Appendix B

Geotechnical Report

SNC·LAVALIN

SHOAL COVE BROOK BRIDGE REPLACEMENT - GROS MORNE, NL

Geotechnical Investigation

Public Works and Government Services Canada


INFRASTRUCTURE AND BUILDINGS

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1.0 INTRODUCTION

1.1 *Background & Objective*

SNC-Lavalin Inc. (SLI) has completed a geotechnical investigation for the replacement of the existing Shoal Cove Brook Bridge (the Site) with a new bridge, and realignment of the approach roadways.

SNC-Lavalin Inc. (SLI) was engaged in December of 2013 by Publics Works Government Services Canada (PWGSC) to conduct a geotechnical investigation at the Site. The objectives of the proposed work were to: (1) assess the *in situ* overburden, bedrock and groundwater conditions in the area proposed for the new bridge, and (2) provide a factual report detailing the geotechnical recommendations and parameters for site preparation, foundation design, and construction.

Qualified personnel from SLI carried out the geotechnical investigation between February 24 and March 1, 2014. The work was performed in accordance with standard industry practices.

1.2 *Site Description & Geology*

The Site is located along the shore of the east arm of Bonne Bay, in Gros Morne National Park, route 430, approximately 20 km southeast of Norris Point, NL. The parcel of land on which the investigation took place was approximately 660 m^2 and was dominated by riparian habitat, along with thick stands of alders near the road. There were no existing structures present other than the existing bridge and approach roadways.

The existing bridge consists of a concrete superstructure supported by a concrete substructure founded on spread footings. The span of the existing structure is 11 m long. It is approximately 9 m in overall width, and accommodates two lanes of traffic.

Bedrock in the area reportedly belongs to the Mackenzie Mill member of the Forteau Formation (northwest side of Mill Brook), and the Hawke Bay Formation (southeast side of Mill Brook), both of which belong to the Labrador Group (Knight, 2013). Generally speaking, these are shelf and foreland basin rocks of the Humber Zone parautochthon, east of the Humber Arm, which are actually separated by a normal fault, which generally follows the path of Mill Brook upstream for approximately 600 m. More specifically, the Mackenzie Mill member rocks are composed of calcareous black and dark grey shale, calcareous ribbon-bedded siltstone and sandstone, current-bedded and bioturbated sandstone and nodular to lumpy, fine-grained to skeletal-rich limestone (Knight, 2009). The Hawke Bay Formation rocks are composed of dark grey mudstone and dark grey, micaceous sandstone alternating with equally thick units of quartz arenite. These rocks were formed in the Late Proterozoic – Cambrian Period (Colman-Sadd, 1990). The terrain in which these rocks are found is divided by a major north-striking thrust fault

(Knight 2013). The Forteau Formation rests conformably upon the Bradore Formation and is conformably overlain by the Hawke Bay Formation, all of which belong to the Labrador Group (Knight, 2013).

Surficial geology in the area has been reportedly classified as bedrock concealed by vegetation mat, developed on either colluvial surfaces or a thin layer of angular frost-shattered and frost-heaved rock fragments overlying bedrock. It includes areas of shallow (less than 1 m) overburden (Kirby *et al.*, 2010). Field observations and photographs do not conform to this description. The overburden is indeed a glacial till (albeit overlain by approximately 0.9 m – 2.0 m of fill material presumably from the original bridge and roadway construction); however, it extends down to a depth range of approximately 10.6 m to greater than 20.8 m. It is composed of varying mixtures of grain sizes, with gravel and sand typically accounting for the majority, and a significant distribution of cobble and boulder sized clasts or erratics. There were also some silt deposits encountered at depths ranging from approximately 6.2 metres below ground surface (mbgs) to roughly 7.7 mbgs on either side of the brook. During this investigation the original ground surface was disturbed and reworked as a result of site preparation activities, and every effort was made to return the site conditions back to original upon completion of the field investigation.

1.3 *Health, Safety, Environment*

A Health and Safety Plan (HSP) was written by SLI and approved by PWGSC in January 2014. The HSP provided project specific descriptions of health risks and safety hazards. In addition, risk management strategies, responsibility delegation, general safety rules, and activity specific protective equipment requirements were explained in detail. Emergency measures and communication procedures for a number of different potential hazards were also explicitly listed. In addition to this, a Basic Impact Analysis (BIA) was completed by Parks Canada and given to SLI, which identified several valued ecological components that needed to be safe guarded at all times during the investigation.

Prior to initial mobilization, a safe work plan/job safety analysis for the geotechnical investigation was created and acknowledged by the field personnel completing the work. General hazards associated with the field work, as well as site specific potential hazards were identified and assigned a risk level and appropriate mitigation measures were identified. A list of emergency contact numbers was also prepared and carried by the field team. A vehicle (light duty) pre-operational safety check was performed prior to the start of the field work. Additionally, each member of the field team, including subcontractors, were required to wear appropriate PPE at all times while on site.

2.0 METHODOLOGY

2.1 *Fieldwork Procedures*

The fieldwork for the investigation was carried out by qualified SLI personnel between February 24, 2014 – March 1, 2014, at which time three (3) boreholes were drilled and two (2) test pits were excavated at the approximate locations shown on the drawing and figure presented in **Appendix A**. For clarification purposes it should be noted that although the bridge is called Shoal Cove Brook Bridge, the waterway that it crosses is in fact called Mill Brook. The drilling investigation was carried out using a CME-55 drill rig, and the test pitting investigation was carried out using a CAT 315CL, both of which were supplied by CABO Drilling Corp from Springdale, NL.

The site investigation was supervised by SLI, who were also responsible for logging the subsurface conditions in the field. The boreholes were advanced using continuous flight augers and NW casing, with field sampling and testing performed in the open boreholes. Standard Penetration Tests (SPT) were performed in two of the three boreholes to obtain soil blow counts (i.e. N-values) using a 50 mm O.D. split spoon sampler. It is important to note that due to the frequency of boulders and cobbles encountered during the drilling process, it was necessary to discontinue Standard Penetration Testing after a certain point in each borehole. Material was not able to enter the split spoon sampler due to jamming and this caused a great deal of equipment failure and/or breakage. Once sufficient data regarding the overburden was collected at each location, emphasis was shifted towards advancing the boreholes to reach bedrock. In one instance, sufficient material was collected within the split spoon sampler and was stored in a waterproof bag for further laboratory testing. When encountered bedrock was drilled using NQ-sized (i.e. 47.6 mm dia.) coring equipment. All samples were sent for classification and testing to SLI's in-house geotechnical laboratory in Mount Pearl, NL.

The test pits were excavated adjacent to borehole locations in order to obtain relevant and comparable data relating to the overburden. Each test pit was excavated to the point of refusal; in both cases this was due to impenetrable boulders being encountered. Both test pits were subsequently backfilled upon completion of geotechnical observations. Soil descriptions were made in the field and in accordance with the Canadian Engineering Foundation Manual, 4th Edition. The compactness condition of the *in situ* soils was estimated based on the resistance of the soil to excavation and the encountered soils were also described with respect to gradation, compactness, weathering, colour, and inferred moisture content. The depth of the groundwater pool at the bottom of each pit, as well as depth to first groundwater encountered, were recorded prior to backfilling the test pit.

The borehole locations were selected by SLI in the field and the locations of all test pits and boreholes were recorded once selected using a hand held G.P.S. unit and referenced to the NAD 83 map datum and recorded in the UTM coordinate system. The general location of the

Site and the locations of the boreholes and test pits are presented on the drawing and figure in **Appendix A**. Borehole logs are presented in **Appendix B** and detailed test pit logs are presented in **Appendix C**.

2.2 *Laboratory Testing*

Soil samples were processed in SLI's geotechnical laboratory in Mount Pearl, NL. The following tests were carried out:

- Gradation analysis on three (3) soil samples to further classify the soil strata and grain size distribution.

It should be noted that due to the fact that bedrock was encountered at a depth of 12.6 m and was only encountered on one side of the stream, it is highly unlikely that the foundation for the proposed bridge will be founded on bedrock. As such, no laboratory testing on the bedrock core was performed.

The results of all geotechnical laboratory tests have been included in **Appendix D.**

3.0 SUBSURFACE CONDITIONS

The soil stratigraphy and boundaries between the strata indicated on the test pit logs were inferred from field observations. These boundaries generally represent a transition from one material type to another and may not necessarily represent exact surfaces of geological change.

In general, the soil profiles throughout the Site consisted of topsoil, fill and unaltered glacial till overlying bedrock. A summary table of the subsurface conditions at each test pit and borehole location is provided in **Table 1**.

Table 1: Summary of subsurface stratigraphy and conditions.

Excavation ID	Coordinates UTM NAD 83 Zone 21		Depth to Bedrock (mbgs)	Depth to Refusal (mbgs)	End of Borehole (mbgs)	Depth to Groundwater (mbgs)
	Easting	*Northing*				
BH-SC-002-2014	441347 E	5484650 N	N.E.	N/A	20.8 m	0.7 m
BH-SC-003-2014	441358 E	5484626 N	N.E.	N/A	10.6 m	1.3 m
BH-SC-004-2014	441368 E	5484630 N	12.6 m	N/A	14.7 m	N.R.
TP-SC-001-2014	441363 E	5484626 N	N.E.	1.1 m	N/A	0.6 m
TP-SC-002-2014	441340 E	5484632 N	N.E	1.9 m	N/A	0.9 m

Note: BH = Borehole. TP = Test Pit. N/A = not applicable. N.E. = not encountered. N.R. = not recorded.

The subsurface conditions encountered during the investigations are presented below. All descriptions are in accordance with the Canadian Foundation Engineering Manual, 4th Edition, and were made at the time of the field investigation.

3.1 *Boreholes*

Adjacent to Mill Brook and on the upstream side of Shoal Cove Brook Bridge, three (3) boreholes were drilled (BH-SC-002-2014 through BH-SC-004-2014). The boreholes revealed three main material types: fill, glacial till, and, in one case, bedrock. Groundwater was present in all three (3) of the boreholes, ranging in depth from 0.7 mbgs to 1.3 mbgs. All boreholes were located in a recently cleared area that was partially riparian habitat and partially re-vegetated with alders near the upstream toe of the slope. Boreholes BH-SC-003-2014 and BH-SC-004-2014 were located according to plan; however, the location of borehole BH-SC-002-2014 was adjusted due to site conditions, and BH-SC-001-2014 was removed from the drilling program. BH-SC-004-2014 was drilled without completing any SPT or collecting any samples in order to try and determine the depth to bedrock on at least one side of the brook. It is also important to note that the environmental concerns that arose during the course of the field investigation hampered the borehole program, in addition to the problems caused due to the time of year. For example, during active drilling, in the case of all boreholes, minor siltation issues were noted in Mill Brook, which were deemed unacceptable by Parks Canada representatives on site due to the presence of sensitive mussel bed habitat downstream. Several avenues were explored in order to combat this issue, such as siltation fences and the use of drilling mud to decrease hydraulic conductivity between the boreholes and Mill Brook. However, due to extreme weather, difficult site conditions, and the environmental risk of potentially introducing drilling mud into a sensitive habitat, the best solution was to cease drilling periodically to reduce the occurrence of siltation as much as possible. With the three boreholes that were drilled, sufficient data was collected on the subsurface conditions, which are described in detail below.

3.1.1 Fill

The ground surface was frozen and covered in snow and ice, which made it difficult to characterize the topmost portion of the fill layer and to record an accurate elevation for the collar of the borehole. Fill material was confirmed in one (1) of the boreholes (BH-SC-003-2014), beginning at the surface, and extending to a depth of approximately 2.0 mbgs. Fill material was inferred to be present in the remaining two (2) boreholes (BH-SC-002-2014 and BH-SC-004-2014) based on the observations recorded during the drilling, particularly any colour change in the water returning to the surface within the first couple of metres of drilling. However, due to limited sample collection resulting from cobbles and boulders blocking the opening of the split spoon, it was not possible to fully confirm the presence of fill in these two boreholes. Where the fill material was actually recovered in the spilt spoon, it was dark brown, wet, compact, sandy gravel with some cobbles and boulders and a trace amount of silt. Gravel-sized clasts collected within the split spoon sampler were angular to sub-angular; indicating that the material was not likely transported naturally and therefore was probably imported during construction of the original bridge. On each side of the brook a test pit was also excavated adjacent to the borehole location(s), therefore making it possible to provide additional comment on the possible stratigraphic divisions within the first couple of metres of the ground surface. For a discussion of the strata encountered in test pits please refer to section 3.2.

3.1.2 Till

The presence of glacial till was observed in all three (3) boreholes. The thickness of the till layer varied; it was observed as beginning approximately at a depth of 0.9 mbgs to 2.0 mbgs and extending down to a depth of 12.6 mbgs to greater than 20.8 mbgs. The till was observed as varying shades of brown and grey, wet, and compact to very dense. The grain size distribution exhibited varying proportions of gravel, sand and silt, but typically always contained cobble and boulder sized clasts. The clasts within the glacial till that were collected within the split spoon were well rounded, indicating significant erosion due to being transported, i.e., by glacial action. Glacial erratics were encountered throughout the layer and small pieces that fractured off and became wedged in the split spoon sampler revealed that the composition of the erratics was variable.

Silt Layer

In two (2) of the boreholes (BH-SC-002-2014 and BH-SC-003-2014), layers of gravelly to clayey silt were encountered. The thickness of these layers could not be accurately defined; however, it was inferred that each silt layer was approximately 0.3 m thick. In BH-SC-002-2014, the fine-grained material was first observed at a depth of approximately 6.2 mbgs, with a second layer observed at a depth of roughly 7.3 mbgs. In BH-SC-003-2014, a single layer was observed at a depth of approximately 7.7 mbgs. The fact that a layer of silt was encountered on either side of the brook and roughly the same depth suggests that the silt layers may have lateral continuity across the brook and may exist as a distinct layer within the glacial till.

In BH-SC-003-2014 the unconfined compressive strength of this material was tested using a soil penetrometer and was recorded as 4.5 kg/cm^2, which is approximately 440 kPa, indicating that this material is Hard according to Bowles, 1996. The corrected N_{70} value was 23 for this interval, indicating that the material is Very Stiff, which generally agrees well with the classification presented based on the soil penetrometer.

Within BH-SC-002-2014 the material encountered at 6.2 mbgs had a corrected N_{70} value of 11, indicating that the material is Stiff according Bowles, 1996. The unconfined compressive strength can therefore be estimated as 115 kPa (Bowles, 1996). The second silt layer encountered within the borehole was less compact than the first; however this cannot be readily seen on the borehole logs because the N value reported is elevated due the presence of a boulder within the second test interval. Using the blow counts recorded during the seating interval (which is generally not recommended due to the potential for disturbance within this interval from the drilling process) and the first test interval, the corrected N_{70} value is 4, indicating that the material is Soft and has an unconfined compressive strength of roughly 40 kPa (Bowles, 1996).

The range of unconfined compressive strengths and compactness condition for the silt layers is variable. However, due to the depth at which the silt layers were encountered, this material is not expected to have any negative effect on the performance of the footings for the proposed bridge. This is based on the assumed contact pressure that the footing will transfer to the soil, the depth of the footings and size of footings as discussed in **Section 4.0**. If the assumptions made in that section do not remain valid due to changes in the design, etc., then the effect that the silt layer may have on the performance of the footings must be re-evaluated.

3.1.3 Bedrock

Bedrock was encountered in only one (1) of the boreholes, BH-SC-004-2014, beginning at a depth of 12.6 mbgs. The borehole was drilled down to a maximum depth of 14.7 m, and based on the total amount of recovered core (2.1 m) and consistent lithology; it is SLI's opinion that bedrock was proven rather than a large boulder. The bedrock is a very fine-grained quartz arenite (orthoquartzite), which agrees with the fieldwork and mapping published by Knight in 2013. The Rock Quality Designation (RQD) was calculated for the only full core run that was drilled and the RQD was 56%, which can be classified as Fair quality rock; the percent recovery for the core run was 93%. Photographs of the recovered core are presented on the following pages.

Core Photograph (wet): BH-SC-003-2014, 12.6 mbgs – 14.7 mbgs.

Core Photograph (dry): BH-SC-003-2014, 12.6 mbgs – 14.7 mbgs.

3.2 *Test Pits*

Two (2) test pits were excavated in order to supplement the borehole program, since sampling within the boreholes was very difficult, due to the time of year and environmental constraints, as previously discussed. Test pit TP-SC-001-2014 was excavated at the midpoint between boreholes BH-SC-003-2014 and BH-SC-004-2014, and test pit TP-SC-002-2014 was excavated adjacent to the upstream toe of the slope associated with the current Shoal Cove Brook Bridge. Both test pits were located in a recently cleared area that was partially riparian habitat and partially re-vegetated with alders. Three material types were encountered, and both test pits were terminated due to refusal on large boulders.

3.2.1 Topsoil

Topsoil was encountered in both test pits with a thickness of approximately 0.1 m. It contained woody root material and organics, and in TP-SC-002-2014, a strong odour of organics was observed.

3.2.2 Fill

The fill material encountered within both test pits was dark brown, wet, and compact. In TP-SC-001-2014, it was classified as a cobbley, bouldery gravel, with some sand and trace silt. In TP-SC-002-2014, it was classified as gravelly, sandy boulders with some cobble and trace silt. The thickness of this layer ranged between 0.8 and 1.0 m, extending down to a maximum depth of 1.1 mbgs. The clasts of fill material observed and sampled were angular to sub-angular, indicating that the material was not likely transported naturally and therefore was probably imported during construction of the original bridge and roadway.

3.2.3 Till

The presence of glacial till was observed in TP-SC-002-2014 only, beginning at a depth of 0.9 mbgs and extending to the base of the pit at 1.9 mbgs at which point the excavation was terminated due to refusal on boulders. The till was observed as brown, wet, compact, cobbley gravel with some sand, some boulder and trace silt. The clasts within the till were well rounded, indicating significant erosion due to being transported, i.e., by glacial action.

3.3 *Groundwater*

Groundwater was encountered in all three (3) of the boreholes and the two (2) test pits. The depth to groundwater ranged between 0.6 to 1.3 mbgs. Groundwater levels can be expected to fluctuate during periods of heavy precipitation associated with seasonal weather trends, or particular precipitation events, site use, adjacent site use, and construction activities. In addition, due to the proximity of the Site to Mill Book it may be assumed that the groundwater table will reach the ground surface at certain times of the year, e.g., spring. There is also a low-lying and very wet area in the vicinity of BH-SC-002-2014 and the groundwater table in this area is likely close to or at ground surface for most of the year.

4.0 DISCUSSION AND RECOMMENDATIONS

Details regarding site development and the foundation plan were available at the time of report preparation. However, many of the details provided were at the Issued for Review stage and therefore may be subject to change. As such, only preliminary geotechnical design recommendations have been provided herein and are for general planning purposes only. We strongly suggest that the comments and recommendations presented herein be reviewed by SLI once the site development details have been finalized. For the purpose of discussion, and in order to provide recommendations, it has been assumed that the proposed development will be a single span, portal frame bridge. The abutments for the bridge will be strip footings with a mud slab incorporated into the footing design. The least plan dimension of the footing/mud slab will be 2.7 m.

4.1 *Site Preparation*

The ground within the footprint of the proposed bridge has already undergone some development during the construction of the original bridge. The encountered fill contained 25 % to 40% boulders (>200 mm). The amount of fill encountered throughout the footprint of the proposed bridge ranged from 0.8 m to 2.0 m. The source and quantity of the fill material that was imported to site, as well as the method of placement are unknown to SLI at this time. However, based on the presence of large boulders, some of which were as large as 600 mm, it doesn't appear that any sort of formal compaction techniques were employed in this area. It is assumed that the fill material was brought to site during construction of the existing embankments.

Within the footprint of the proposed bridge the depth to bedrock was recorded as 12.6 m and this was confirmed through drilling. The average depth to groundwater was roughly 1.0 m. Groundwater is expected to be an issue during site preparation and throughout construction. Any seepage into excavations during construction should be controlled to prevent softening of *in situ* soils. Placement of an approved structural fill is not anticipated to be required underneath the strip footings/mud slab; however, seepage into the excavations must be controlled during construction. If it is not, it could result in excessive total and/or differential settlements of structures. The following are the major considerations related to site preparation.

4.1.1 Weak Soil Stripping

For the strip footings/mud slab any loose and/or weathered till and fill should be removed to expose undisturbed and competent native soils or bedrock. If bedrock is encountered, which is highly unlikely, all highly weathered and fractured bedrock must be removed from within the footprint of the footings/mud slab. It is also recommended, that the site soils in the areas where the test pits were located be re-excavated and properly compacted during construction, especially if the test pit is located in a load bearing area.

4.1.2 Bearing Surface

Where the encountered bearing surface is competent, i.e., compact, unaltered glacial till, it should be proof rolled using a 10 - 12 tonne roller, or heavy plate, vibrating tamper, i.e., after stripping of all deleterious/weak soil and before placement of any structural fill, if required due to the unsuitability of *in situ* soils. A test strip can be isolated in order to determine the minimum number of passes required to fully compact the bearing surface. Areas softened by water, disturbed by construction activity or in a loose condition should be further excavated to a suitable depth and replaced with an approved structural fill. If bedrock is encountered before the underside of the footings/mud slab is reached, then all highly weathered and fractured rock must be removed down to competent bedrock and the bedrock surface must be inspected by a qualified geotechnical professional.

4.1.3 Structural Fill

Structural fill should be a clean (generally less than 10% fines), well-graded, free draining, granular soil or processed blast rock free of deleterious material. The use of blast rock fill is recommended in areas such as excavation bases where wet conditions are encountered. While the results of the laboratory testing indicated the fill material present throughout the Site contains less than 10% fines, the test pit investigation has shown that the majority of the existing site fill contains large boulders and therefore is generally unsuitable for reuse as a structural fill. This same comment applies to the glacial till as it is known from the borehole program that large glacial erratics (>200 mm) are common within the till. In order to reuse the till it would be required that all boulders be either removed prior to compaction or broken up using a hydraulic hammer.

Granular Fill

Soft spots and/or loose *in situ* soils requiring structural fill and/or areas requiring the placement of structural fill in order to reach the underside of footings/mud slab should be placed in lifts and compacted to the specifications outlined in **Table 4.1**. The maximum particle size for the structural fill should be restricted to 200 mm. Particles with diameters greater than 200 mm should be removed or broken prior to placement of the loose lifts. The structural fill should be free of deleterious materials and the frost susceptibility of the material must be evaluated prior to its use. The loose lift thickness used during placement of the structural fill should be compatible with the type of equipment used to ensure that the required density is achieved throughout the lift and in general should not exceed 300 mm.

Optimum roller passes (10 - 12 tonne vibratory roller) can be determined from surveyed settlement versus roller pass curves or correlated with the number of roller passes required to achieve the recommended percent compaction. Each roller path should overlap the edge of the preceding path by approximately 10%. Should space restraints prevent the use of a 10 -12 tonne vibratory roller, loose lift thickness should be reduced to a maximum thickness of 200 mm, depending on the size of the tamper, and be compacted using a heavy plate, vibrating, tamper to obtain required compaction results.

If the structural fill is not amenable to nuclear densometer testing, then the surveyed settlement versus roller pass curves can be used, along with the amount of visual deflection present after each pass of the 10 - 12 tonne vibratory roller, to properly evaluate and certify the compaction condition of the granular fill.

Table 4.1. Summary of recommended compaction requirements.

Structural Granular Fill Application	Compaction Requirements Percent of Standard Proctor Maximum Dry Density (ASTM D698)
Foundation Bases	100
Roads	95
General Backfill	95

Processed Blast Rockfill

If a processed blast rockfill is used as a structural fill and the footings/mud slab are to be founded on the blast rockfill, then we offer the following recommendations:

- Loose lift thicknesses should generally not exceed 450 mm. The blast rockfill should come from an approved quarry source, be well graded, free of deleterious and friable material, and should contain a low percentage of fines. The maximum particle size should not exceed 300 mm.
- Optimum roller passes (10 - 12 tonne vibratory roller) can be determined from surveyed settlement versus roller pass curves; however, as a general rule the number of roller passes should be limited to approximately 6 passes, in order to prevent crushing of the rockfill surface and the subsequent generation of additional fines. Each roller path should overlap the edge of the preceding path by approximately 10%. Should space restraints prevent the use of a 10 -12 tonne vibratory roller, loose lift thickness should be reduced to a maximum thickness of 300 mm and be compacted using a heavy plate, vibrating, tamper to obtain required compaction results.
- The surveyed settlement versus roller pass curves should also be backed up by observing the amount of visual deflection present after each pass of the 10 - 12 tonne vibratory roller. Using both techniques will allow the compaction condition of the blast rock fill to be properly evaluated and certified.

4.1.4 General Backfill

Backfill material used around the abutments must be free of deleterious material, free-draining and the frost susceptibility of the material should be evaluated before its use to reduce the potential of adfreeze effects. It should be noted that many Provincial (Newfoundland) Government Specifications indicate that material with a fines (< 0.075 mm) content less than 20% can be considered non frost susceptible. However, this is not necessarily the case. The determination of a material's frost susceptibility needs to be made based on several factors and should be made on a case by case basis by a qualified geotechnical professional. The backfill should be capped with less permeable soil and a surface grade provided to shed runoff before it enters the backfill. To limit horizontal earth pressures on the bridge abutments during compaction, larger compaction equipment should not be used within 1.5 m of the abutments and the zone should be compacted using hand operated or walk-behind equipment.

4.2 *Foundation Design*

Conventional shallow foundations, e.g., strip footings, are suitable for the proposed development provided that all weathered and/or loose soils are removed from within the development area associated with the footings/mud slab and that all fractured and highly weathered bedrock, if encountered, is also removed down to competent bedrock and the bedrock surface is inspected and approved by a qualified geotechnical professional.

4.2.1 Shallow Footings on Granular Soils or Processed Blast Rock

Based on the test pit excavations and drilling program it is apparent that the majority of the till is in a compact condition. It should be noted that some of the recorded N values from the SPT are elevated due to the presence of cobbles and boulders and extreme care must be taken when interpreting the results presented on the borehole logs. In order to achieve the bearing capacities cited below, it is recommended that all existing fill material and weathered till be removed from within the footprint of the abutments. If bedrock is encountered, then all highly weathered and fractured bedrock must be removed until competent bedrock has been reached. Prior to the placement of any structural fill, the exposed surface of the excavation, if granular soils are present, should be proof rolled using a 10 - 12 tonne vibratory roller or a heavy plate, vibrating, tamper if space restrictions exists. Additionally, the strip footings should have a minimum soil cover of 1.4 metres or equivalent insulation for frost protection, if founded on granular soils. Riprap is generally not acceptable as soil cover to protect against frost penetration due to its uniform size and this must be accounted for by incorporating a sufficient thickness of filter material underneath the rip rap and/or or adequate insulation into the design.

Bearing capacity calculations with respect to Ultimate Limit State (ULS) for the *in situ* soils were performed using Terzaghi's bearing capacity equation for general shear failure as presented in Bowles, 1996. The gross ultimate bearing capacity obtained from the equation was then reduced using a geotechnical resistance factor of 0.5 (Canadian Foundation Engineering Manual 4th Edition). The Serviceability Limit State (SLS) for the *in situ* glacial till was based on settlement, and it was assumed that the total allowable settlement is not to exceed 25 mm. As a general rule, limiting the total settlement to 25 mm will ensure that differential settlement does not exceed 19 mm. The allowable bearing pressure with respect to settlement was determined using the design charts presented in Peck *et al.*, 1974 and a geotechnical resistance factor was not applied to the allowable bearing pressure obtained from the charts.

The minimum dimension/least plan dimension of the strip footings/mud slab, which will be used in the following discussion, was selected based on preliminary information available at the time of report preparation, which indicates that the least plan dimension is 2.7 m and the following discussion on ULS has been prepared based on this caveat. It is should also be mentioned that the surcharge term has been neglected in the equation due to the proximity of the brook in relation to the outside edge of the footings/mud slab and the general lack of soil cover in this area.

Ultimate Limit States

In order to apply the Terzaghi equation several soil parameters needed to be estimated including the unit weight of the soil and the angle of internal friction and these values are presented in the following paragraph.

Based on the results of the geotechnical investigation groundwater was encountered close to the existing grade and therefore is expected to be above the underside of the footings/mud slab. As such, the unit weight of soil used in the Terzaghi equation has been based on a moist, well graded, compact glacial till and a submerged, well graded, compact glacial till. The sieve results support the classification of the glacial till as well graded and the results of the geotechnical investigation indicated that the majority of the existing site soils are in a compact condition, based on uncorrected N values. The wet unit weight of compact, well graded sand and gravel was estimated to be 19.2 kN/m^3 and the buoyant unit weight was estimated at 11.6 kN/m^3. The angle of internal friction has been estimated at 34° (Hough, 1969).

Using the above soil parameters and foundation conditions, the ultimate bearing capacity is approximately 520 kPa. Using the geotechnical resistance factor of 0.5, the factored geotechnical resistance at ULS is approximately **260 kPa**.

Serviceability Limit States

The compactness condition of the glacial till has been based on the results of the SPT. In the absence of laboratory derived geotechnical design parameters, the design charts presented in Peck *et al.*, 1974 have been used to provide an estimate of the net allowable bearing pressure for the glacial till. In order to do this, a SPT N value has been assigned for the glacial till based on the results of the SPT. The N values within the zone of major stressing, which according to Bowles, 1996, is from about one half the footing width (B) above the estimated base location to a depth of about 2B below, were examined. N values, which based on the field notes, were noted as being abnormally high due to the presence of cobbles and/or boulders were given appropriate consideration as were the N values near the estimated elevation for the underside of the footing/mud slab. Based on the information available at the time of report preparation, it appears that the underside of the footings/mud slab will be approximately 1.6 m below the elevation of the brook, which has been assigned an elevation of 0.0 m. After careful evaluation of the borehole logs an N value of 20 was selected for use in the design charts. However, before the design charts can be used the N value needs to be converted to a N_{60} value in order to allow for a direct comparison with the design charts. The conversion was done in accordance with Bowles, 1996, and the N_{60} value obtained was 43, indicating that the compactness condition of the soil that will be directly underneath the footing/mud slab is Dense according to the Canadian Foundation Manual, 4th Edition.

Using the design charts provided in Peck *et al.*, 1974, the net allowable bearing pressure is approximately 450 kPa. Groundwater is known to be present within the surficial soils, and will likely reach the ground surface on occasion, therefore a correction factor needs to be applied to the allowable bearing capacity previously cited (Peck *et al.*, 1974). The net allowable bearing pressure, with the groundwater correction applied is **225 kPa**. The use of a geotechnical resistance factor to further reduce the allowable bearing pressure is not warranted when using this method.

The bearing resistance is based on the assumption that all existing fill material and weathered till will be removed from within the footprint of the bridge foundations until undisturbed and competent native soils or bedrock is encountered and that the underside of the footings/mud slab is at the approximate elevation as described above. Any structural fills, if required, to reach the elevation for the underside of the footing must be placed for a suitable distance beyond the outside edge of the footings to permit access of compaction equipment and to ensure that proper compaction of the structural fill can occur. There are no major issues with the bearing capacity of the glacial till, therefore further comment on bearing splay is not required. Any structural fill must be properly compacted as previously recommended.

4.2.2 Shallow Footings on Bedrock

The results of the geotechnical investigation have shown that bedrock will, in all likelihood, not be encountered. Therefore no further comment has been provided.

4.3 *Foundation Drainage*

Requirements for long term or permanent drainage control around and below the proposed structure will depend on the details of the site development and the finished site grades.

4.4 *Temporary Excavations*

Excavation work must conform to the regulations of the Occupational Health and Safety Act of the Province of Newfoundland and Labrador at all times and we recommend that a qualified professional review all proposed excavation slopes. In addition, we recommend that excavation sides be carefully monitored and, if necessary, the contractor should slope excavation sides appropriately or use adequate bracing. This may also require review by a qualified professional. Additional measures may be required if excavations extend below the water table, which is known to be the case for this site.

4.5 *Inspection and Testing*

We recommend that qualified geotechnical personnel carry out an inspection and testing program during earthworks, structural fill and backfill placement, and foundation construction. The program should include verification of excavation bases and approval before placement of structural fills; verification of the type of structural fill, including potential for frost susceptibility, and compaction testing during structural fill placement, if applicable; structural pad certification; founding level inspections, e.g., approvals for footings; backfill certification; and laboratory testing as required, e.g., standard Proctors.

5.0 CLOSURE

Subsurface descriptions and statements regarding their condition are based on the site conditions encountered and observations made by SNC Lavalin Inc. at the time of the investigation as reported herein. Conditions between and beyond the test locations may differ from those encountered at the test locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. Extrapolation of *in situ* conditions can only be made to a limited extent and it is recommended that the Geotechnical Consultant of record be retained during preparation of the subgrade surface and during construction to ensure that the assumptions and geotechnical recommendations presented herein are consistent with the subsurface conditions encountered at the time of construction and that the recommendations contained herein remain valid.

This report has been prepared exclusively for Public Works and Government Services Canada (the Client) and their agent (SNC-Lavalin Inc.). The quality of information, conclusions and estimates contained herein is consistent with the level of effort involved in SNC-Lavalin Inc.'s services and based on: i) information available at the time of preparation, ii) data supplied by outside sources, and iii) the assumptions, conditions and qualifications set forth in this report. Unless expressly stated otherwise, any assumptions, data and information supplied by, or gathered from other sources (including the Client, other consultants, testing laboratories, etc.) upon which SNC-Lavalin Inc.'s opinion as set out herein is based has not been verified by SNC-Lavalin Inc. SNC-Lavalin Inc. makes no representation as to its accuracy and disclaims all liability with respect thereto.

This report is intended to be used by Public Works and Government Services Canada subject to the terms and conditions of its contract with SNC-Lavalin Inc. and may not be used by a third party without the express written consent of SNC-Lavalin Inc. and the Client. Any other use of, or reliance on, this report by a third party is at that party's sole risk and SNC-Lavalin Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

Preparation of this report, and all associated work, has been carried out in accordance with the normally accepted standard of care in the province of execution for the specific professional service provided to the Client. No other warranty, expressed or implied, is made.

We trust this report meets with your current requirements. Should additional information be required, please do not hesitate to contact our office at your convenience.

Yours truly,

SNC-LAVALIN INC.

REGISTERED PROFESSIONAL GEOSCIENTIST
Andrew C. Peach
SIGNATURE
April 11, 2014
DATE
NEWFOUNDLAND & LABRADOR

Andrew Peach, P. Geo.
Senior Engineering Geologist
SNC-Lavalin Inc. (Atlantic)

Reviewed by

Claude Gou, Ing.
Senior Geotechnical Engineer
SNC-Lavalin Inc. (Montreal)

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PERMIT HOLDER
This Permit Allows
SNC-LAVALIN INC.
To practice Professional Geoscience in Newfoundland and Labrador.
Permit No. as issued by PEGNL #N0458
which is valid for the year 2014

6.0 REFERENCES

Bowles, J.E., 1996. Foundation Analysis and Design, Fifth Edition, McGraw-Hill.

Colman-Sadd, S. P., 1990. Geology of the Island of Newfoundland. Newfoundland Department of Mines and Energy, Geological Survey Branch, Map 90-01.

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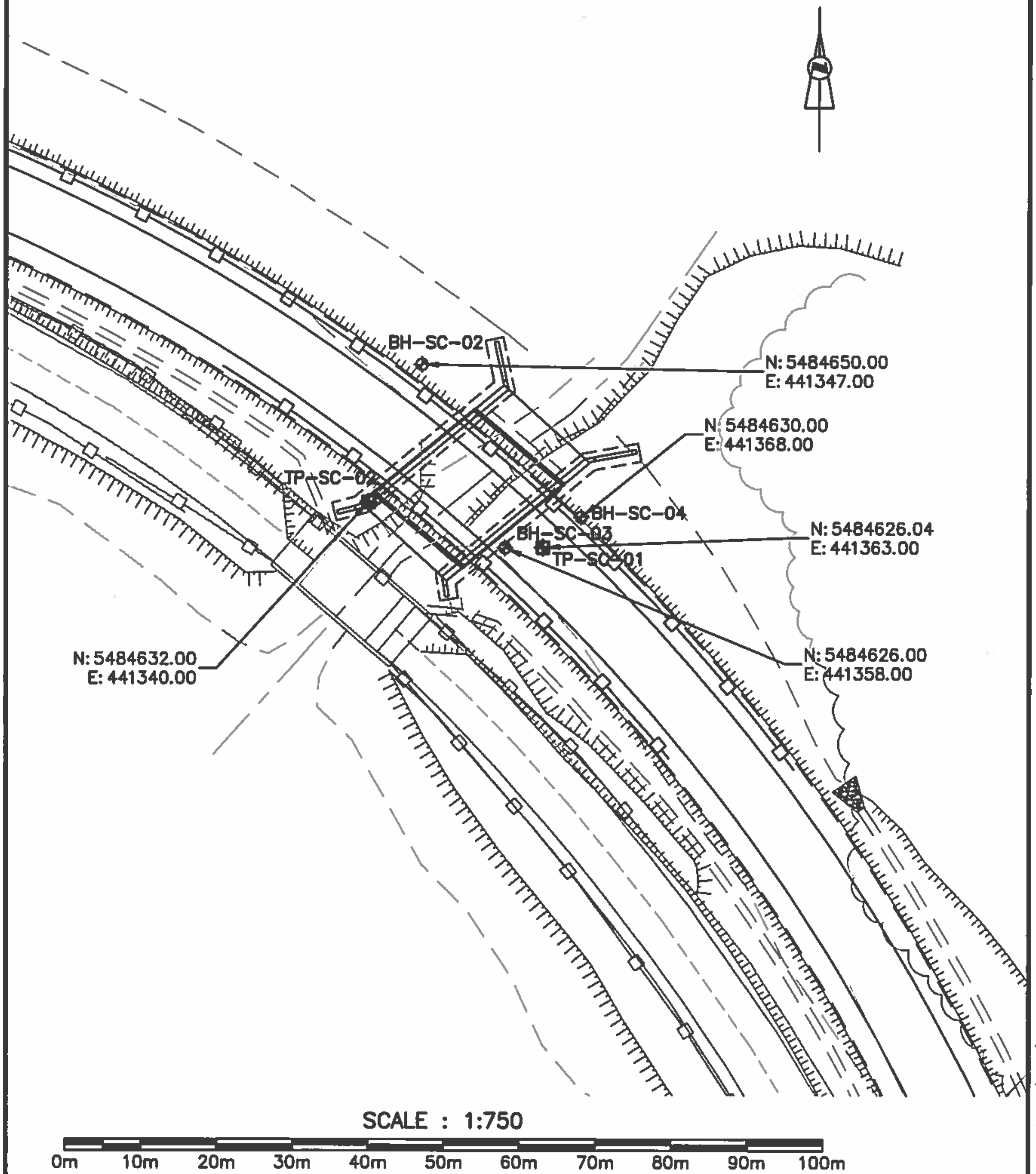
Peck, R. B., Hanson, W.E. and Thornburn, T.H.,1974. Foundation Engineering, Second Edition, Wiley.

DRAWING & FIGURE

APPENDIX A



616809-0001-D-CI-SKT-0001-01 (C05)

Public Works and Government Services Canada | Travaux publics et Services gouvernementaux Canada

project / projet: SHOAL COVE, NL BRIDGE REPLACEMENT	Drawing title / Titre du dessin: BOREHOLE LOCATIONS	designed / conçu: A.G.M. AGM	date: 2014/02/19
		drawn / dessiné: D.P.B.	date
		approved / approuvé	date
	Tender / Soumission		
	PWGSC Project Manager / Administrateur de projets TPSGC		
	project number / no. du projet	drawing no. / no. du dessin: CSK-01	

PWGSC L1P (2004)

E-DRM/GDD-E: 482827

SNC·LAVALIN
1133 TOPSAIL RD.,
MOUNT PEARL, NL, A1N 5G2
TEL: (709) 368-0118, FAX: (709) 368-3541

Borehole and Test Pit Reference Map

Location: SHOAL COVE, NL - GROS MORNE NATIONAL PARK
Client: PWGSC
Project: 616809 Created By: S. HILL Date: Mar 26, 2014

0 10 20 30 40 50
Metres
Map Scale 1:1,500

Legend
Borehole Test Pit

BOREHOLE LOGS

APPENDIX B

SNC·LAVALIN

BOREHOLE LOG

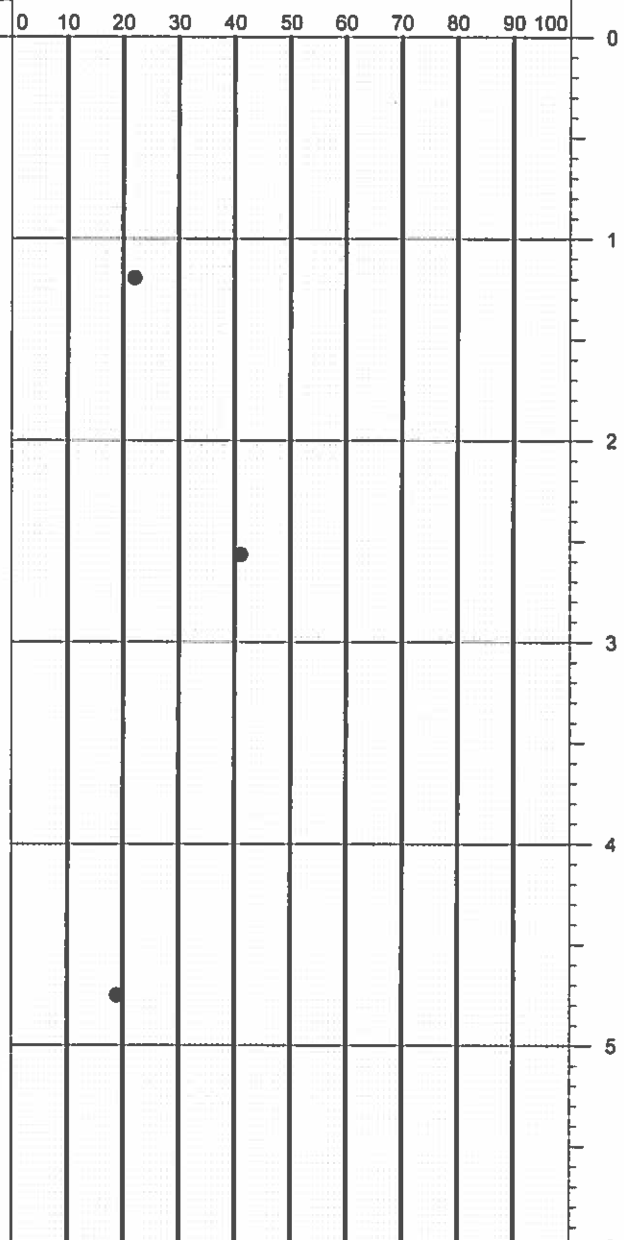
BH-SC-002-2014

Project: Shoal Cove Brook Bridge Replacement	SNC Project Number: 616809
Location: Mill Brook, Gros Morne National Park	Date: March 31, 2014
Client: PWGSC	Position: 441347 E, 5484650 N (NAD83, UTM Zone 21)
Borehole Diameter: NW/NQ	Contractor: CABO Drilling Corp.
Borehole Depth: 20.75 m	Logged By: A. Peach, P. Geo.
Datum: Mean Sea Level	Equipment: CME 55
Drilling Date: February 28 - March 1, 2014	Drilling Method: Auger; Casing; Diamond Drill
Borehole Elevation: 0.77 m	Water Level Date: March 1, 2014 (0.70 mbgs)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	TYPE	NUMBER	% RECOVERY	N-VALUE OR RQD	OTHER TESTS
0	0.77	Dark brown, wet, gravelly boulder, some sand, some cobble, trace silt. (Fill Material - description supplemented by TP-SC-002-2014; split spoon was empty)					
	-0.12	Brown, wet, compact to dense, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till - description supplemented by TP-SC-002-2014; split spoon was empty) Note: For SS #1, no advancement after last 10 blows; stopped as per ASTM Standard D1586-11. For SS #2, elevated N-value due to cobble/boulder	SS	1	0	22/102	
			SS	2	0	41	
	-3.98	Brown, wet, compact, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till)	SS	3	33	19	

UNDRAINED SHEAR STRENGTH - kPa

STANDARD PENETRATON TEST, BLOWS/0.3m ●



SHOAL COVE-BH 616809.GPJ 14/4/7

☐ Field Vane Test ■ (Remolded)
○ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

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BOREHOLE LOG

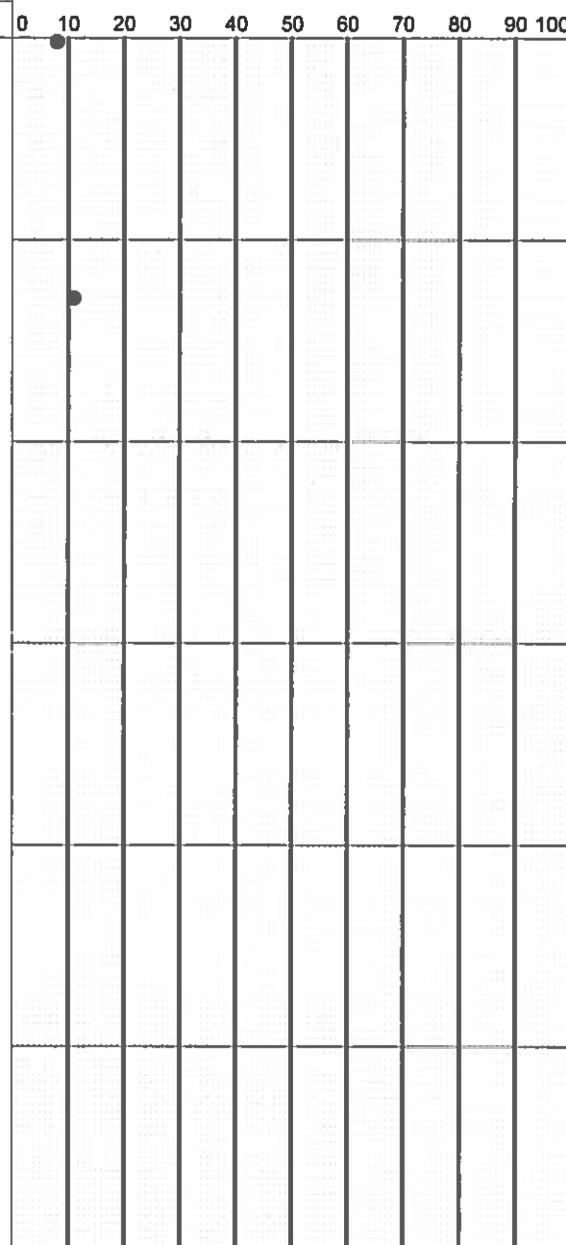
BH-SC-002-2014

Project: **Shoal Cove Brook Bridge Replacement**	SNC Project Number: **616809**
Location: **Mill Brook, Gros Morne National Park**	Date: **March 31, 2014**
Client: **PWGSC**	Position: **441347 E, 5484650 N (NAD83, UTM Zone 21)**
Borehole Diameter: **NW/NQ**	Contractor: **CABO Drilling Corp.**
Borehole Depth: **20.75 m**	Logged By: **A. Peach, P. Geo.**
Datum: **Mean Sea Level**	Equipment: **CME 55**
Drilling Date: **February 28 - March 1, 2014**	Drilling Method: **Auger; Casing; Diamond Drill**
Borehole Elevation: **0.77 m**	Water Level Date: **March 1, 2014 (0.70 mbgs)**

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	% RECOVERY	N-VALUE OR RQD	OTHER TESTS
6	-5.25								
	-5.40	Grey, wet, compact sandy gravel, some cobble, some boulder, trace silt. (Glacial Till)			SS	4	22	8	
	-5.71	Orange, wet, stiff, gravelly silt.							
7		Brown to grey, wet, compact, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till, inferred)							
	-6.52	Brown, wet, very soft, clayey silt. (Inferred based on N-values and presence of mud on outside of split spoon) Note: For SS #5, no advancement after last 10 blows; stopped as per ASTM Standard D1586-11.			SS	5	0	11/102	
	-6.82								
8	-7.59	Diorite boulder (erratic)							
9		Brown to grey, wet, compact, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till, inferred) Stopped SPT to confirm depth to bedrock.							
10									
11									
12									

UNDRAINED SHEAR STRENGTH - kPa

STANDARD PENETRATON TEST, BLOWS/0.3m ●



DEPTH (m)

SHOAL COVE-BH 616809.GPJ 14/4/7

☐ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

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BOREHOLE LOG

BH-SC-002-2014

Project: **Shoal Cove Brook Bridge Replacement**	SNC Project Number: **616809**
Location: **Mill Brook, Gros Morne National Park**	Date: **March 31, 2014**
Client: **PWGSC**	Position: **441347 E, 5484650 N (NAD83, UTM Zone 21)**
Borehole Diameter: **NW/NQ**	Contractor: **CABO Drilling Corp.**
Borehole Depth: **20.75 m**	Logged By: **A. Peach, P. Geo.**
Datum: **Mean Sea Level**	Equipment: **CME 55**
Drilling Date: **February 28 - March 1, 2014**	Drilling Method: **Auger; Casing; Diamond Drill**
Borehole Elevation: **0.77 m**	Water Level Date: **March 1, 2014 (0.70 mbgs)**

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	SAMPLES: NUMBER	SAMPLES: % RECOVERY	SAMPLES: N-VALUE OR RQD	SAMPLES: OTHER TESTS	UNDRAINED SHEAR STRENGTH - kPa / STANDARD PENETRATON TEST, BLOWS/0.3m ● (0 10 20 30 40 50 60 70 80 90 100)	DEPTH (m)
12											12
13											13
14											14
15											15
16											16
17											17
18											18

SHOAL COVE-BH 616809.GPJ 14/4/7

□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

SNC·LAVALIN BOREHOLE LOG BH-SC-002-2014

Project: Shoal Cove Brook Bridge Replacement	SNC Project Number: 616809
Location: Mill Brook, Gros Morne National Park	Date: March 31, 2014
Client: PWGSC	Position: 441347 E, 5484650 N (NAD83, UTM Zone 21)
Borehole Diameter: NW/NQ	Contractor: CABO Drilling Corp.
Borehole Depth: 20.75 m	Logged By: A. Peach, P. Geo.
Datum: Mean Sea Level	Equipment: CME 55
Drilling Date: February 28 - March 1, 2014	Drilling Method: Auger; Casing; Diamond Drill
Borehole Elevation: 0.77 m	Water Level Date: March 1, 2014 (0.70 mbgs)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	SAMPLES: NUMBER	SAMPLES: % RECOVERY	SAMPLES: N-VALUE OR RQD	SAMPLES: OTHER TESTS	UNDRAINED SHEAR STRENGTH - kPa / STANDARD PENETRATON TEST, BLOWS/0.3m ● (0 10 20 30 40 50 60 70 80 90 100)	DEPTH (m)
18											18
19											19
20											20
	-19.98	Bedrock not encountered									
		Bottom of hole: 20.75									
21											21
22											22
23											23
24											24

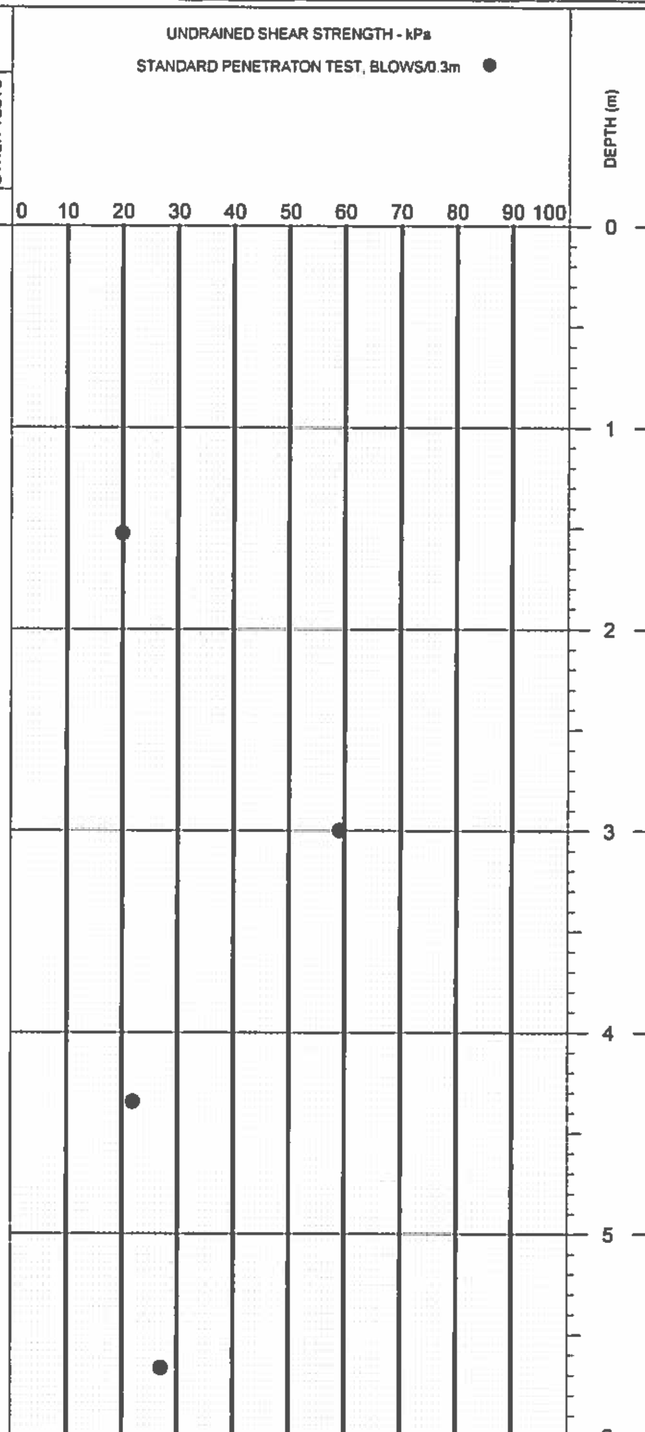
SHOAL COVE-BH 616809.GPJ 14/4/7

☐ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

SNC·LAVALIN BOREHOLE LOG BH-SC-003-2014

Project: Shoal Cove Brook Bridge Replacement	SNC Project Number: 616809
Location: Mill Brook, Gros Morne National Park	Date: March 31, 2014
Client: PWGSC	Position: 441358 E, 5484626 N (NAD83, UTM Zone 21)
Borehole Diameter: NW/NQ	Contractor: CABO Drilling Corp.
Borehole Depth: 10.64 m	Logged By: A. Peach, P. Geo.
Datum: Mean Sea Level	Equipment: CME 55
Drilling Date: February 24 - February 26, 2014	Drilling Method: Auger; Casing; Diamond Drill
Borehole Elevation: 1.12 m	Water Level Date: February 24, 2014 (1.27 mbgs)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	NUMBER	% RECOVERY	N-VALUE OR RQD	OTHER TESTS
0	1.12	Dark brown, wet, compact, sandy gravel, some cobble, some boulder, trace silt. (Fill Material) Note: No N-value for SS #1 due to lack of benchmark			SS	1	28	N.R.	
				∇ (1.27)	SS	2	22	20	
2	-0.86	Light brown, wet, very dense to compact, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till) Note: Elevated N-value for SS #3 due to cobble/boulder			SS	3	11	59	
					SS	4	0	22	
	-4.54	Light grey, wet, compact, sand and gravel, some			SS	5	17	27	



□ Field Vane Test ■ (Remolded)
○ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

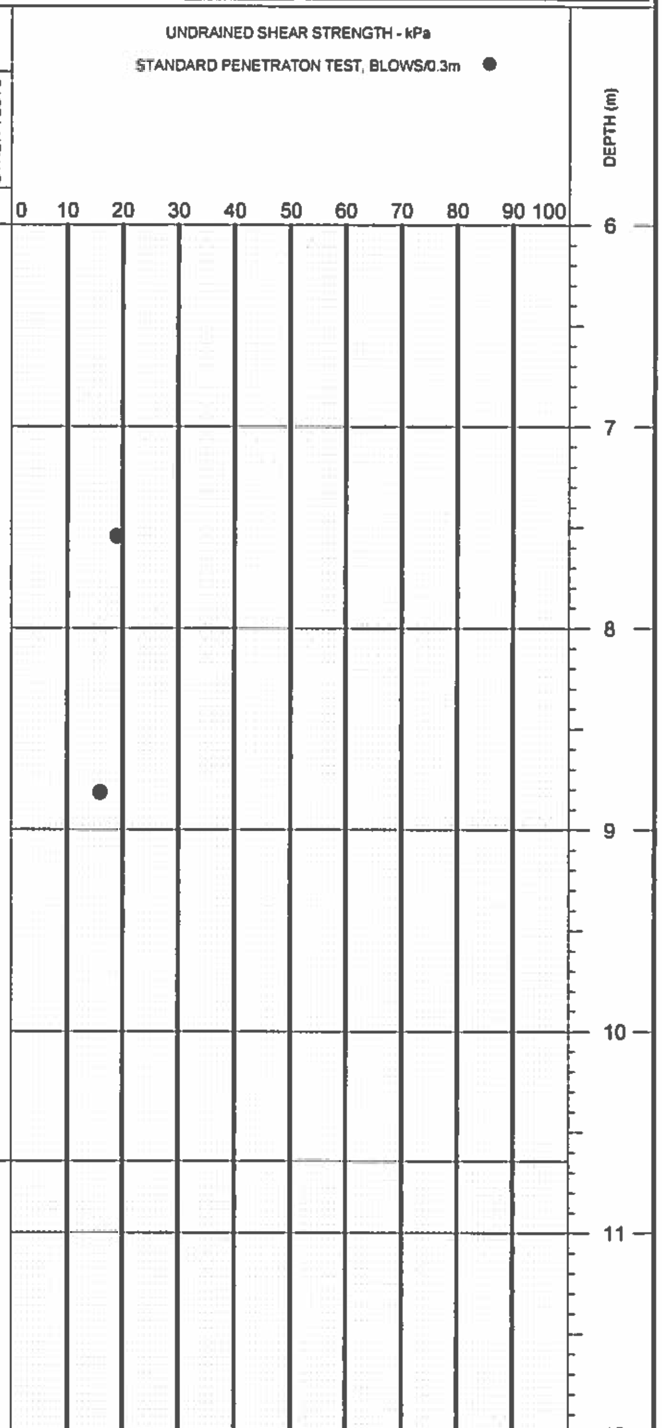
SNC·LAVALIN

BOREHOLE LOG

BH-SC-003-2014

Project: Shoal Cove Brook Bridge Replacement	SNC Project Number: 616809
Location: Mill Brook, Gros Morne National Park	Date: March 31, 2014
Client: PWGSC	Position: 441358 E, 5484626 N (NAD83, UTM Zone 21)
Borehole Diameter: NW/NQ	Contractor: CABO Drilling Corp.
Borehole Depth: 10.64 m	Logged By: A. Peach, P. Geo.
Datum: Mean Sea Level	Equipment: CME 55
Drilling Date: February 24 - February 26, 2014	Drilling Method: Auger; Casing; Diamond Drill
Borehole Elevation: 1.12 m	Water Level Date: February 24, 2014 (1.27 mbgs)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES % RECOVERY	SAMPLES N-VALUE OR RQD	OTHER TESTS
6		cobble, some boulder, trace silt. (Glacial Till)							
7									
	-6.58				SS	6	33	19	
	-6.88	Medium grey, wet, very stiff, sandy silt, trace gravel. (3" observed, thickness inferred based on N-values) Tested with pocket penetrometer UCS = 440 kPa							
8		Light grey, wet, compact, sand and gravel, some cobble, some boulder, trace silt. (Glacial Till, inferred).							
9					SS	7	0	16	
10									
	-9.52	Bottom of hole: 10.64							
11									
12									



SHOAL COVE-BH 616809.GPJ 14/4/7

□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

SNC·LAVALIN — BOREHOLE LOG — BH-SC-004-2014

Project: Shoal Cove Brook Bridge Replacement	SNC Project Number: 616809
Location: Mill Brook, Gros Morne National Park	Date: March 31, 2014
Client: PWGSC	Position: 441368 E, 5484630 N (NAD83, UTM Zone 21)
Borehole Diameter: NW/NQ	Contractor: CABO Drilling Corp.
Borehole Depth: 14.73 m	Logged By: A. Peach, P. Geo.
Datum: Mean Sea Level	Equipment: CME 55
Drilling Date: February 27, 2014	Drilling Method: Auger; Casing; Diamond Drill
Borehole Elevation: 1.05 m	Water Level Date: N.R.

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	NUMBER	% RECOVERY	N-VALUE OR RQD	OTHER TESTS	UNDRAINED SHEAR STRENGTH - kPa / STANDARD PENETRATON TEST, BLOWS/0.3m ● (0 10 20 30 40 50 60 70 80 90 100)	DEPTH (m)
0	1.05	Dark brown, moist to wet, compact, sandy gravel, some cobble, some boulder, trace silt. (Fill Material, inferred - description supplemented by TP-SC-001-2014). No SPT; borehole drilled to confirm depth to bedrock									0
1.53	-0.48	Brown, wet, loose to compact, sandy gravel, some cobble, some boulder, trace silt. (Glacial Till, inferred - description supplemented by TP-SC-001-2014). Glacial erratics encountered throughout (e.g., felsic granite, diorite)									

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8

SHOAL COVE-BH 616809.GPJ 14/4/7

☐ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ⧅ Torvane

BOREHOLE LOG

SNC·LAVALIN

BH-SC-004-2014

Project: **Shoal Cove Brook Bridge Replacement**	SNC Project Number: **616809**
Location: **Mill Brook, Gros Morne National Park**	Date: **March 31, 2014**
Client: **PWGSC**	Position: **441368 E, 5484630 N (NAD83, UTM Zone 21)**
Borehole Diameter: **NW/NQ**	Contractor: **CABO Drilling Corp.**
Borehole Depth: **14.73 m**	Logged By: **A. Peach, P. Geo.**
Datum: **Mean Sea Level**	Equipment: **CME 55**
Drilling Date: **February 27, 2014**	Drilling Method: **Auger; Casing; Diamond Drill**
Borehole Elevation: **1.05 m**	Water Level Date: **N.R.**

UNDRAINED SHEAR STRENGTH - kPa

STANDARD PENETRATON TEST, BLOWS/0.3m ●

0 10 20 30 40 50 60 70 80 90 100

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	% RECOVERY	N-VALUE OR RQD	OTHER TESTS	DEPTH (m)
8										8
9										9
10										10
11										11
12										12
	-11.58	Bedrock: orthoquartzite			RC	1	93	56		
13										13
14										14
	-13.68	Bottom of hole: 14.73								
15										15
16										16

SHOAL COVE-BH 616809.GPJ 14/4/7

☐ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Pocket Penetrometer Test ◪ Torvane

TEST PIT LOGS

APPENDIX C

Test pit excavation.

Test pit spoilage. Levelling rod for scale.

Test Pit Identification Number	Depth* (mbgs) From - To	Soil Description
TP-SC-001-2014 UTM NAD 83 Zone 21 5484626N 441363E	0.0 – 0.1	**Topsoil** – Dark brown, frozen root mat with organic matter.
	0.1 – 1.1	**Fill** – Dark brown, wet, compact, cobbley, bouldery gravel, some sand, trace silt. Approximately 25% boulder; maximum boulder diameter approximately 600 mm. Boulders are angular to sub-angular. Composition on boulders indiscernible.
	1.1	Test pit terminated due to refusal on large boulders.

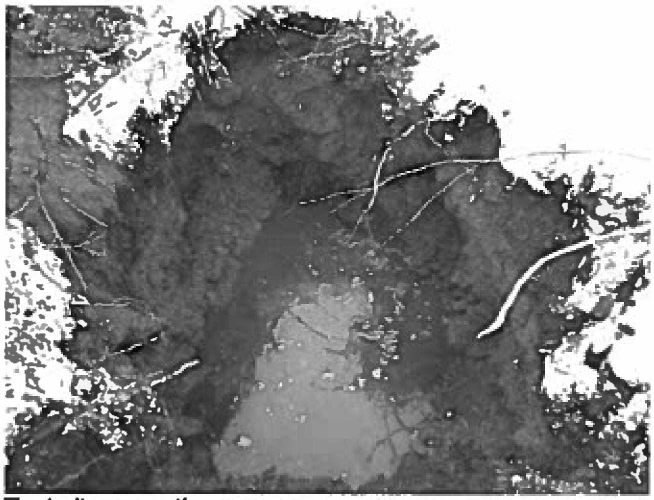
*All depths are approximate.

Location: Shoal Cove Brook Bridge – Route 430, Near Norris Point, NL

Date: February 26, 2014

Notes

- Sample taken between 0.4 – 0.6 mbgs
- Seepage noted at 0.6 mbgs.
- Groundwater pool at base of pit approximately 0.2 m deep.
- CAT 315CL hydraulic excavator used to dig test pits (provided by CABO Drilling Corp.).



Test pit excavation.

Test pit spoilage with levelling rod for scale.

Test Pit Identification Number	Depth* (mbgs) From - To	Soil Description
TP-SC-002-2014 UTM NAD 83 Zone 21 5484632N 441340E	0.0 – 0.1	**Topsoil** – Dark brown, frozen root mat with organic matter.
	0.1 – 0.9	**Fill** – Dark brown, wet, compact gravelly, sandy boulders, some cobble, trace silt. Approximately 30 – 40% boulder; maximum boulder diameter approximately 500 mm. Boulders and cobbles are angular.
	0.9 – 1.9	**Glacial Till** – Medium brown, wet, compact, cobbley gravel, some sand, some boulder, trace silt. Approximately 10 – 15% boulder; maximum boulder diameter approximately 300 mm. Boulders and cobbles comparatively much more rounded.
	1.9	Test pit terminated due to refusal on large boulders.

*All depths are approximate.

Location: Shoal Cove Brook Bridge – Route 430, Near Norris Point, NL

Date: February 28, 2014

<u>Notes</u>

- Sample taken between 0.9 – 1.9 mbgs
- Seepage noted at 0.9 mbgs.
- Groundwater pool at base of pit approximately 0.2 m deep.
- Sharp boundary between fill and glacial till layers; seepage occurring at the interface.
- CAT 315CL hydraulic excavator used to dig test pits (provided by CABO Drilling Corp.).

LABORATORY RESULTS

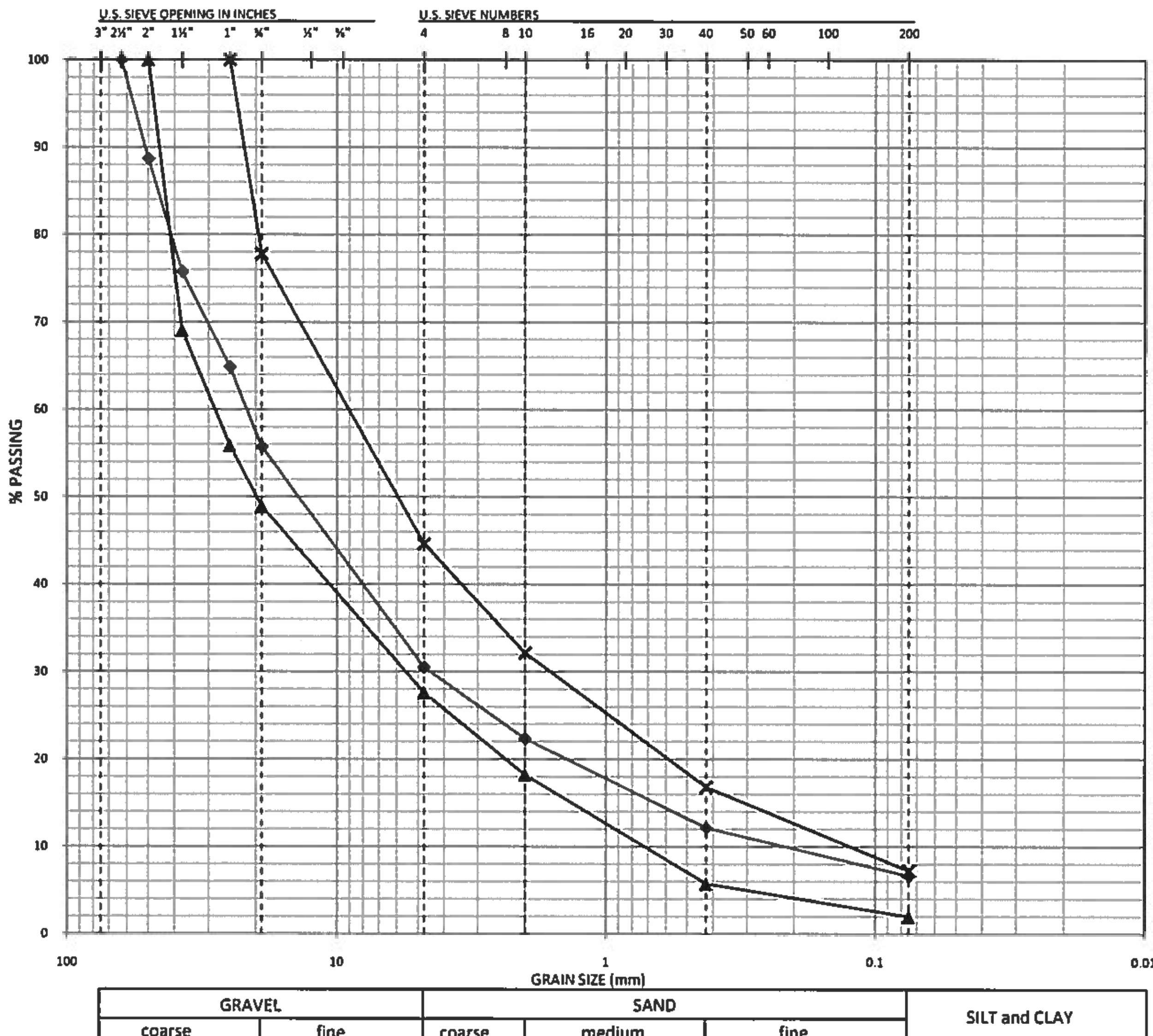
APPENDIX D

SNC • LAVALIN

1133 Topsail Road
Mount Pearl, NL A1N 5G2
Phone: (709) 368-0118
Fax: (709) 368-0158

SIEVE ANALYSIS REPORT

CLIENT **Public Works Canada** DATE RECEIVED **March 3, 2014**
PROJECT **Shoal Cove Geotechnical Investigation** DATE TESTED **March 14, 2014**
JOB NUMBER **616809** TESTED BY **Karen Lalonde, B.Sc.**



	Sample	Depth (mbgs)	Soil Classification (as per ASTM D2487)	%Gravel	%Sand	%Silt	%Clay
♦	TP-SC-001-2014	0.4 - 0.6	Poorly graded gravel with silt and sand (GP-GM)	70	24	6.4	
▲	TP-SC-002-2014	0.9 - 1.9	Well graded gravel with sand (GW)	72	26	2.0	
x	BH-SC-002-2014	4.7	Well graded gravel with silt and sand (GW-GM)	55	37	7.2	

	Sample	MC%	LL	PL	PI	D100	D60	D30	D10	Cc	Cu
♦	TP-SC-001-2014	10.44				63	22	4.7	0.23	4.37	95.65
▲	TP-SC-002-2014	12.61				50	29	5.6	0.71	1.52	40.85
x	BH-SC-002-2014	10.47				25	9	1.7	0.14	2.29	64.29

N.R. = not recorded

COMMENTS

TP-SC-001-2014: 25% boulder; 30% cobble (field estimate, as per Canadian Foundation Engineering Manual, 4th Ed.).

TP-SC-002-2014: 30-40% boulder; 10-15% cobble (field estimate, as per Canadian Foundation Engineering Manual, 4th Ed.)

APPROVED BY: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