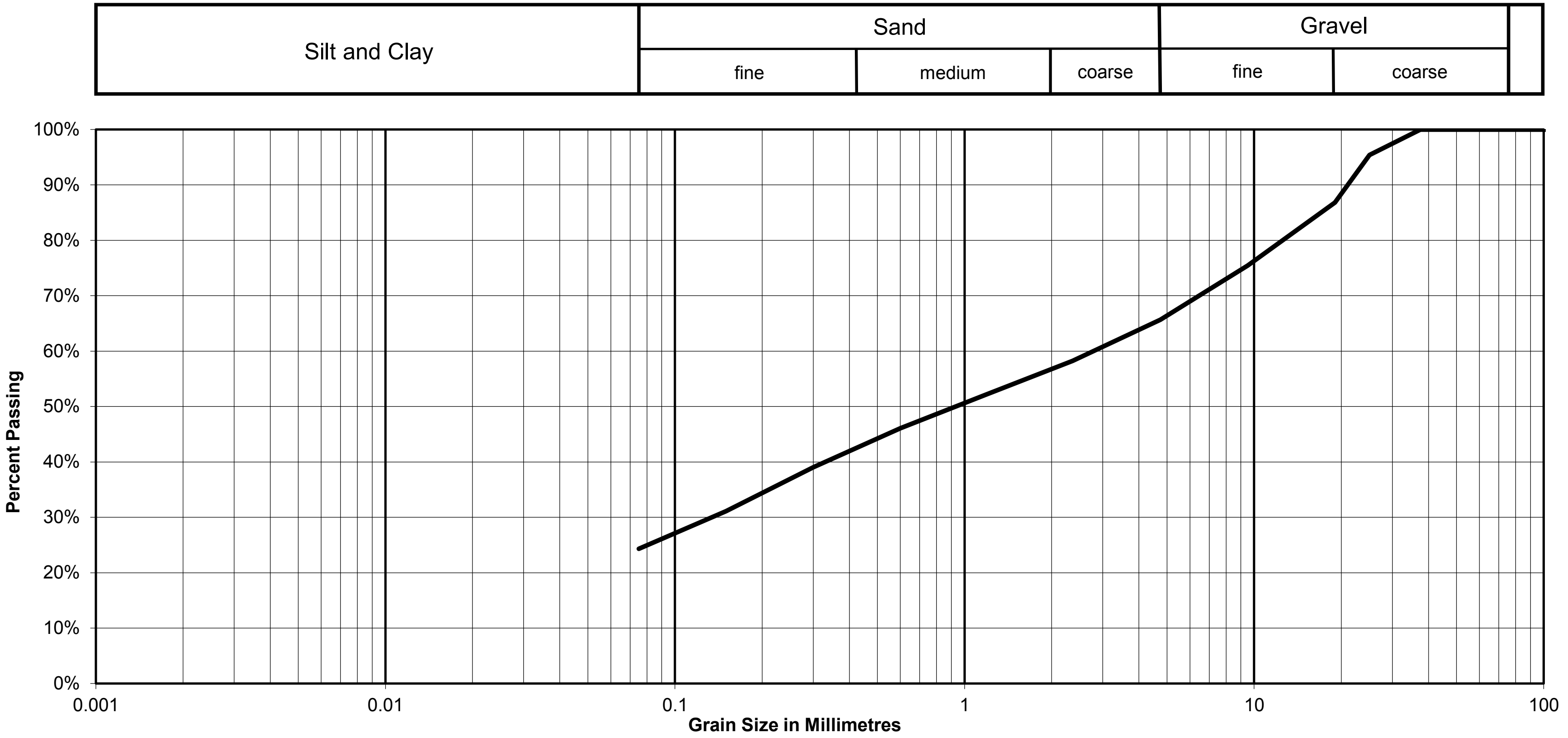
CLIENT HEC

PROJECT Dicks Brook Bridge Replacement

LOCATION Dicks Brook, Gros Morne National Park, NL

Checked: VCG

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH01	SS19	14.2-14.8	34%	41%	24%	Silty Sand with Gravel

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PROJECT Dicks Brook Bridge Replacement
LOCATION Dicks Brook, Gros Morne National Park, NL

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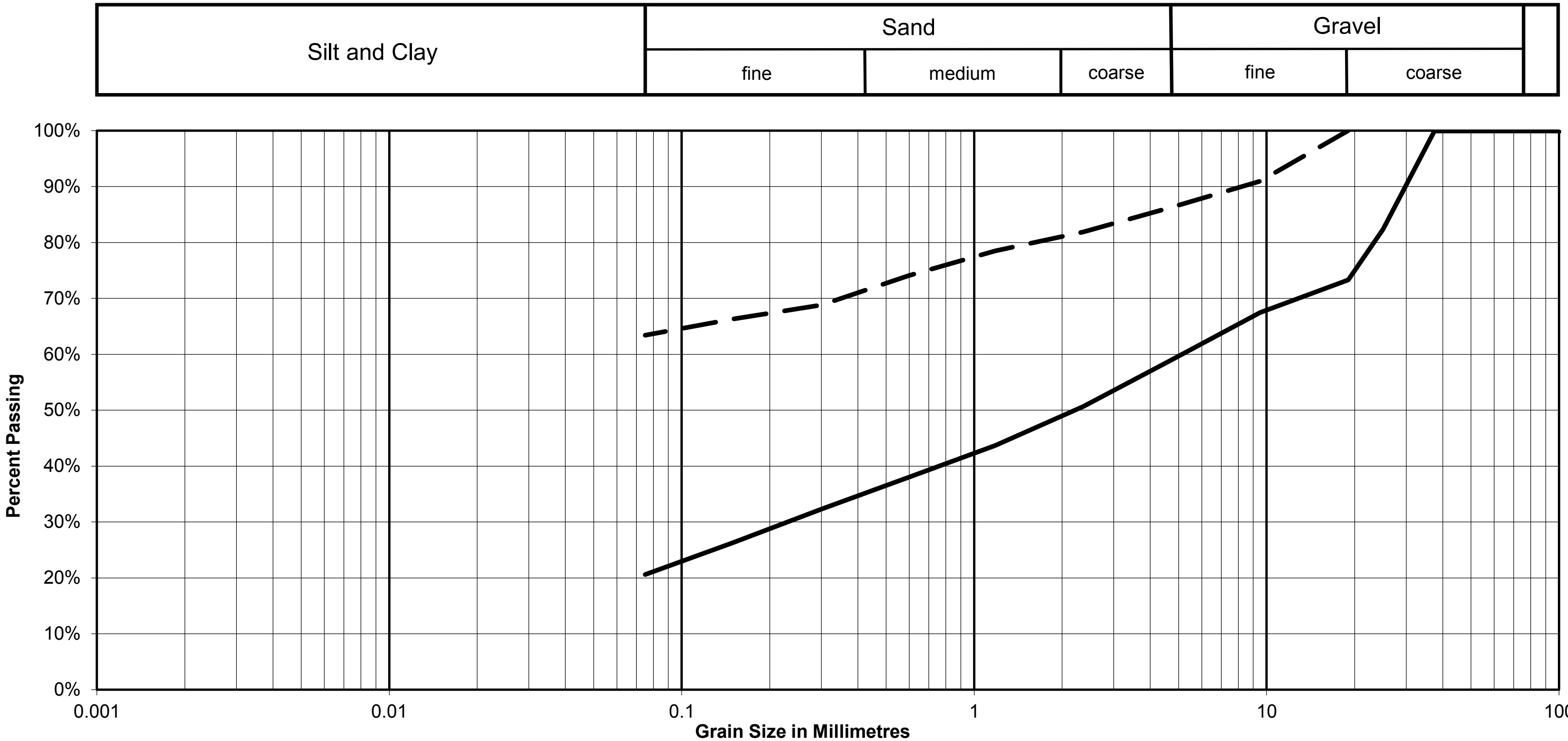
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH02	SS19	14.8-15.4	41%	38%	21%	Silty Gravel with Sand
— — -	BH02	SS22	19.5- 20.1	14%	23%	63%	Sandy Silt

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LOCATION Dicks Brook, Gros Morne National Park, NL

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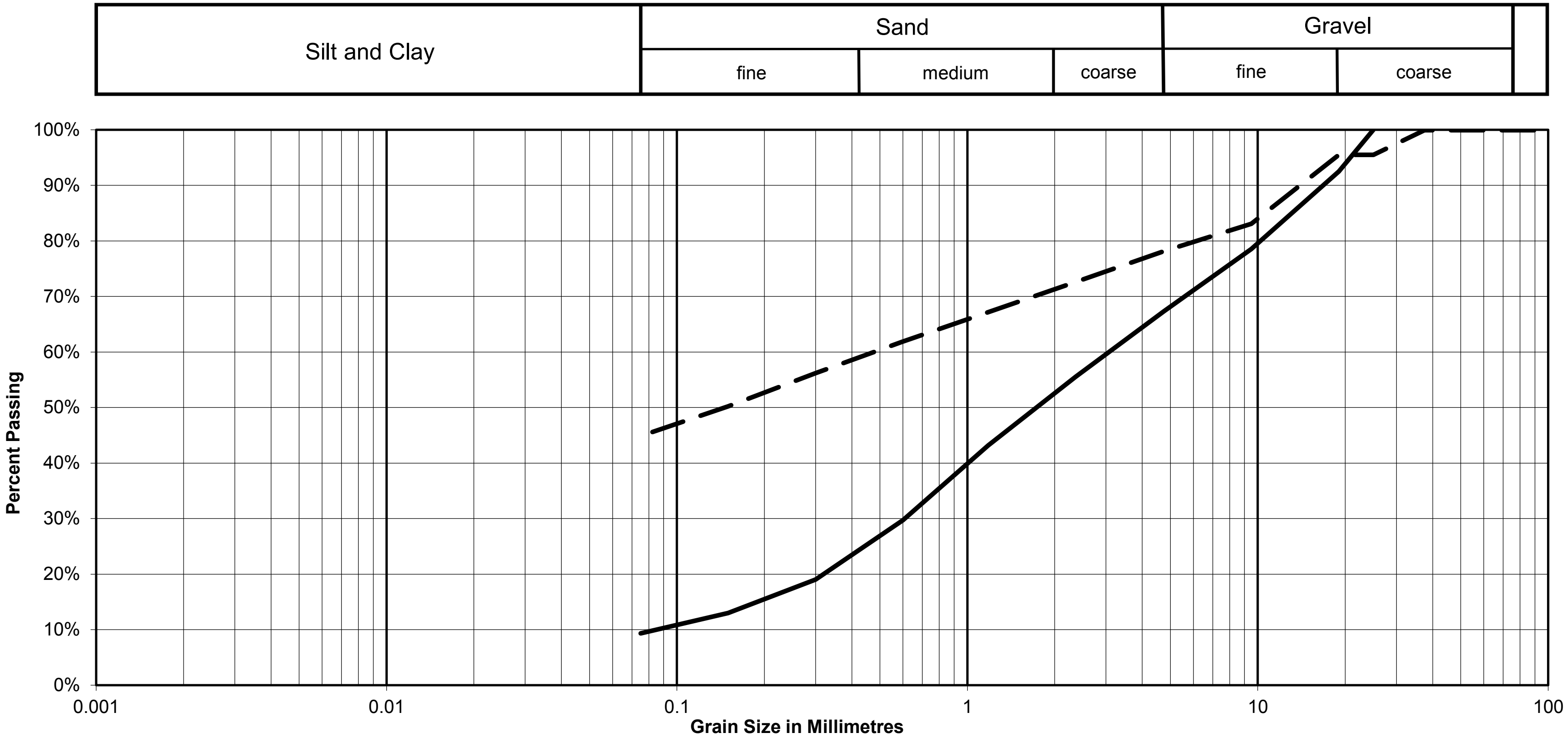
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH06	SS2	0.8-1.4	33%	58%	9%	Well-Graded Sand with Silt and Gravel
— — -	BH06	SS11	7.2-7.9	22%	33%	45%	Clayey Sand with Gravel

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PROJECT Dicks Brook Bridge Replacement

LOCATION Dicks Brook, Gros Morne National Park, NL

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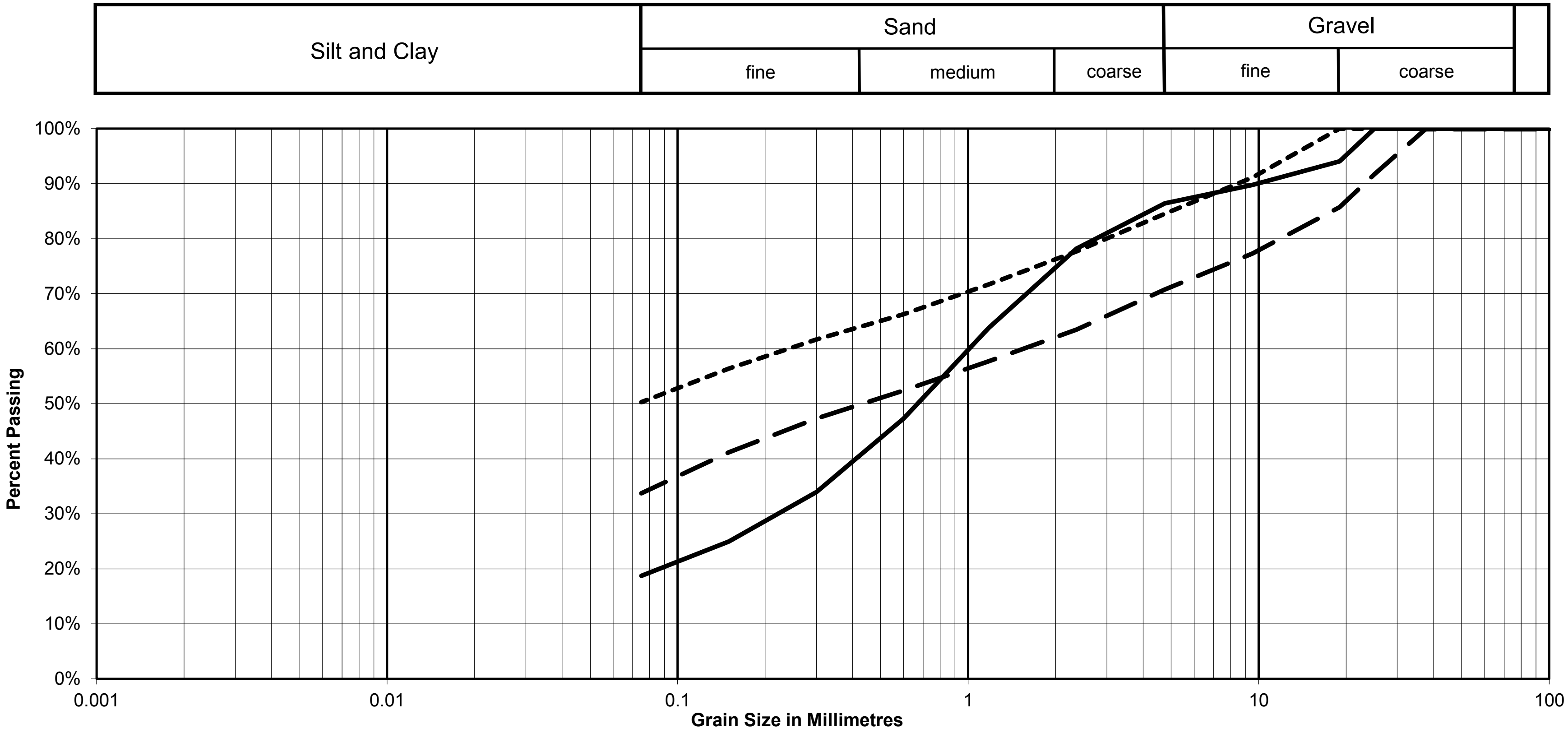
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH11	SS4	2.2-2.8	14%	68%	19%	Silty Sand
— — -	BH11	SS14	9.0-9.6	29%	37%	34%	Silty Sand with Gravel
- - - -	BH11	SS22	16.0-16.6	15%	34%	50%	Sandy Silt with Gravel

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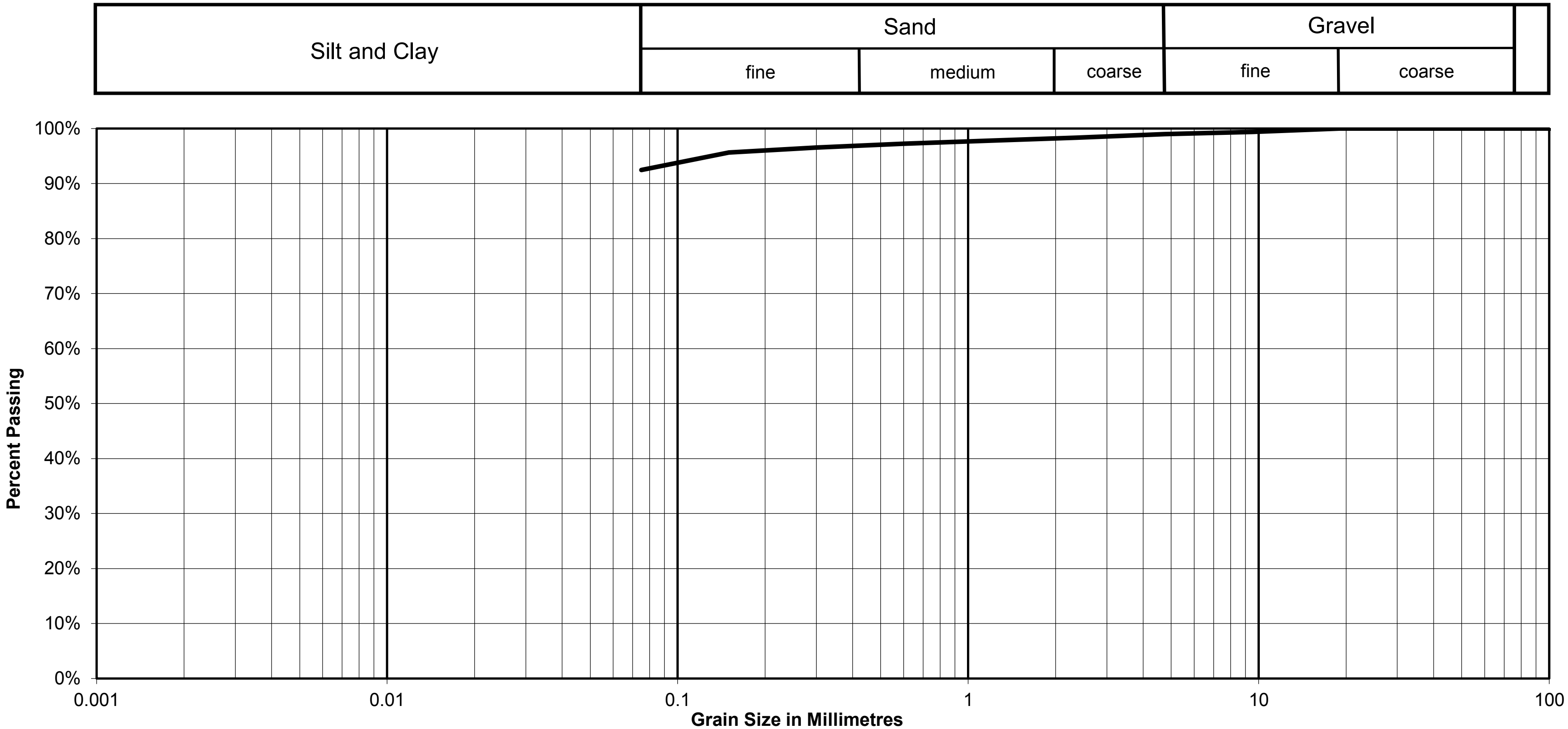
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH11	ST09	5.3-5.9	1%	7%	92%	Lean Clay

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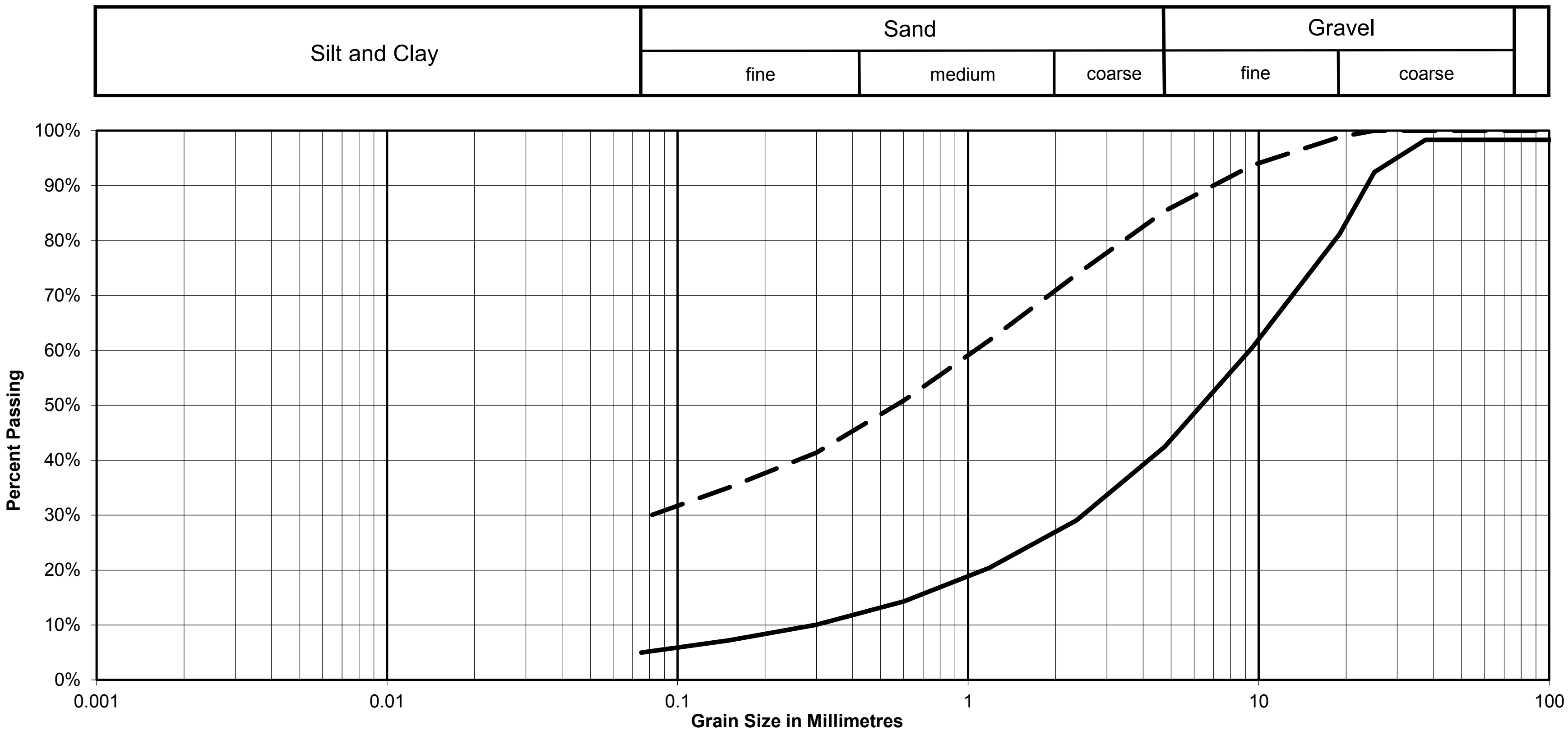
PROJECT Dicks Brook Bridge Replacement

LOCATION Dicks Brook, Gros Morne National Park, NL

Checked: **VCG**

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH12	SS8A	4.4-4.7	58%	37%	5%	Welll-Graded Gravel with Silt and Sand
— — -	BH12	SS17	11.6-12.2	15%	56%	29%	Silty Sand with Gravel

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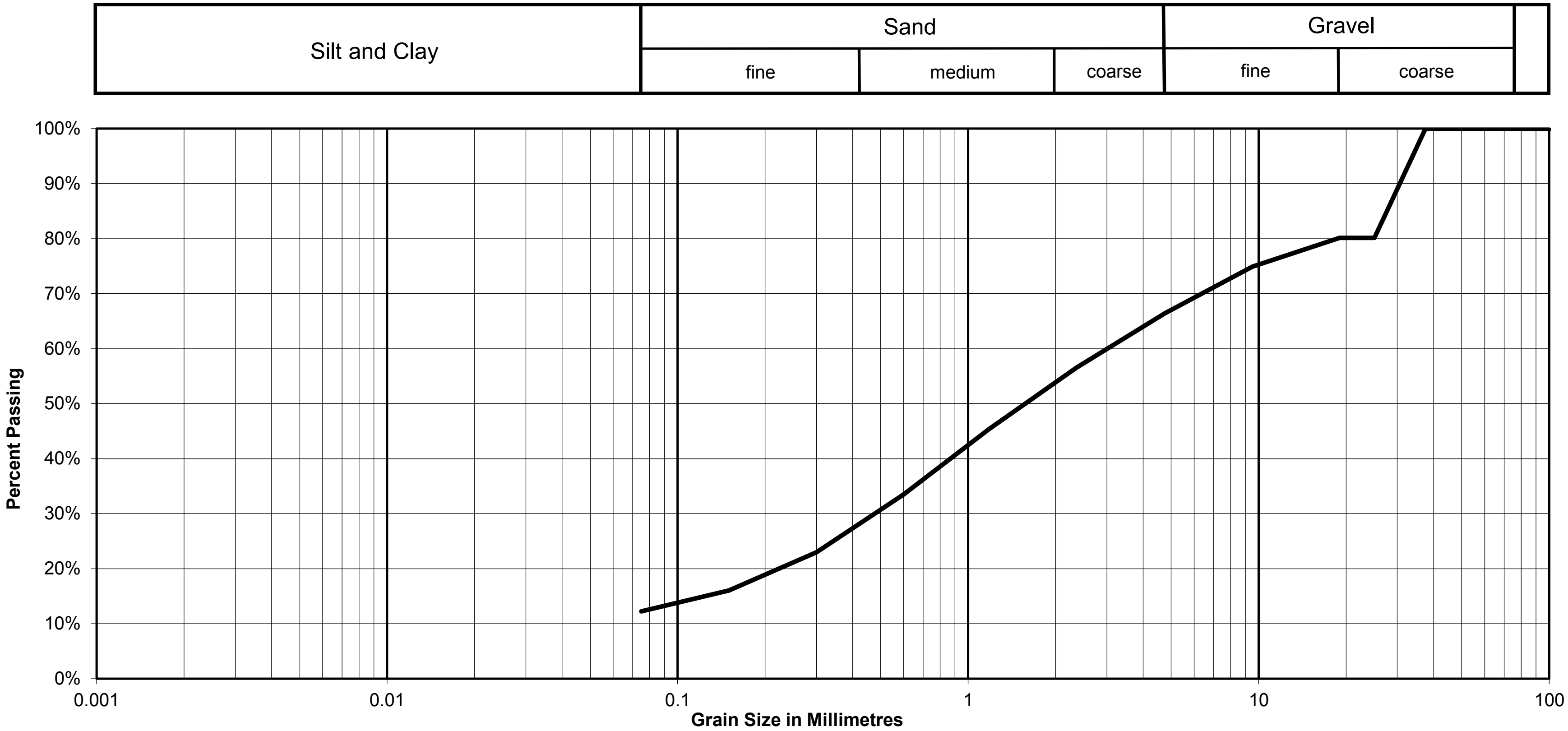
CLIENT HEC
PROJECT Dicks Brook Bridge Replacement
LOCATION Dicks Brook, Gros Morne National Park, NL

Checked: **VCG**

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH14	SS14	8.0-8.6	34%	54%	12%	Poorly-graded Sand with Silt and Gravel

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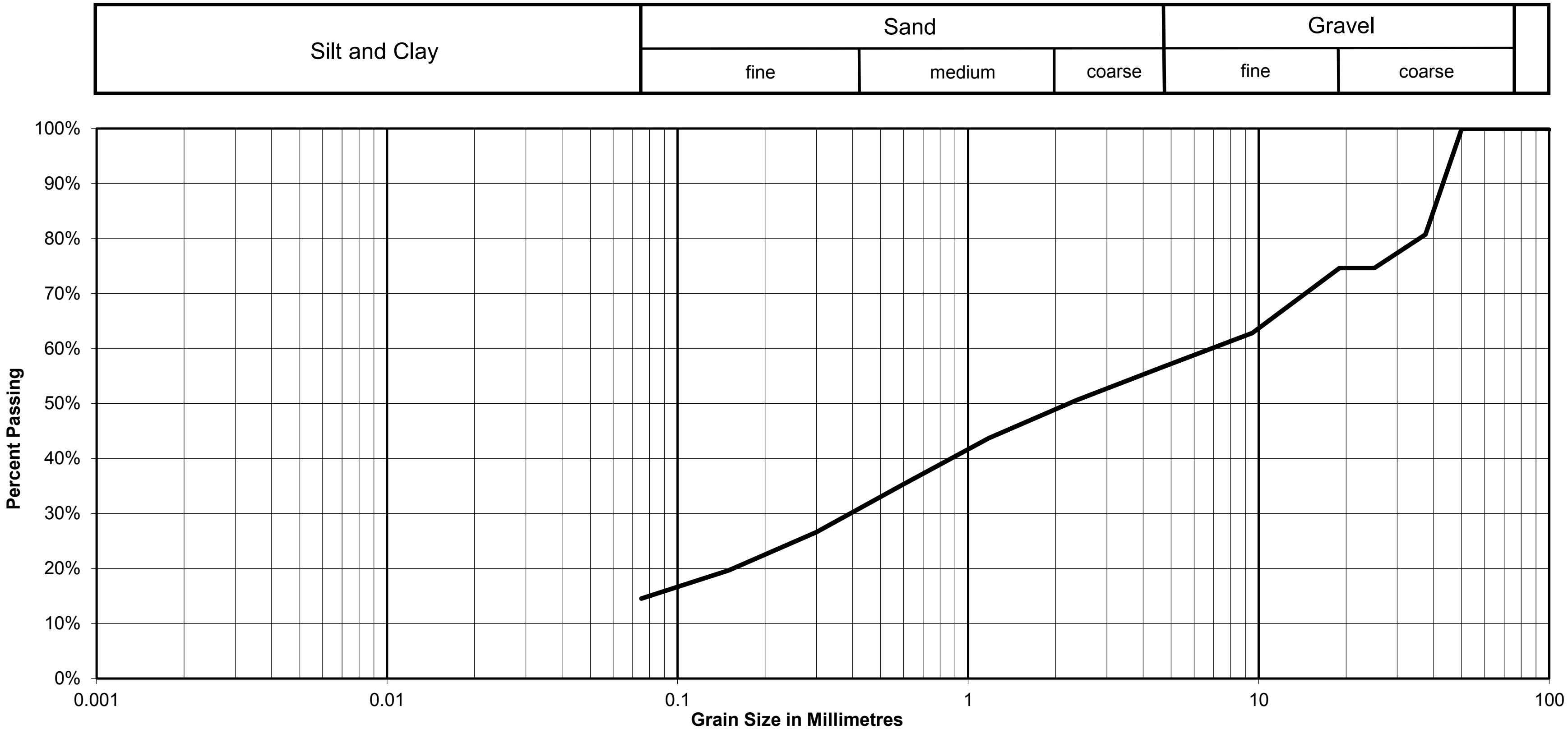
CLIENT	HEC
PROJECT	Dicks Brook Bridge Replacement
LOCATION	Dicks Brook, Gros Morne National Park, NL

Checked: **VCG**

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	TP01	GS01	1.5-1.8	43%	42%	15%	Silty Gravel with Sand

PROJECT No.: 163567

H G C

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CLIENT HEC
PROJECT Dicks Brook Bridge Replacement
LOCATION Dicks Brook, Gros Morne National Park, NL

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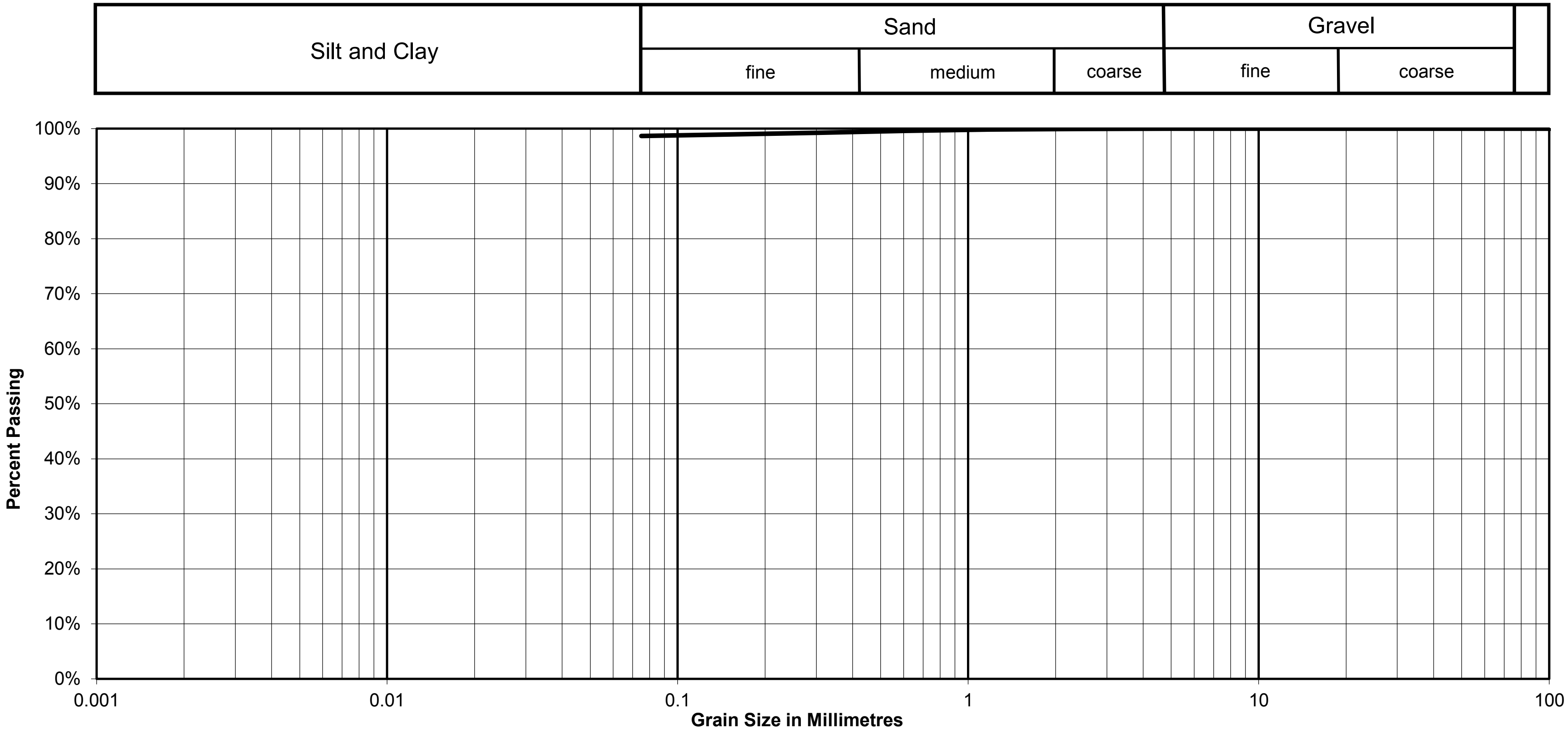
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	TP02	GS01	4.6-4.9	0%	1%	99%	Lean Clay

PROJECT No.: 163567

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CLIENT HEC

PROJECT Dicks Brook Bridge Replacement

LOCATION Dicks Brook, Gros Morne National Park, NL

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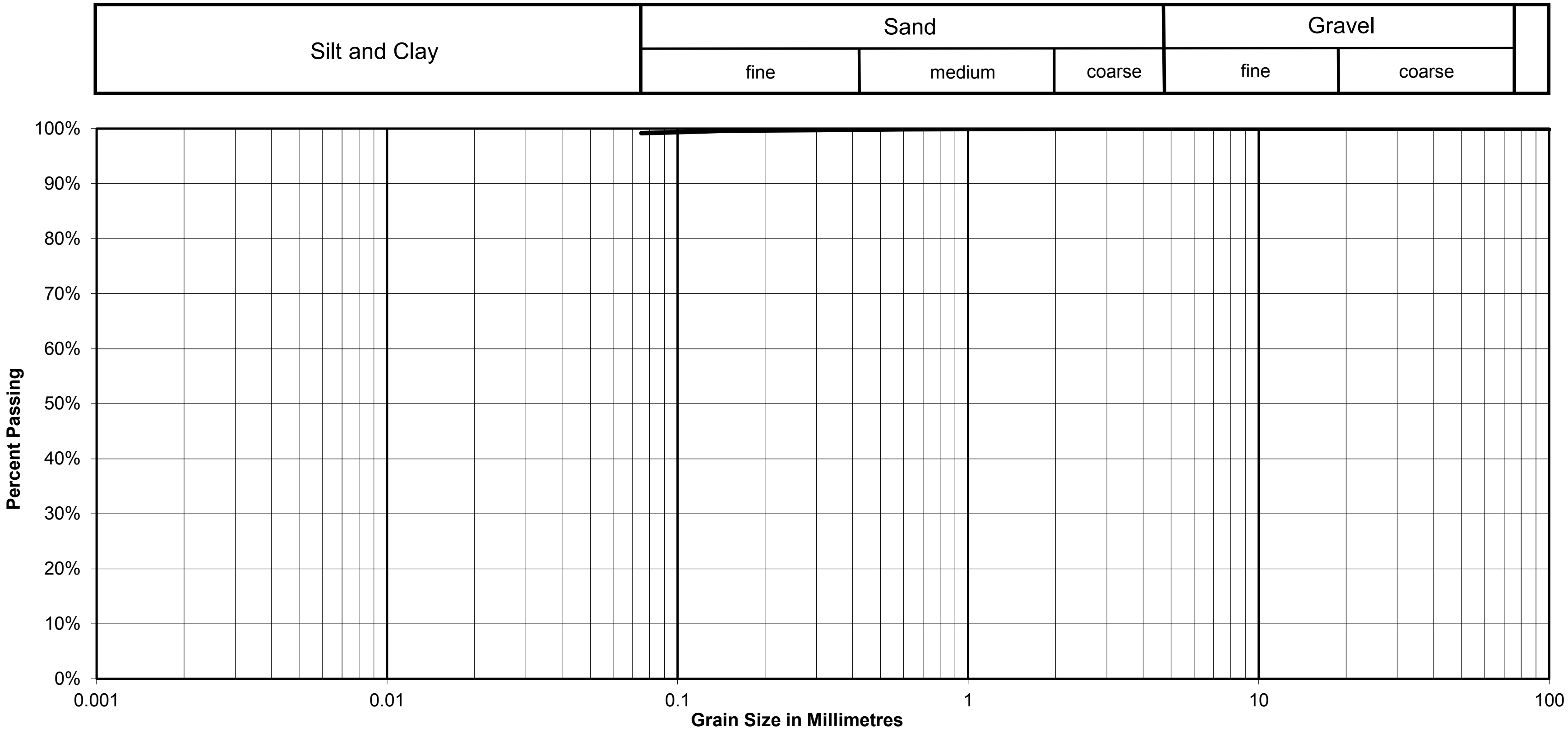
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	TP03	GS01	5.2-5.5		1%	99%	Lean Clay

PROJECT No.: 163567

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PROJECT Dicks Brook Bridge Replacement

LOCATION Dicks Brook, Gros Morne National Park, NL

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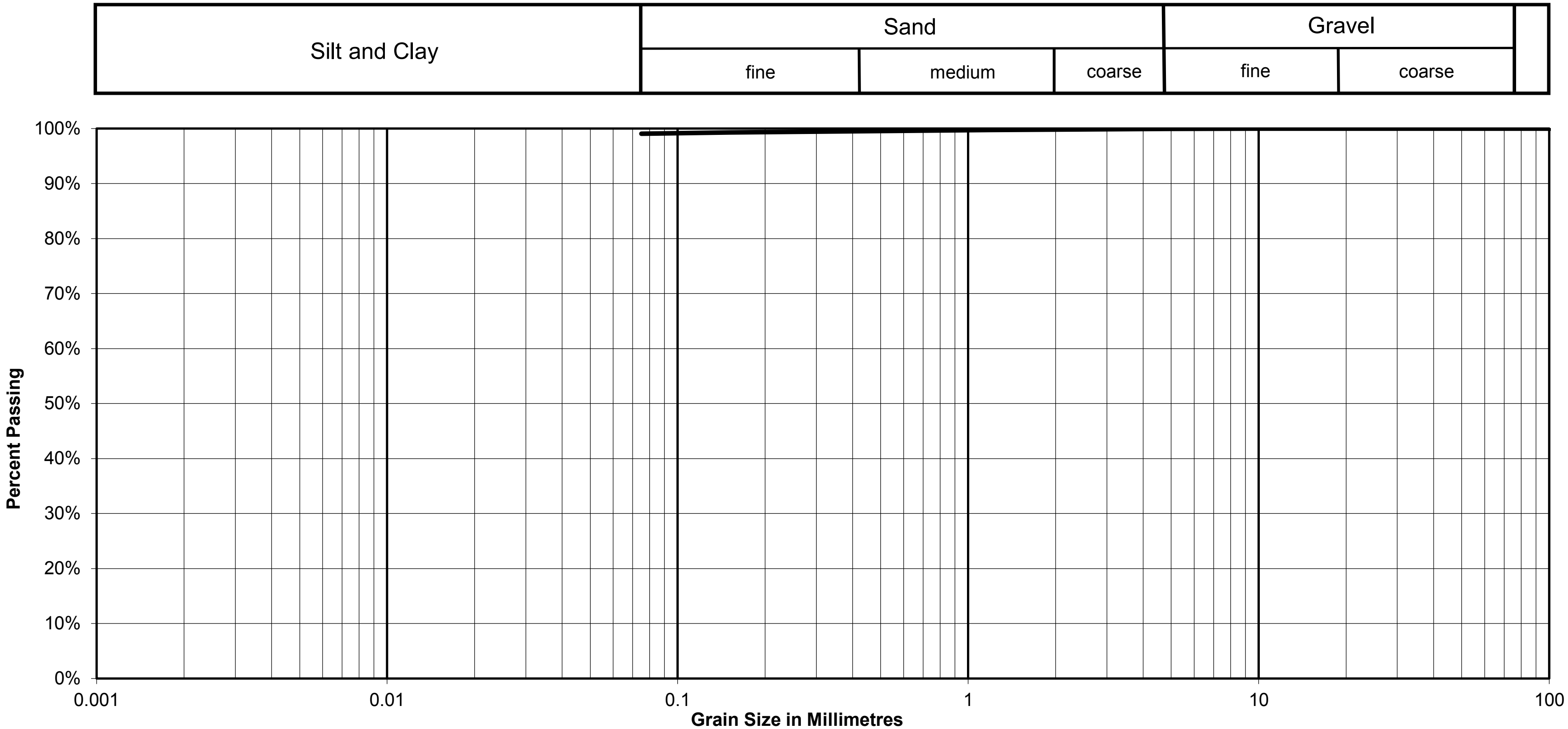
GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	TP04	GS01	4.3-4.6	0%	1%	99%	Lean Clay

PROJECT No.: 163567

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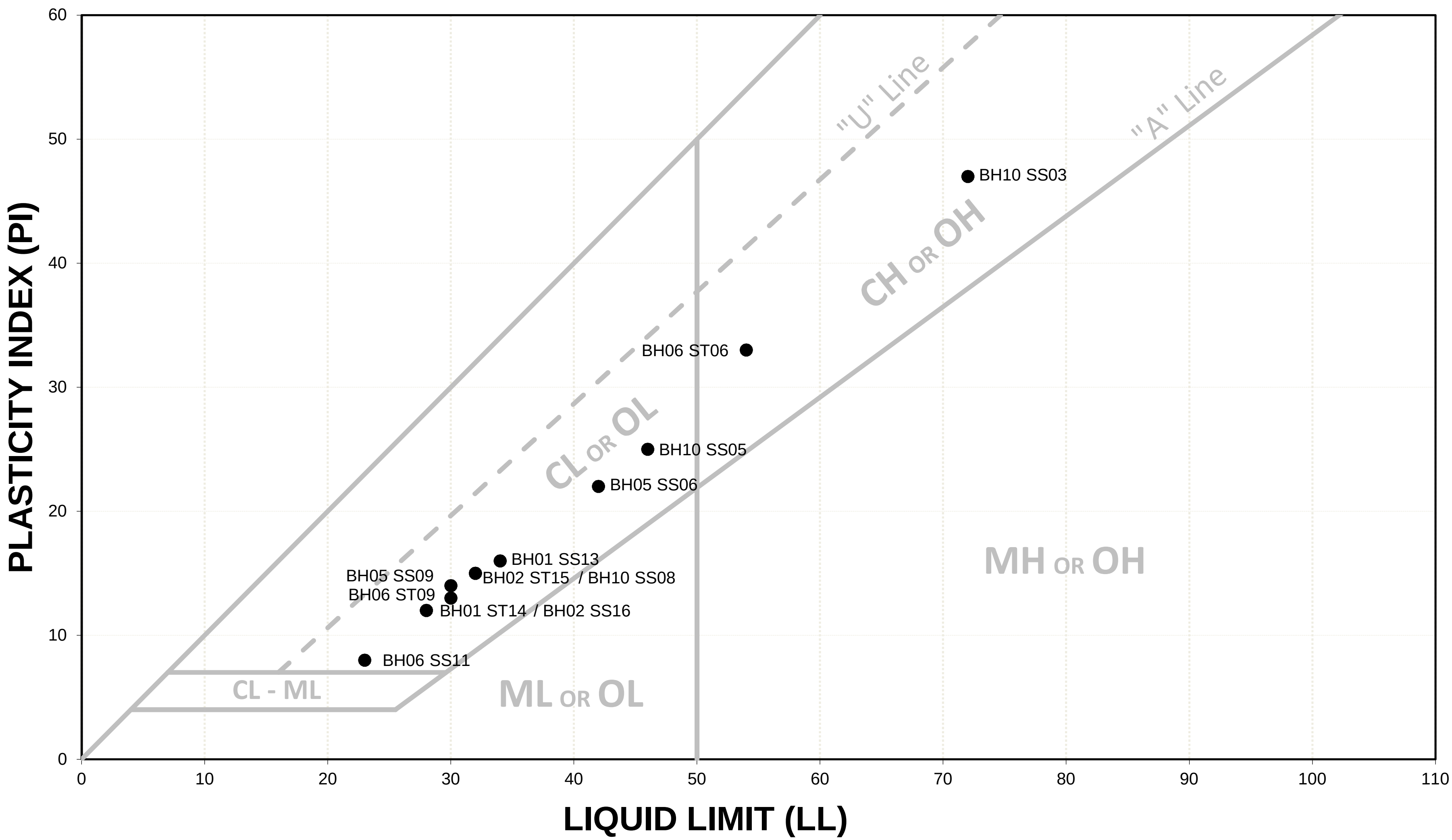
CLIENT	HEC
PROJECT	Dicks Brook Bridge Replacement
LOCATION	Dicks Brook, Gros Morne National Park, NL

Checked: VCG

ATTERBERG LIMITS



PROJECT No.: 163567

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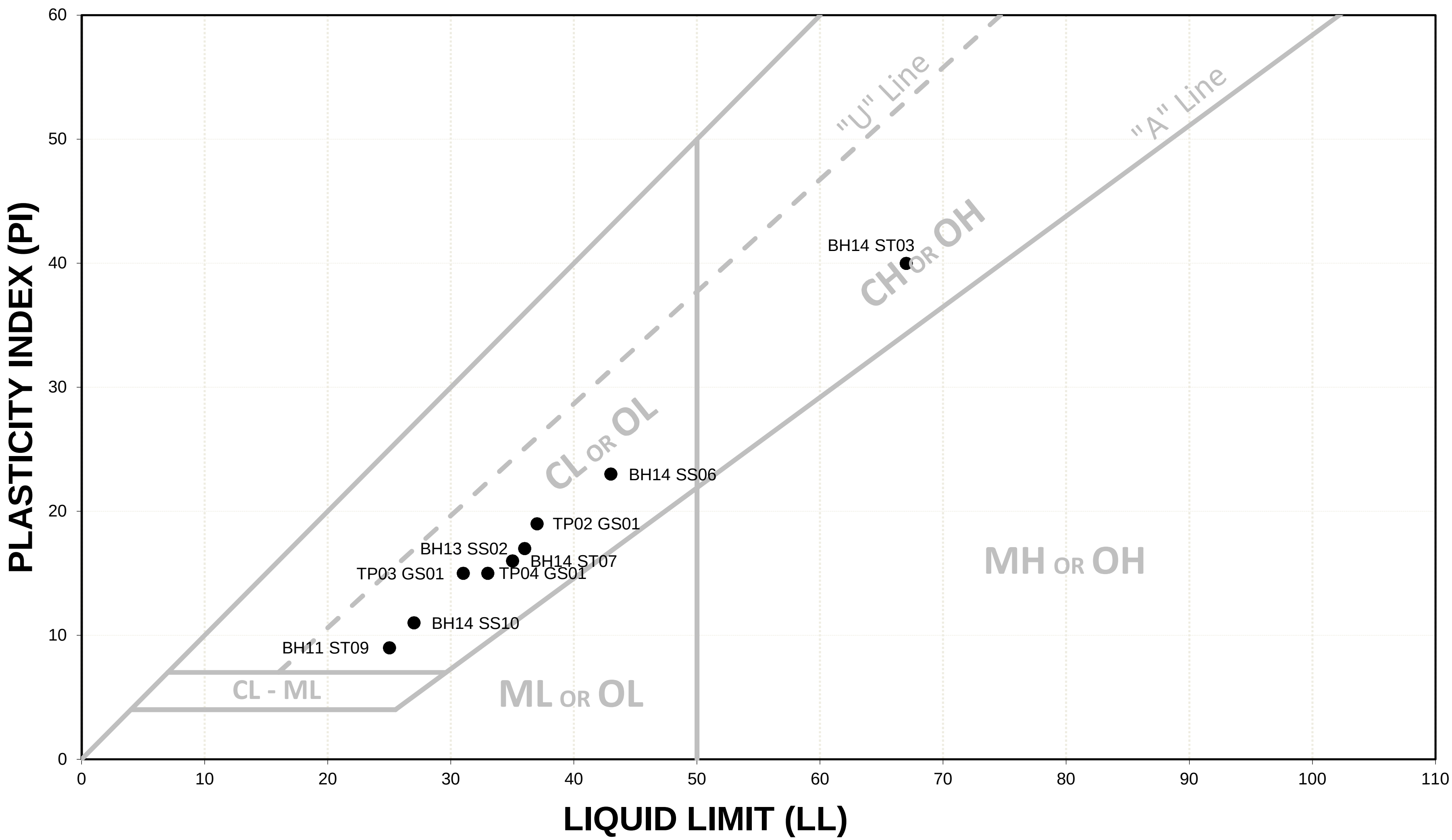
CLIENT	HEC
PROJECT	Dicks Brook Bridge Replacement
LOCATION	Dicks Brook, Gros Morne National Park, NL

Checked: VCG

ATTERBERG LIMITS



PROJECT No.: 163567

HGC

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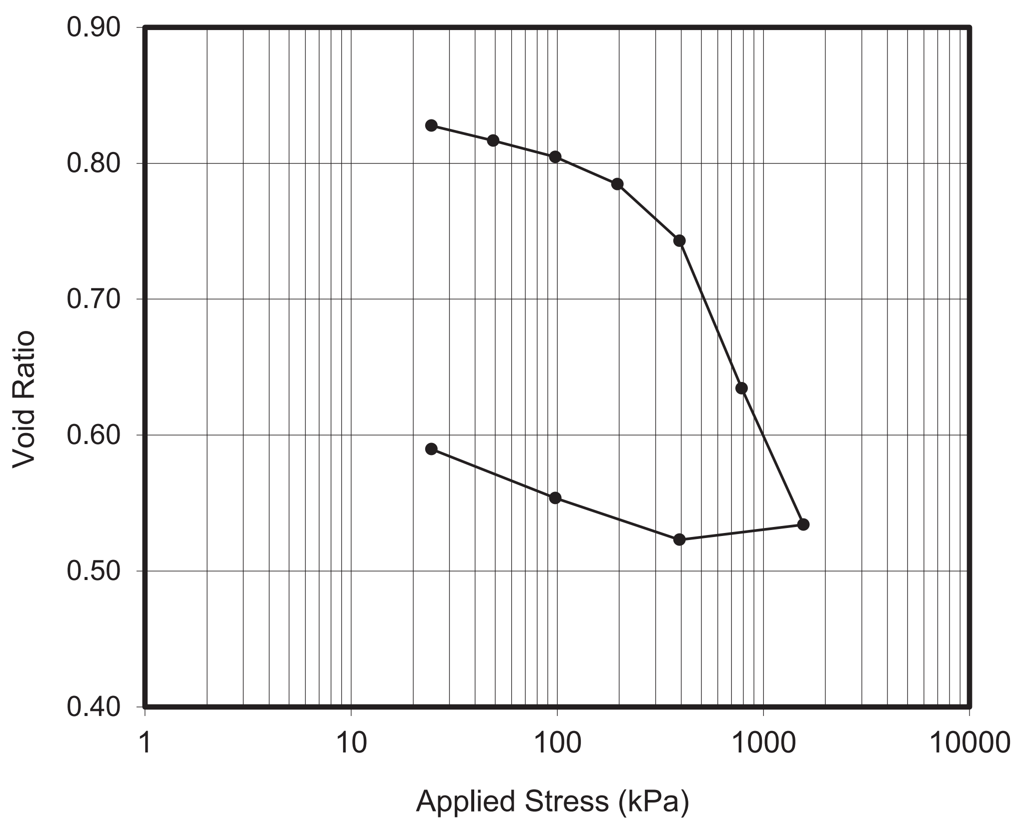
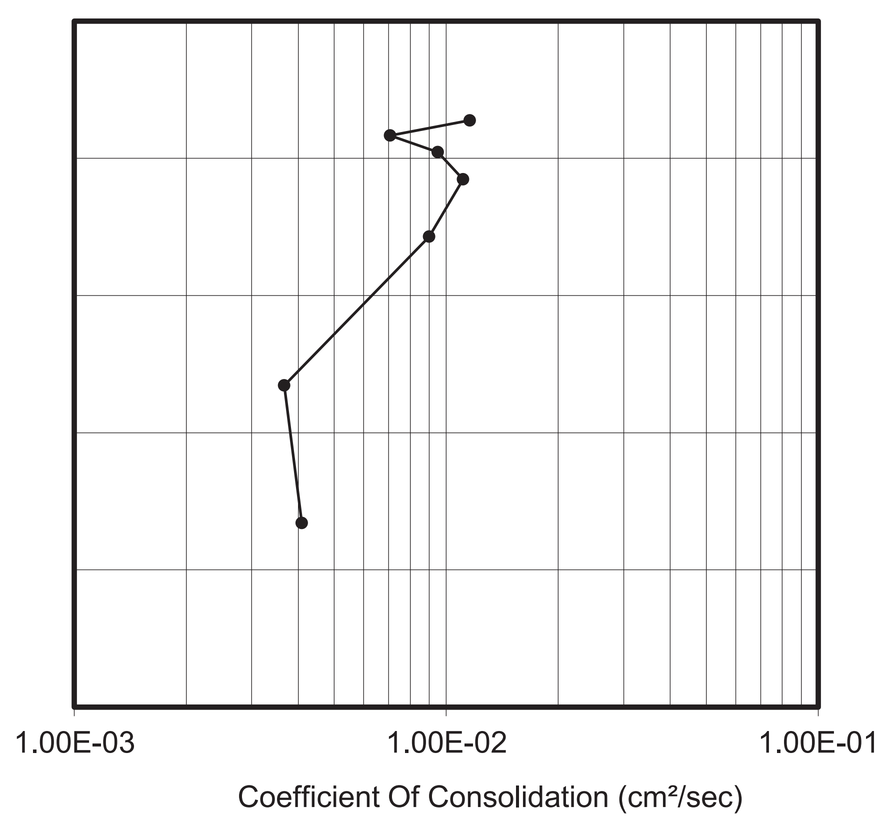
CLIENT	HEC
PROJECT	Dicks Brook Bridge Replacement
LOCATION	Dicks Brook, Gros Morne National Park, NL





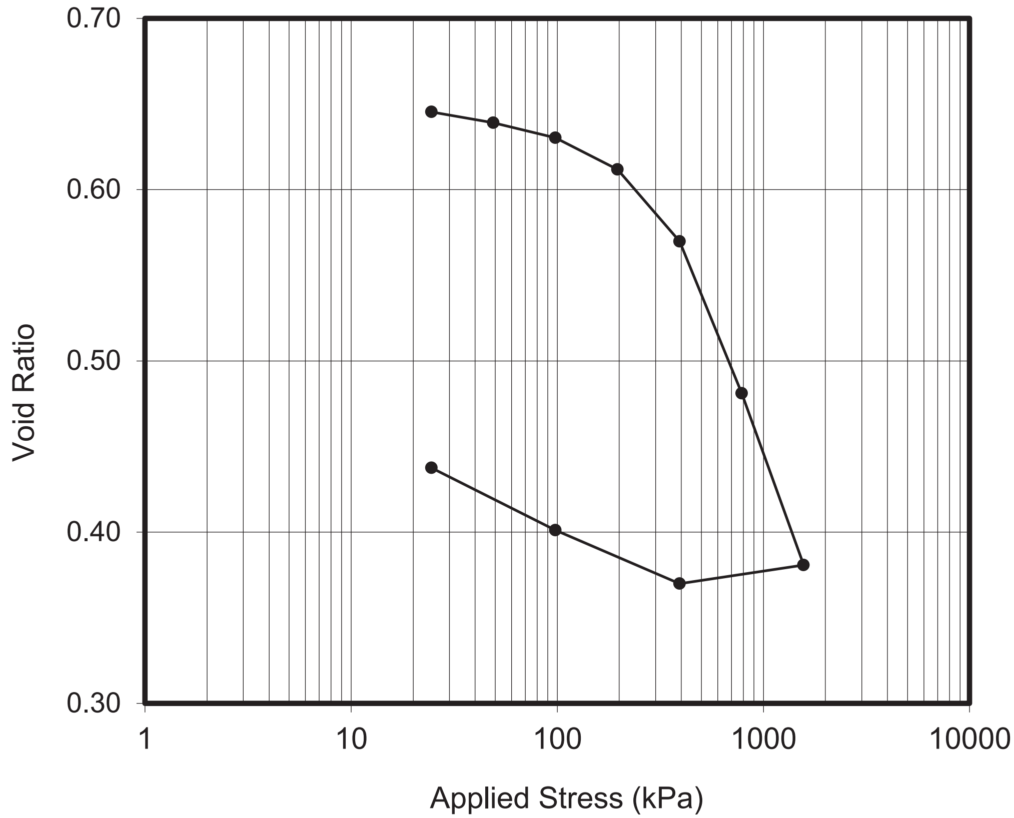
Stantec Consulting Ltd.
121619497.300.125

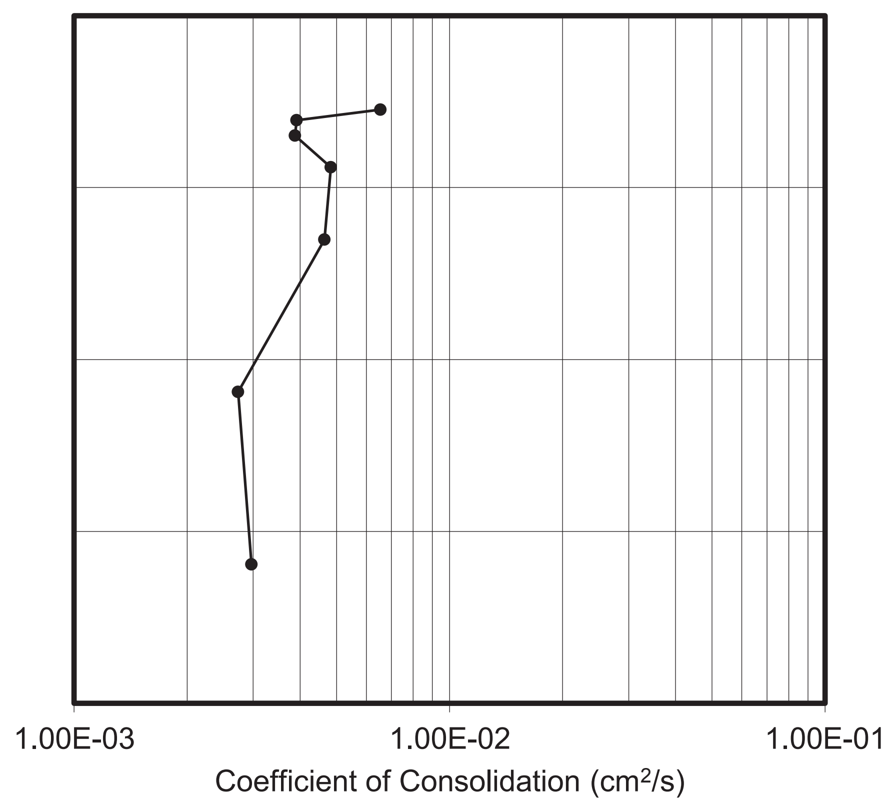
Harbourside - 163567

Boring No.	Sample No.	Depth
BH 05	SA 7	14'2" - 14'6"

One-Dimensional Consolidation ASTM D2435

Tested By: MVG
Checked By: HMW
Date: 2-Feb-17



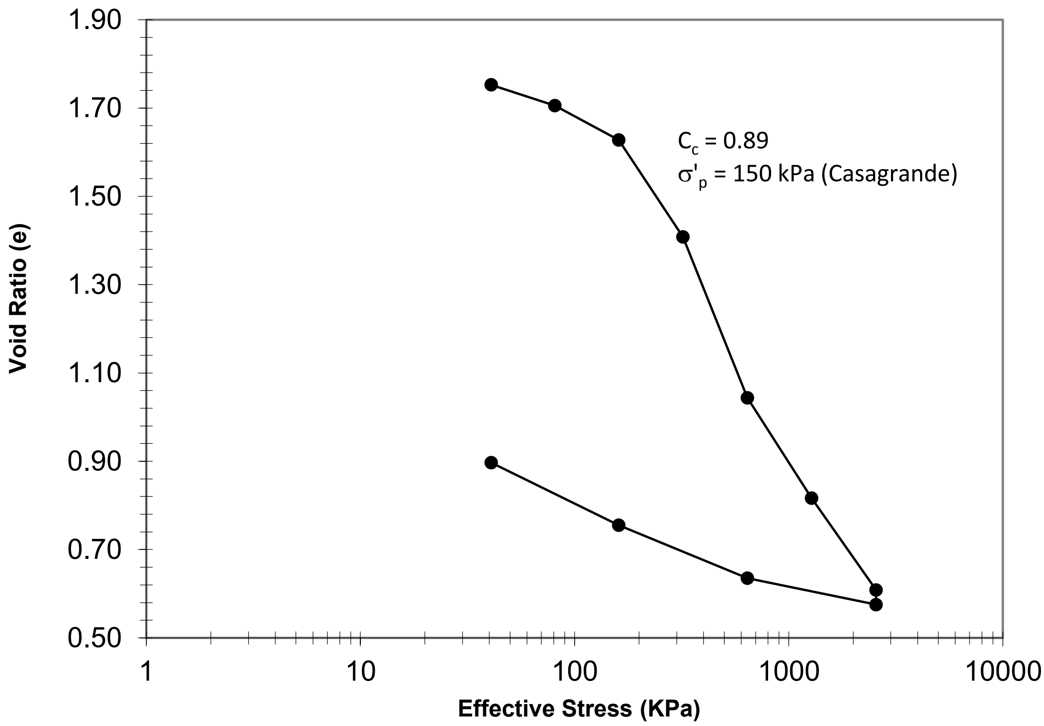


Stantec Consulting Ltd. 121619497.300.125 Stantec	Harbourside - 163567		
	Boring No.	Sample No.	Depth
	BH 05	SA 10	20' - 20'4"

One-Dimensional Consolidation ASTM D2435

Tested By: MVG
Checked By: HMW
Date: 2-Feb-17

Consolidation Test Results



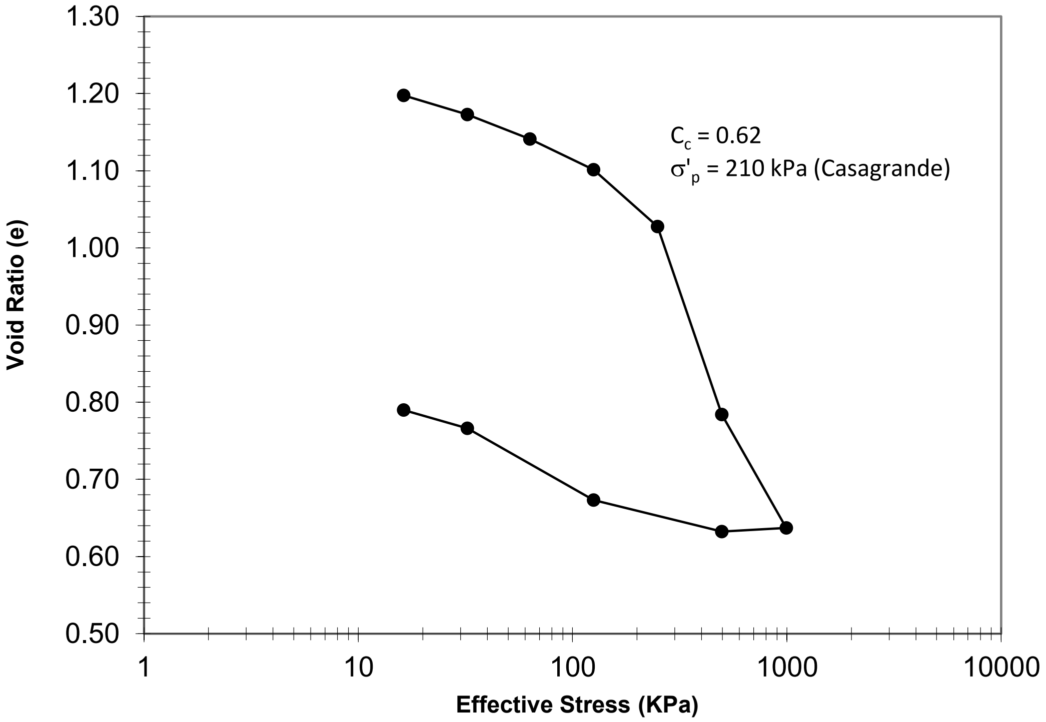
Project Name : Dicks Brook Bridge
Project Number: 163567

Sample: BH14 ST3 **Sample Depth (m):** 2.1

Sample Information

Initial Moisture Content:	69	%
Initial Dry Unit Weight:	9.5	kN/m^3
Assumed Specific Gravity	2.7	

Consolidation Test Results



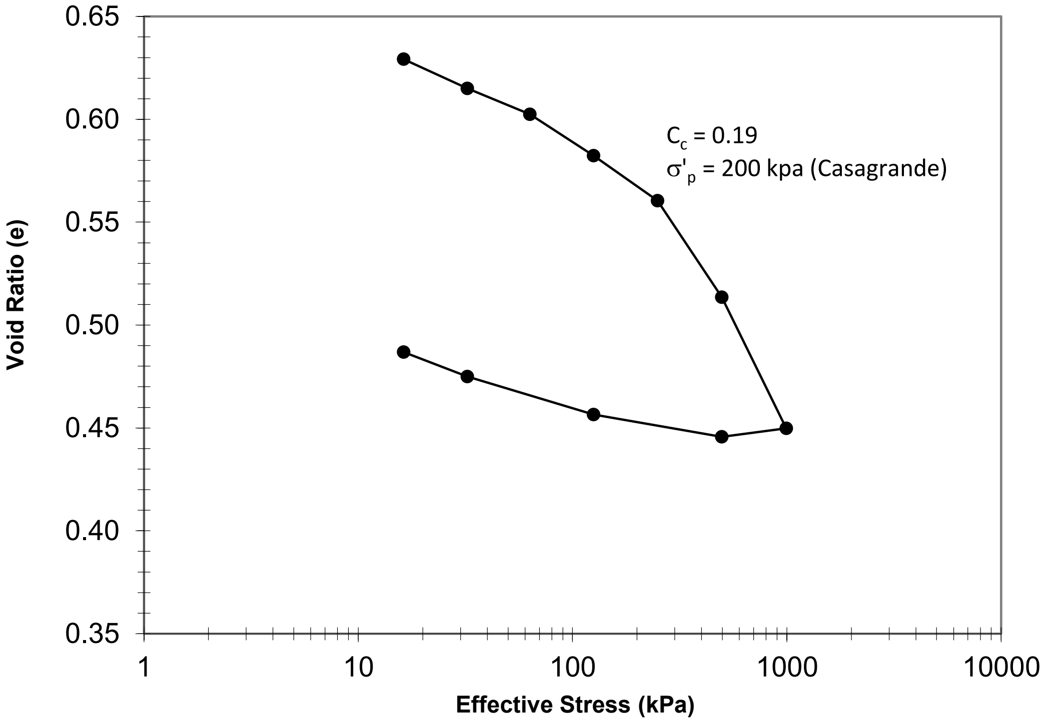
Project Name : Dicks Brook Bridge
Project Number : 163567

Sample: BH14 ST5 **Sample Depth (m):** 3.3

Sample Information

Initial Moisture Content:	46	%
Initial Dry Unit Weight:	11.7	kN/m^3
Assumed Specific Gravity	2.7	

Consolidation Test Results



Project Name : Dicks Brook Bridge
Project Number: 163567
Sample: BH14 ST9 **Sample Depth(m):** 5.5

Sample Information

Initial Moisture Content:	27	%
Initial Dry Unit Weight:	15.87	kN/m^3
Assumed Specific Gravity	2.7	

Stantec Consulting Ltd.
102-40 Highfield Park Drive, Dartmouth NS B3A 0A3

July 5, 2017
File: 121619497.140

Attention: Mr. Vince Goreham
Harbourside Geotechnical Consultants
219 Waverley Rd., Suite 200
Dartmouth, NS B2X 2C3

Dear Mr. Goreham,

Reference: Consolidated Undrained Triaxial Test on BH14 SA7 & SA3 - Dicks Brook Bridge

This letter presents the results of one consolidated undrained triaxial test carried out on the above referenced specimens in accordance with ASTM D4767, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils. Specimens were submitted to the Stantec Dartmouth Laboratory on June 15, 2017.

The test was conducted using two specimens from BH14 SA7 (150 kPa and 450 kPa applied stress), and one specimen from BH14 SA3 (300 kPa applied stress). Table 1, below, provides initial specimen moisture contents and the time required for 50% consolidation (t_{50}) for each specimen.

Table 1: Specimen Moisture Contents and t_{50}

Specimen [applied stress]	Initial Moisture Content (%)	t_{50} (minutes)
BH14 SA7 [150 kPa]	33	27
BH14 SA3 [300 kPa]	72	68
BH14 SA7 [450 kPa]	36	25

See attached plots for detailed results of the testing.

This letter provides test results only and does not purport to provide engineering interpretation, analysis, or recommendations of any kind.

Reference: Consolidated Undrained Triaxial Test on BH14 SA7 & SA3 - Dicks Brook Bridge

We trust that the information contained in this letter is adequate for your present purposes. If you have any questions about the results presented herein or if we can be of any other assistance, please contact us at your convenience.

Regards,

STANTEC CONSULTING LTD.

Heidi McKnight-Whitford, M.A.Sc., P.Eng.
Geotechnical Engineer
Phone: (902) 468-7777
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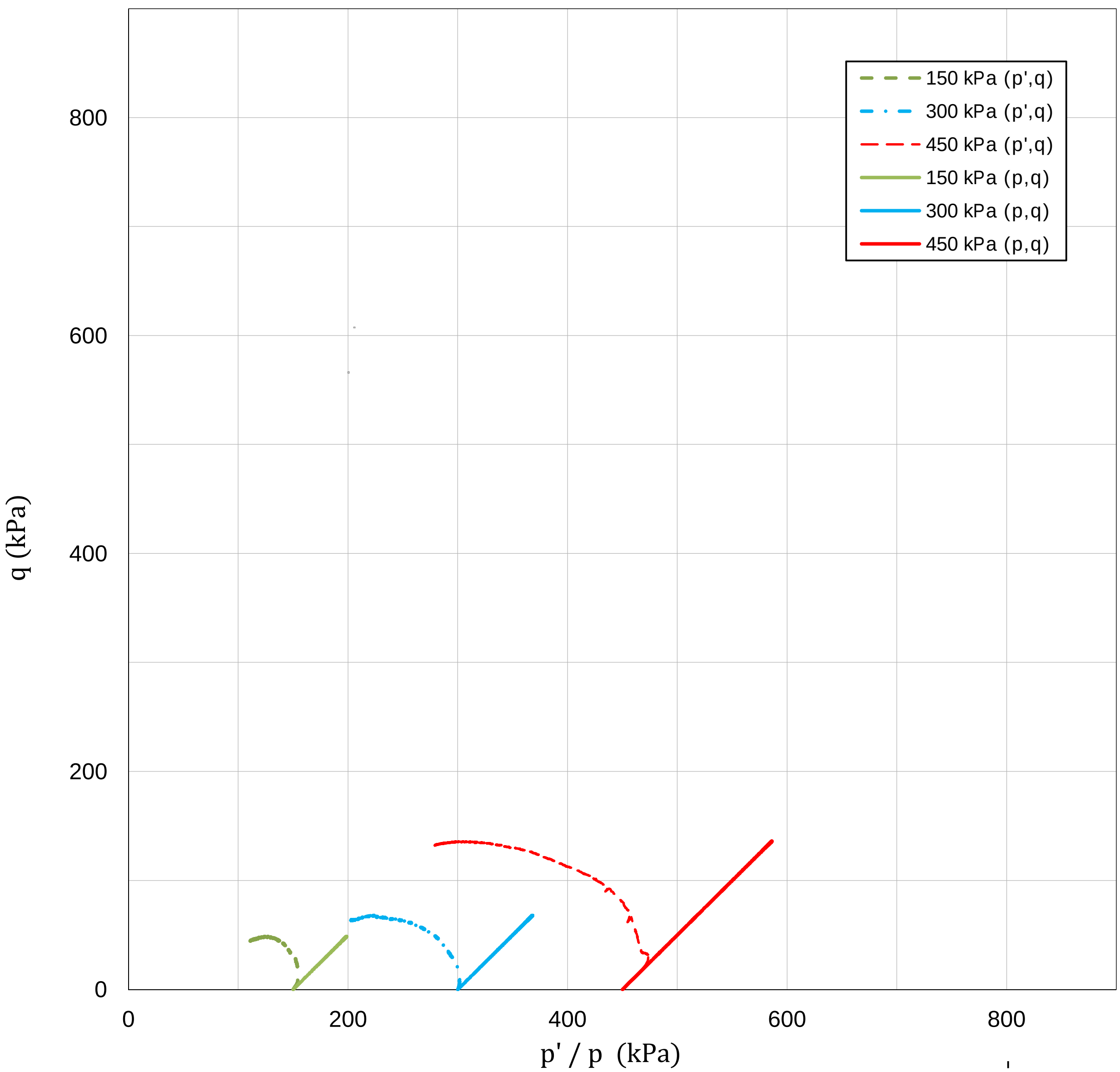
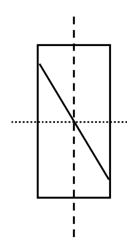
CONSOLIDATED UNDRAINED TRIAXIAL TEST



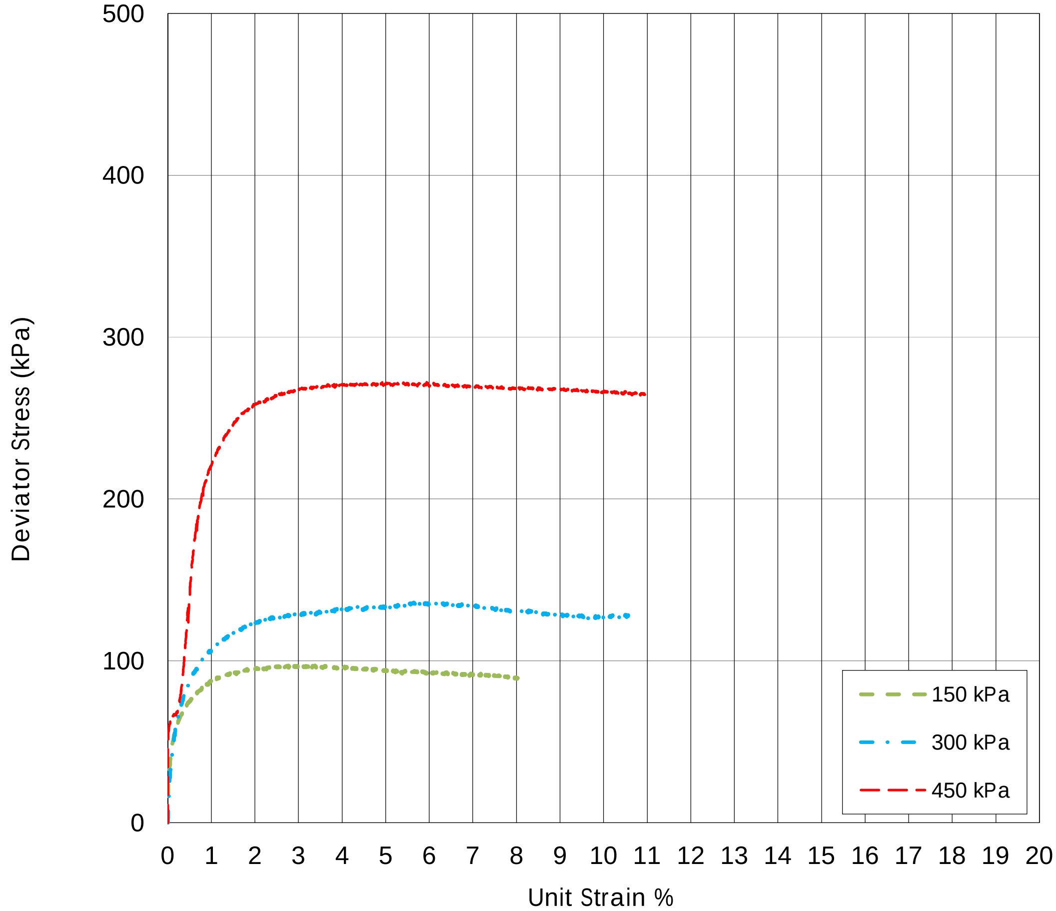


Tested By: MVG June.2017

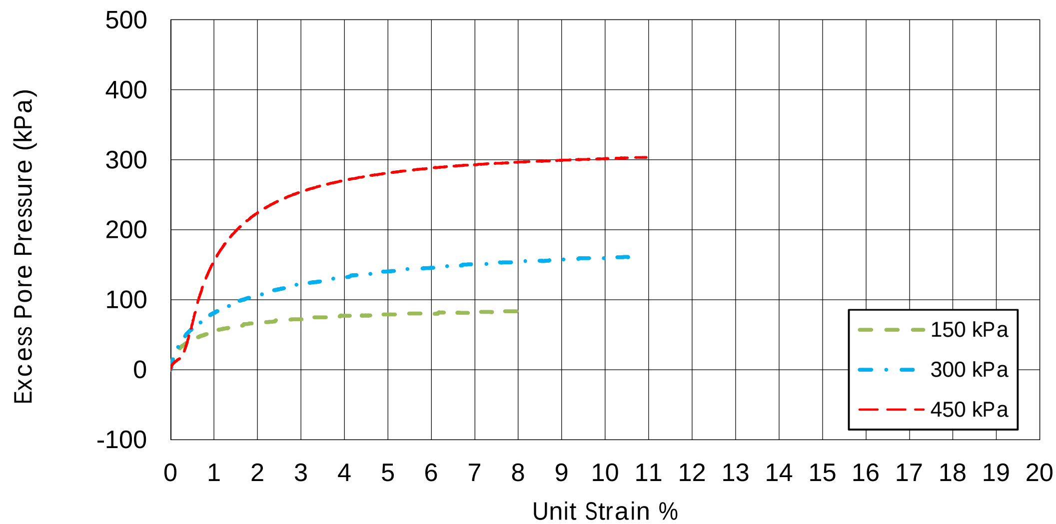
Checked By: HMW July.2017

Dicks Brook Bridge CIU Testing
BH 14 SA7/SA3
Project Number: 121619497
Stantec Consulting LTD.

CONSOLIDATED UNDRAINED TRIAXIAL TEST



Checked By: HMW July.2017



Tested By: MVG June.2017

Dicks Brook Bridge CIU Testing
BH 14 SA7/SA3
Project Number: 121619497
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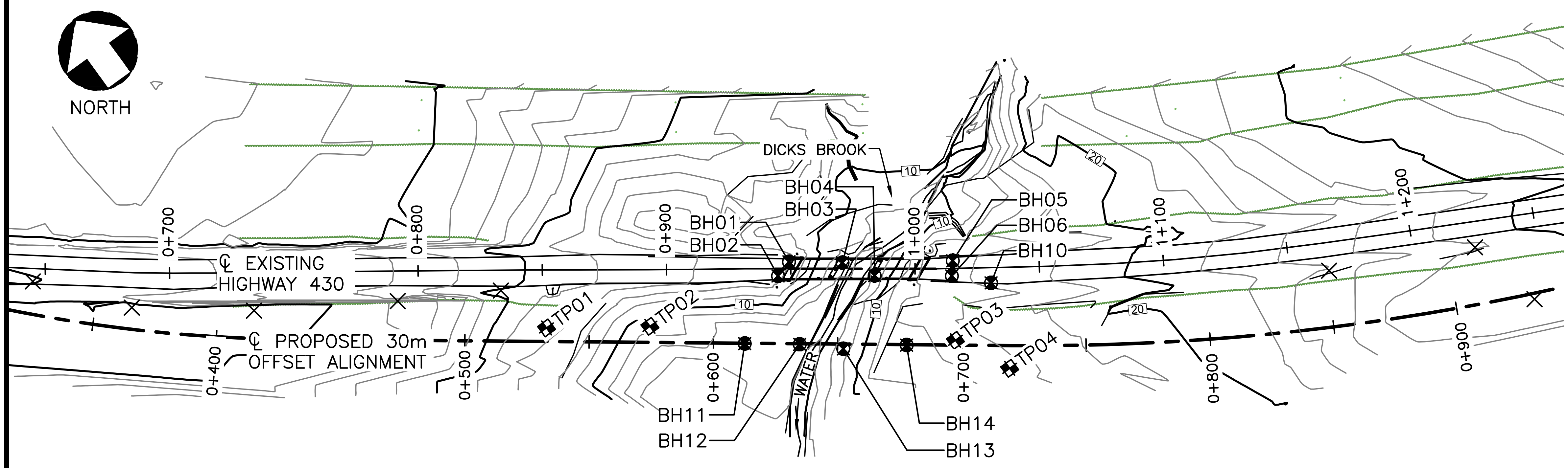
Stantec

APPENDIX C

Drawing G1 – Borehole and Test Pit Location Plan



BOREHOLE LOCATION PLAN

SCALE: 1:2000

0m 50m 100m 150m 200m 250m

BOREHOLE LOCATIONS

	NORTHINGS	EASTINGS	ELEVATION (m)
BH01	5,480,065.0	448,077.5	14.98
BH02	5,480,063.0	448,070.6	14.99
BH03	5,480,052.0	448,094.7	6.93
BH04	5,480,040.2	448,102.1	6.61
BH05	5,480,026.5	448,131.0	15.39
BH06	5,480,021.8	448,127.1	15.37
BH10	5,480,010.2	448,138.2	16.39
BH11	5,480,048.9	448,043.7	4.28
BH12	5,480,035.6	448,061.4	5.12
BH13	5,480,023.8	448,074.5	7.39
BH14	5,480,010.0	448,096.0	13.67

TEST PIT LOCATIONS

	NORTHINGS	EASTINGS	ELEVATION (m)
TP01	5,480,101	447,983	16.5
TP02	5,480,077	448,017	8.0
TP03	5,480,000	448,113	14.7
TP04	5,479,978	448,124	15.5

HARBOURSIDE
Geotechnical Consultants

HARBOURSIDE
Engineering Consultants

LEGEND:

- BH01 BOREHOLE
- TP01 TEST PIT
- BEDROCK OUTCROP
- 10 MAJOR CONTOUR ELEVATION (m)
- ℄ ROADWAY

revisions		date
A	ISSUED FOR INFORMATION	JUNE 06 2017

project / projet
DICK'S BROOK BRIDGE REPLACEMENT

GROS MORNE NATIONAL PARK

drawing / dessin
BOREHOLE AND TEST PIT LOCATION PLAN

designed / conçu: V. GOREHAM
date: JUNE 2017

drawn / dessiné: D. LARADE
date: JUNE 2017

approved / approuvé: T. MENZIES
date: JUNE 2017

Tender / Soumission

PWGSC Project Manager / Administrateur de projets TPSGC

project number / no. du projet
163567

drawing no. / no. du dessin
G1