FINAL REPORT, GEOTECHNICAL INVESTIGATION, NORTHWEST RIVER BRIDGE RECONSTRUCTION, TERRA NOVA NATIONAL PARK, NL

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Prepared for:
Harbourside Engineering Consultants
219 Waverly Road, Suite 200
Dartmouth NS B2X 2C3

Prepared by:
Stantec Consulting Ltd.
102-40 Highfield Park Drive
Dartmouth NS B3A 0A3
Tel: (902) 468-7777
Fax: (902) 468-900

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1.0 INTRODUCTION

Acting on the request and authorization of Harbourside Engineering Consultants (Harbourside), Stantec Consulting Ltd. (Stantec) has completed a geotechnical investigation for the proposed reconstruction of the Northwest River Bridge in Terra Nova National Park, Newfoundland and Labrador.

We understand that the proposed work consists of the reconstruction of the existing two span concrete slab and concrete girder bridge. The purpose of this geotechnical investigation is to determine the subsurface soil and rock conditions at the site to provide geotechnical comments and recommendations to assist with site earthworks and foundation design.

The scope of work completed for this project was in general accordance with Stantec's proposal dated May 15, 2014 and included the following:

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

This report has been prepared specifically and solely for the proposed project described herein and contains all of the findings of this investigation.

2.0 SITE AND GEOLOGY

The proposed reconstruction project is located where the Trans-Canada Highway in Terra Nova National Park, Newfoundland and Labrador spans the Northwest River. The location of the existing bridge (to be reconstructed) is shown on Drawing No. 1, Borehole and Test Pit Location Plan in Appendix A. The proposed replacement structure lies to the south of the existing bridge.

Based on available information from geological mapping in the vicinity of the site, the native overburden material consists of a thin blanket of glacial till (less than 3 m in thickness) extending to bedrock. Bedrock geology at the site is mapped as sedimentary rocks of the Musgravetown Group.

3.0 FIELD PROCEDURES

The geotechnical field investigation was completed from November 26th to December 1st, 2014 and consisted of six boreholes and four test pits. Boreholes were drilled to depths ranging from 3.0 m to 4.0 m below the ground surface on the bridge approach and depths ranging from 10.7 m to 14.0 m below ground surface near the bridge abutments. Test pits were excavated to depths of 1.2 to 1.8 m near the bridge approaches. Upon completion of drilling, standpipe was installed in the boreholes and they were backfilled with sand; test pits were filled with the excavated material.

The field work was conducted under the supervision of Stantec personnel, who maintained detailed field records of the various soil strata and groundwater conditions encountered. The soils were classified in general accordance with the procedures outlined in the appended explanatory key: Symbol and Terms Used on Borehole and Test Pit Records. Soil samples were obtained from the split spoon sampler during the investigation. Bedrock was cored using HQ-sized core barrel. The Rock Quality Designation (RQD) and recovery of the samples were measured and recorded. RQD is the ratio of the sum of all the core recovered greater than 100 mm in length divided by the total length drilled, expressed as a percentage. Soil samples were stored in moisture-proof containers and rock core was stored in core boxes. Samples of soil and rock were sent to our laboratory in St. John’s, Newfoundland and Labrador for classification and testing. A bulk sample of soil was shipped to Dalhousie University, Halifax, Nova Scotia for direct shear testing.

The approximate locations of each test location are shown on the attached Drawing No. 1, Borehole and Test Pit Location Plan. Boreholes were established in the field based on measurements from existing infrastructure and a hand-held, recreational grade, GPS unit. Surface elevations at the test locations were estimated by using these coordinates in concert with a topographic survey sent by Harbourside via e-mail on January 14, 2015 and thus should be used with caution.

4.0 LABORATORY TESTING

Laboratory testing consisting of soil gradations and water content determinations were performed on selected soil samples. The laboratory test results are summarized in the following section and presented on the Borehole Records, Test Pit Records, or on separate figures in Appendix A.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions observed in the boreholes and test pits are summarized in Table 1 and in the following paragraphs and described in detail on the appended Borehole Records and Test Pit Records.

Table 1 Summary of Subsurface Conditions

Location	Ground Elevation (m)	Asphalt Thickness (m)	Fill Thickness (m)	Till Thickness (m)	Bedrock		Groundwater		Total Depth (m)
					Depth to Surface (m)	Surface Elev. (m)	Depth (m)	Elev. (m)	
BH-01	17.06	0.20	7.57	-	7.77	9.29	6.8	10.3	10.82
BH-02	17.04	0.15	7.47	-	7.62	9.42	6.6	10.4	10.70
BH-03	17.04	0.20	6.07	-	6.27	10.77	7.0	10.0	11.73
BH-04	17.04	0.20	8.33	-	8.53	8.51	7.2	9.8	14.00
BH-05	24.22	0.20	3.15	>0.71	-	-	3.4	20.82	4.06
BH-06	19.75	0.20	1.47	>1.38	-	-	1.6	18.15	3.05
TP-01	17.8	-	1.2	>0.6	-	-	1.2	16.6	1.8
TP-02	19.6	-	1.2	>0.3	-	-	-	-	1.5
TP-03	21.8	-	0.9	>0.6	-	-	-	-	1.5
TP-04	20.5	-	>1.2	-	-	-	-	-	1.2

Note: Elevations are estimated from approximate borehole locations and a topographic survey and should be considered approximate.

5.1 ASPHALT AND FILL

A layer of asphalt approximately 0.2 m thick was encountered at the surface at all borehole locations.

5.2 FILL

Fill, ranging in thickness from 0.9 to 8.3 m, was encountered underlying the asphalt at all six borehole locations and at the surface of all four test pits. Two layers of fill were noted and are discussed separately in the following subsections.

5.2.1 Grey Poorly Graded Gravel

Grey poorly-graded gravel (GP) fill ranging in thickness from 0.3 m to 0.6 m was encountered underlying the asphalt in all borehole locations except BH-04 and at the surface of all test pits.

5.2.2 Brown Well-Graded Sand with Silt and Gravel

Brown fill was encountered underlying the asphalt in Borehole BH-05 and underlying the grey poorly graded gravel fill in the other five boreholes. This layer ranged in thickness from 0.6 m to 2.9 m at the bridge approach (Boreholes BH-05, BH-06 and Test Pits TP-01, TP-02, TP-03, TP-04) and 5.8 m to 8.3 m near the bridge abutments (Boreholes BH-01, BH-02, BH-03, and BH-04). Based on our field classifications and gradation analyses, the fill may be described as brown, well-graded sand with silt and gravel (SW-SM) with occasional cobbles and boulders. Gradation analyses conducted on two samples of the material indicated 26 to 27% gravel, 63 to 66% sand and 8 to 11% silt- and clay-sized particles. The result of the gradation analyses are shown on the gradation curves included in Appendix A. The water contents of the two tested samples from this layer were 3% and 4%. Our interpretation of this material is that it likely comprises re-worked native till from the surrounding area.

Direct shear testing on a sample of this material was performed at Dalhousie University's Geotechnical Laboratory in Halifax, Nova Scotia. The results are appended and indicate a drained friction angle of 40 degrees.

5.3 GLACIAL TILL

Native grey glacial till was encountered or inferred underlying the fill materials in Boreholes BH-05 and BH-06 and Test Pits TP-01, TP-02 and TP-03. Based on our field classifications and gradation analyses, the till was generally classified as well-graded sand with silt and gravel (SW-SM). Gradation analyses conducted on two samples of the soil indicated of 26 to 37% gravel, 45 to 48% sand and 7 to 15% silt-and clay-sized particles. The results of the gradation analyses are appended. The natural water contents of two samples were 9% and 14%.

Based on Standard Penetration Test N-Values, as well as drilling and excavator performance, the relative density of the till may be described as dense to very dense.

5.4 BEDROCK

Bedrock was encountered at BH-01, BH-02, BH-03 and BH-04 at depths ranging from 6.3 m to 8.5 m below the ground surface. Bedrock may be described as a poor to excellent quality, light grey to purple, weak to medium strong, fresh to slightly weathered, sedimentary rock (fine-grained sandstone to siltstone). Unconfined compressive strength testing was performed on four samples of HQ-sized rock core; the results are included in Appendix A and summarized in Table 2.

Table 2 Unconfined Compressive Strength Test Results

Borehole and Sample	Depth (m)	Compressive Strength (MPa)
BH-02, HQ-1	7.9	43.6
BH-02, HQ-2	10.1	103.0
BH-03, HQ-1	7.0	132.5
BH-04, HQ-2	7.2	128.0

5.5 GROUNDWATER

Groundwater levels were measured in the boreholes at the time of the investigation. Near the bridge approach (i.e. Boreholes BH-05, BH-06) the measured groundwater levels were 3.4 m and 1.6 m below ground surface, respectively. In Test Pit TP-01 the inferred groundwater level was 1.2 m below ground surface.

Near the abutments (i.e. Boreholes BH-01, BH-02, BH-03, and BH-04) the groundwater levels were measured to be 6.6 m to 7.2 m below ground surface. Due to the close proximity of these boreholes to the Northwest River, it is anticipated that groundwater levels near the abutments will be at or near the river elevation. It should be noted that water levels may fluctuate with construction activity, and in response to precipitation events and seasonal weather trends.

6.0 DISCUSSION AND RECOMMENDATIONS

We understand that a single-span bridge is proposed just south of the existing bridge. We further understand that the proposed design requires the existing roadway to be widened and the alignment shifted to the south of the existing bridge.

6.1 SITE PREPARATION

Based on the current drawings of the proposed replacement bridge, it is anticipated that approach fills up to approximately 6.5 m thick may be necessary to achieve design grades.

It is recommended that any existing rootmat/topsoil beneath the footprints of the approach fill embankments be grubbed. Exposed site soils may be susceptible to deterioration due to trafficking, especially during periods of precipitation or when working below/near the groundwater table. Therefore, prepared surfaces should be protected to minimize the amount of degradation. It may be prudent to provide a stabilizing layer of rock fill (300 to 600 mm in thickness) in areas where exposed soils will be subject to high construction traffic.

Existing site materials, including some existing fill materials, may be suitable to support the approach fills. All material within the zone of influence of the approach fills should be inspected by qualified geotechnical personnel and any deleterious materials (i.e. soft sols, organic materials, debris) should be removed. The exposed soil surface should be re-graded and compacted prior to fill placement.

Portions of the existing fill material or local glacial till would be a suitable option for use as approach fills. Imported fill should consist of well-graded granular material. Approach fill should generally be compacted to at least 95% of the standard Proctor maximum dry density (SPMDD) and the upper 300 mm below subgrade should be compacted to a minimum of 98% SPMDD. Approach fill should be placed in lifts compatible with the compaction equipment used and at a water content that will allow compaction to the specified density.

Prior to placement of pavement gravels, the subgrade should be tested with a loaded tandem truck under the supervision of qualified geotechnical personnel. Any soft areas or yielding material with deflections greater than 20 mm within the subgrade should be removed and replaced with suitable material.

If water is encountered in excavations, it should be directed to sumps and pumped. Good practice suggests that surface water should be directed away from excavations using ditches/swales. Any water discharged from site should meet all applicable regulatory requirements.

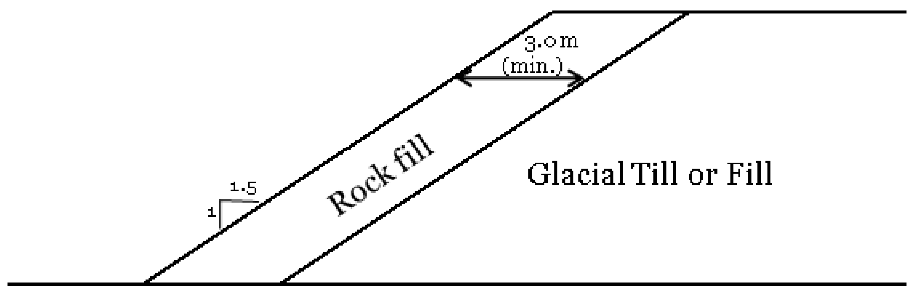
Gradation analyses and field classification of the site fill and glacial till materials suggests that they have a relatively high hydraulic conductivity (in the range of 1.0×10^{-4} to 1×10^{-2} cm/s). This high hydraulic conductivity combined with the proximity of the river suggests that dewatering excavations below the river level may require substantial pumping.

We understand that excavation of overburden and bedrock is required to increase the 'free length' of the piles supporting the abutment to allow them to behave as desired for the integral abutment design. Based on the rock quality and unconfined compressive strength results (Section 5.4) we do not expect the bedrock to be rippable with an excavator. A hydraulic hammer or blasting will likely be required to break the rock to the required depth.

6.2 SLOPES

All permanent slopes in native glacial till or compacted site fill should be no steeper than 2 horizontal to 1 vertical (2H:1V). If steeper slopes are required, permanent slopes as steep as 1.5H:1V may be constructed provided that a minimum 3 m buttress (measured horizontally as noted in Figure 1) of angular well-graded rock fill is placed on the slope.

Figure 1 Rock Fill Buttress Required for Permanent 1.5H:1V Slopes



Temporary slopes for construction in the fill or glacial till material up to 6 m in height may be constructed at 1.5H:1V.

6.3 ABUTMENTS (PILES)

The use of steel H-piles or open-ended pipe piles driven to practical refusal in bedrock is a suitable option to support the bridge abutments. These piles may be designed using a ULS geotechnical axial compressive resistance of 70 MPa based on the cross-sectional area of the steel. 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.

The factored compressive axial resistance of the preferred section (HP 360x152) is provided below; we would be pleased to review other sections upon your request. To achieve this capacity, the piles should penetrate the overburden and may also be required to penetrate up to 1 to 2 m into the bedrock.

Pile Type	Factored Axial Resistance
HP 360 x 152	1360 kN (compression)

The piles should be driven with a hammer having a minimum rated energy of 350 Joules/cm^2 of steel cross-sectional area although lower energies may be required initially. As discussed in the previous section, we understand that the design requires excavation of rock to increase the 'free length' of the piles. In this instance the piles may be set directly on bedrock and hence there is a risk of overstressing the piles as the preponderance of the driving energy is transferred to the pile tip. Dynamic monitoring should be carried out on the initial pile installations to verify that overstressing does not occur and the hammer is operating within normal efficiencies. Furthermore, we recommend that the driving energy should initially be at the low-end of the hammers rated energy and that dynamic monitoring is used to evaluate stresses prior to driving at higher energies required to mobilize the required resistance. Drive shoes should be used to protect the pile during installation. Furthermore, the contractor should provide full details on the method of installation and equipment prior to starting the work.

Practical refusal in bedrock should be taken as a pile penetration of less than 25 mm for 15 blows. It is recommended that dynamic pile monitoring (e.g. Pile Driving Analyzer system) be performed during the initial pile installations to confirm the refusal criteria and verify that overstressing of the pile does not occur.

To evaluate the potential for relaxation to occur following initial driving, at least 10% of the piles should be re-struck 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 can be achieved during the re-strike. If significant relaxation continues to occur, dynamic pile monitoring should be carried out to determine if the required load capacity is being developed.

The capacity 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 a minimum of three pile diameters. The expected SLS settlement of piles driven to refusal on or in bedrock is expected to be negligible.

Based on correspondence with Harbourside, it is understood that the piles will not be subject to tensile loads.

Dynamic pile monitoring should be performed to confirm the estimated resistance provided for design is achieved at the set criteria. The monitoring should consist of at least one initial drive and one re-strike for each abutment. Furthermore, full-time inspection by qualified geotechnical personnel is recommended during pile installation.

The pile cap should be founded a minimum of 1.2 m below finish grade to provide adequate frost protection.

6.4 RETAINING WALLS (WING WALLS)

We understand that the wing walls are to be cantilevered off of the abutment and thus supported by piles.

Backfill placed against retaining walls should be a non-frost susceptible, non-expansive, non-corrosive, free-draining, well-graded material such as Granular "B" as specified by Newfoundland and Labrador's Department of Transportation and Works Specifications Book. As a minimum, backfill behind retaining walls should account for a wedge beginning at the heel of the wall and extending upward at an angle of 45 degrees from the horizontal.

Retaining walls should be 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 maximum standard Proctor dry density. To limit compaction-induced stresses, compaction immediately adjacent to the wall should be accomplished using lightweight compaction equipment and relatively thin soil lifts.

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

The earth pressure coefficients used for design should be selected based on the appropriate finished back-slope angle. The unfactored values for the parameters presented in Section 6.5 may be used for design purposes. Walls that can tolerate little or no movement should be designed for at-rest lateral earth pressures.

6.5 SOIL PARAMETERS

The following unfactored values for the indicated parameters may be used for design purposes:

Table 3 Unfactored Geotechnical Material Parameters

Parameter	Value	
	In-Situ Native Glacial Till or Compacted Site Fill(a)	Compacted Granular "B" (a) (b)
Effective Angle of Internal Friction, degrees	34	36
Cohesion, kPa	0	0
Total Unit Weight, kN/m3	21.5	21.0
Submerged Unit Weight(c), kN/m3	11.5	11.0
Coefficient of Active Earth Pressure(d)	0.28	0.26
Coefficient of Passive Earth Pressure(d)	3.54	3.85
Coefficient of At-Rest Earth Pressure(d)	0.44	0.41
Friction Factor, Soil/Concrete Interface(e)	0.40	0.50

(a) Material shall be placed in lifts and suitably compacted as described in geotechnical investigation report.
(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 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.

6.6 PAVEMENT DESIGN

Based on the site soils, proposed approach fills, and expected traffic loading, the following pavement design is recommended.

Table 4 Pavement Designs for Light Duty and Heavy Duty Pavement Structures

Materials	Pavement Structure
Asphaltic Surface Course	50 mm
Asphaltic Base Course	60 mm
Granular "A"	150 mm
Granular "B"	300 mm

The pavement design is based on the subgrade soils being in a stable condition at the time the gravels are placed. The subgrade soils may become soft and constructability can be a problem. In such cases, a stabilizing layer of rockfill and/or filter fabric may be required.

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.

6.7 WINTER WEATHER CONDITIONS

If 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 these procedures and precautions experience has shown that earthworks in these types of soils often become impractical at temperatures below approximately -5°C.

6.8 SEISMIC SITE CLASSIFICATION

Based on the findings at the borehole 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 D (stiff soil).

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions, attached. It is the responsibility of Harbourside Engineering Consultants, who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec should any of these not be satisfied. The Statement of General Conditions addresses the following: use of the report; basis of the report; standard of care; interpretation of site conditions; varying or unexpected site conditions; and planning, design or construction.

Stantec requests an opportunity to review the comments and recommendations provided herein when the project specifications and drawings become available. We trust this report meets your present requirements. Should any additional information be required, please do not hesitate to contact our office at your convenience.

Sincerely,

STANTEC CONSULTING LTD.

S. Greg MacNeill, P.Eng.
Principal, Geotechnical Engineering

Mark Bochmann, P.Eng.
Associate, Geotechnical Engineering

Vince C. Goreham, Ph.D., P.Eng.
Geotechnical Engineer

APPENDIX A

Statement of General Conditions
Symbols and Terms Used on Borehole and Test Pit Records
Drawing No. 1, Borehole and Test Pit Location Plan
Borehole Records
Test Pit Records
Gradation Curves
Direct Shear Test Results
Unconfined Compressive Strength of Rock Cores

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

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	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

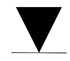
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

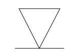
SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



measured in standpipe, piezometer, or well



inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

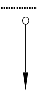
Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

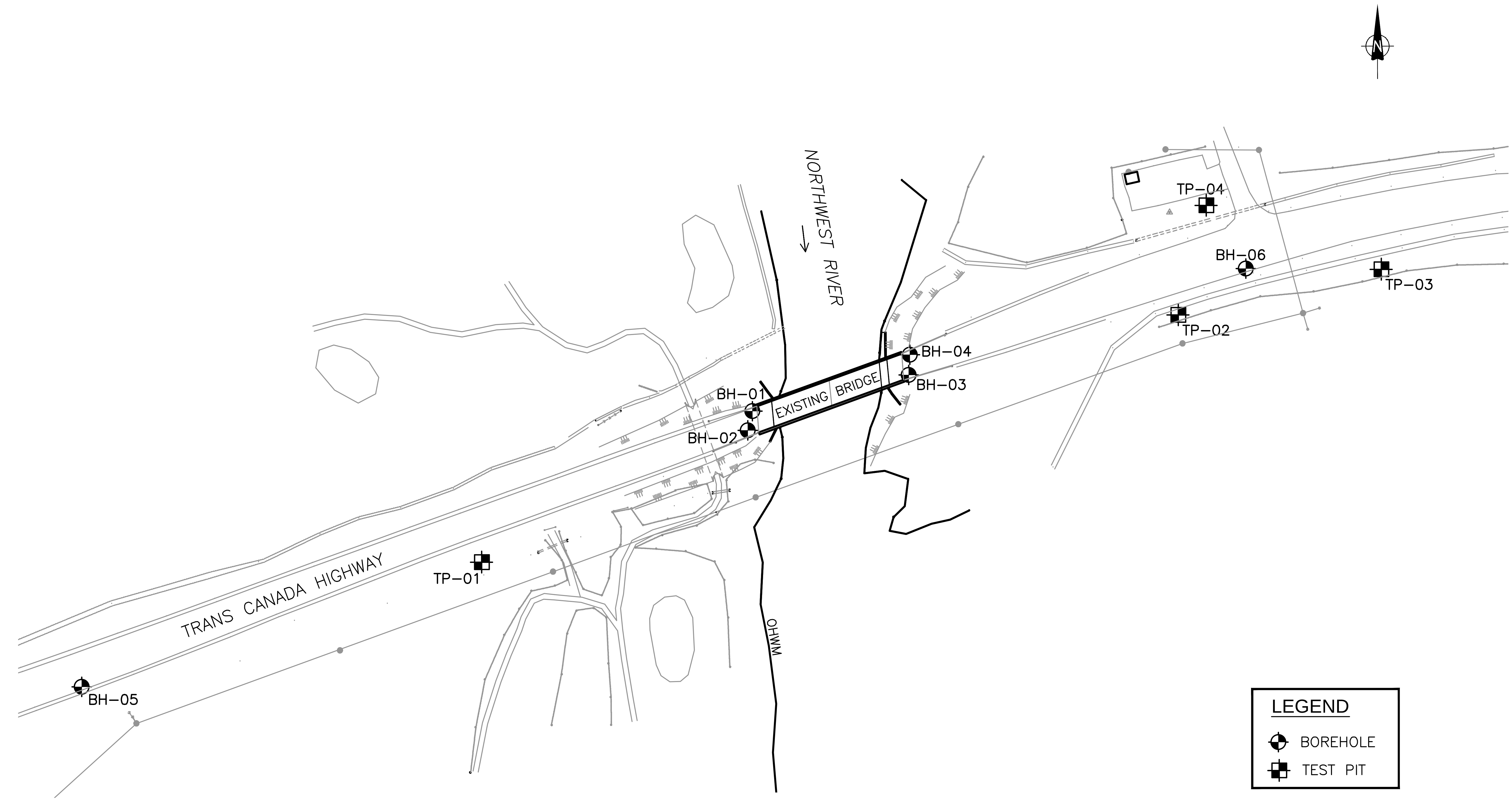
DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests 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 Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer



NOTE: BOREHOLE LOCATIONS ARE APPROXIMATE.

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference:
CAD FILES "14140-BH2" & "TR-NORTHWEST-BRIDGE" PROVIDED BY CLIENT, JAN. 2015

Job No.:	121618147
Scale:	1:1500
Date:	2015/01/14
Dwn. By:	BSP
App'd By:	JCG

Client:
HARBOURSIDE ENGINEERING CONSULTANTS

Site Address
PORT BLANDFORD, NL

Project:
NORTHWEST RIVER BRIDGE RECONSTRUCTION

Drawing Title:
BOREHOLE AND TEST PIT LOCATION PLAN

Dwg. No.: 1

Stantec

T:\1216XXX\121618147 NW Bridge NL\121618147-1.dwg PRINTED: Mar 24, 2015

Stantec

BOREHOLE RECORD

BOREHOLE No. BH-01
PAGE 1 of 1
PROJECT No. 121618147
DRILLING METHOD Wash boring
SIZE
DATUM Geodetic (Approx.)

CLIENT Harbourside Engineering Consultants
PROJECT Northwest River Bridge Reconstruction
LOCATION Terra Nova National Park, NL
DATES (yy-mm-dd): BORING 14-11-29 to 14-11-30 WATER LEVEL 6.8m on 14-12-1

DEPTH (m)	ELEVATION (m)	DESCRIPTION	WATER LEVEL	TYPE	NUMBER	RECOVERY OR TCR(%) (mm)	N-VALUE OR RQD (%)	OTHER TESTS	STANDARD PENETRATION TEST, BLOWS/0.3m
0	17.06	ASPHALT							
0.2	16.86	FILL: grey rockfill - sub-angular to angular		SS	1	356	47		47
0.81	16.25	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till		SS	2	152	34		34
				SS	3	457	37		37
				SS	4	0	27		27
				SS	5	229	20		20
				SS	6	152	14		14
				SS	7	152	19		19
				SS	8	152	14		14
				SS	9	178	9		9
			∇ (6.8 m)	SS	10	178	53		53
		- 125 mm cobble at 7.5 m depth							
7.77	9.29	Good quality light grey SANDSTONE to SILTSTONE - fresh - medium strong to very strong		HQ	11	92	78		
				HQ	12	100	80		
10.82	6.24	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 ●

△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test ◣ Torvane

BOREHOLE RECORD

Stantec

CLIENT	Harbourside Engineering Consultants
PROJECT	Northwest River Bridge Reconstruction
LOCATION	Terra Nova National Park, NL
DATES (yy-mm-dd): BORING	14-11-26 to 14-11-27
WATER LEVEL	6.6m on 14-12-1
BOREHOLE No.	BH-02
PAGE	1 of 1
PROJECT No.	121618147
DRILLING METHOD	Wash boring
SIZE	
DATUM	Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY OR TCR(%) mm	N-VALUE OR RQD (%)	OTHER TESTS
0	17.04	ASPHALT					
	16.89	FILL: grey rockfill - sub-angular to angular	SS	1	381	55	
	16.29	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till	SS	2	305	39	
			SS	3	305	25	
			SS	4	229	26	
			SS	5	102	16	
			SS	6	0	12	
			SS	7	104	21	
		- 300 mm boulder at 4.4 m depth	SS	8	0	17	
			SS	9	0	12	
			SS	10	0	20	
		- 250 mm cobble at 7.1 m depth	SS	11	51	16	
	9.42	Good quality light grey SANDSTONE to SILTSTONE - fresh - medium strong to very strong	HQ	12	100	61	
			HQ	13	100	83	
	6.34		HQ	14	100	100	
		End of Borehole					

Water level symbol at approx. 6.6 m 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 ●

10 20 30 40 50 60 70 80

△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test Torvane

BOREHOLE RECORD

Stantec

BOREHOLE No. BH-03
PAGE 1 of 1

CLIENT: Harbourside Engineering Consultants
PROJECT: Northwest River Bridge Reconstruction
LOCATION: Terra Nova National Park, NL
DATES (yy-mm-dd): BORING 14-11-28 to 14-11-28
WATER LEVEL: 7m on 14-12-1
PROJECT No. 121618147
DRILLING METHOD: Wash boring
SIZE:
DATUM: Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	TYPE	NUMBER	RECOVERY OR TCR(%) (mm)	N-VALUE OR RQD (%)	OTHER TESTS
0	17.04	ASPHALT					
	16.84	FILL: grey rockfill - sub-angular to angular	SS	1	381	35	
	16.54	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till	SS	2	330	33	
			SS	3	178	29	
			SS	4	406	34	
			SS	5	178	15	
			SS	6	229	35	
			SS	7	0	43	
			SS	8	0	23	
			SS	9	127	80/178	
	10.77	Poor quality to good quality purple to light grey SANDSTONE to SILTSTONE - fresh - weak to very strong	HQ	10	66	57	
			HQ	11	100	50	
			HQ	12	100	95	
			HQ	13	100	90	
	5.31	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 ●

10 20 30 40 50 60 70 80

Water level symbol at approx. 7 m depth.

△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test ◣ Torvane

BOREHOLE RECORD

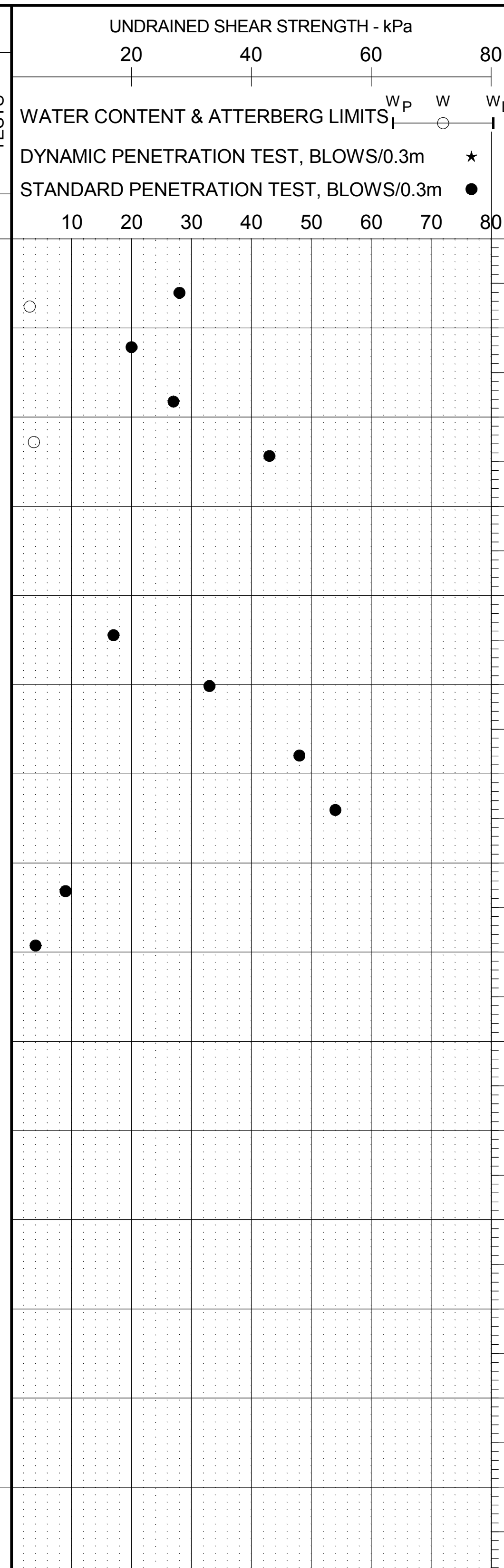
Stantec

CLIENT	Harbourside Engineering Consultants
PROJECT	Northwest River Bridge Reconstruction
LOCATION	Terra Nova National Park, NL
DATES (yy-mm-dd): BORING	14-11-30 to 14-12-1
WATER LEVEL	7.2m on 14-12-1
BOREHOLE No.	BH-04
PAGE	1 of 1
PROJECT No.	121618147
DRILLING METHOD	Wash boring
SIZE	
DATUM	Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT
0	17.04	ASPHALT	
0.2	16.84	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till	
8.53	8.51	Poor to excellent quality light grey SANDSTONE to SILTSTONE - fresh to slightly weathered - weak to very strong - silt seam at 10.0 m depth	
14.0	3.04	End of Borehole	

WATER LEVEL: ∇ at approx. 7.2 m

TYPE	NUMBER	RECOVERY OR TCR(%) (mm)	N-VALUE OR RQD (%)	OTHER TESTS
SS	1	406	28	S
SS	2	254	20	
SS	3	381	27	
SS	4	406	43	S
SS	5	152	17	
SS	6	127	33	
SS	7	229	48	
SS	8	254	54	
SS	9	0	9	
SS	10	0	4	
HQ	11	100	83	
HQ	12	100	52	
HQ	13	100	70	
HQ	14	97	78	



△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test ◪ Torvane

BOREHOLE RECORD

Stantec

CLIENT	Harbourside Engineering Consultants
PROJECT	Northwest River Bridge Reconstruction
LOCATION	Terra Nova National Park, NL
DATES (yy-mm-dd): BORING	14-12-1 to 14-12-1
WATER LEVEL	3.4m on 14-12-1
BOREHOLE No.	BH-05
PAGE	1 of 1
PROJECT No.	121618147
DRILLING METHOD	Wash boring
SIZE	
DATUM	Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY OR TCR(%) (mm)	N-VALUE OR RQD (%)	OTHER TESTS	STANDARD PENETRATION TEST, BLOWS/0.3m
0	24.22	ASPHALT								
	24.02	FILL: grey rockfill - sub-angular to angular			SS	1	356	44		●
	23.72	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till			SS	2	330	41		●
					SS	3	432	36		●
					SS	4	381	56		●
	20.87	Dense grey poorly graded sand with silt and gravel (SP-SM) to silty sand (SM) with gravel TILL - with occasional boulders, - with some to frequent cobbles - angular to sub-angular		∇	SS	5	457	32		●
					SS	6	381	161/405		
	20.16	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 ●

10 20 30 40 50 60 70 80

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test ◩ Torvane

BOREHOLE RECORD

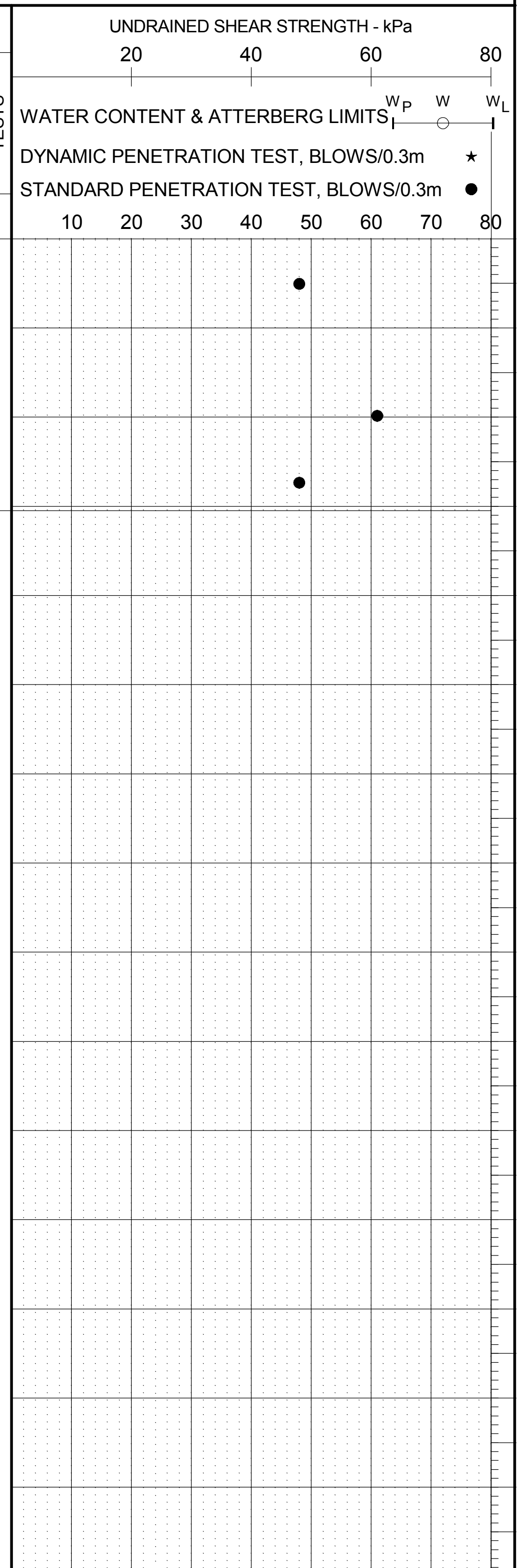
Stantec

BOREHOLE No. BH-06
PAGE 1 of 1

CLIENT: Harbourside Engineering Consultants
PROJECT: Northwest River Bridge Reconstruction
LOCATION: Terra Nova National Park, NL
DATES (yy-mm-dd): BORING 14-11-28 to 14-11-29
WATER LEVEL: 1.6m on 14-11-28

PROJECT No. 121618147
DRILLING METHOD: Wash boring
SIZE:
DATUM: Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY OR TCR(%) (mm)	N-VALUE OR RQD (%)	OTHER TESTS
0	19.75						
	19.55	ASPHALT					
	19.25	FILL: grey rockfill - sub-angular to angular	SS	1	356	48	
	18.08	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till	SS	2	152	71/305	
			SS	3	0	50/0	
	16.70	Dense grey poorly graded sand with silt and gravel (SP-SM) to silty sand with gravel (SM) TILL - with occasional boulders, - with some to frequent cobbles - angular to sub-angular	SS	4	203	61	
			SS	5	229	48	
		End of Borehole					



△ Unconfined Compression Test
□ Field Vane Test ■ (Remolded)
◇ Fall Cone Test ◆ (Remolded)
▽ Hand Penetrometer Test ◪ Torvane

TEST PIT RECORD

Stantec

CLIENT: Harbourside Engineering Consultants
PROJECT: Northwest River Bridge Reconstruction
LOCATION: Terra Nova National Park, NL
DATES (yy-mm-dd): DUG 14-12-1 14-12-1
WATER LEVEL: 1.2m on 14-12-1

TEST PIT No.: TP-01
PROJECT No.: 121618147
DATUM: Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	17.80						
	17.50	FILL: grey rockfill - sub-angular to angular					
		FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till			BS	1	
1	16.58			▽			
	15.97	Dense grey poorly graded sand with silt and gravel (SW-SM) to silty sand with gravel (SM) TILL - with occasional boulders, - with some to frequent cobbles - sub-rounded to sub-angular			BS	2	S
		End of Test Pit Slow water seepage observed at 1.2 m depth below ground surface.					

UNDRAINED SHEAR STRENGTH - kPa ★

20 40 60 80

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

10 20 30 40 50 60 70 80

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

TEST PIT RECORD

Stantec

CLIENT	Harbourside Engineering Consultants
PROJECT	Northwest River Bridge Reconstruction
LOCATION	Terra Nova National Park, NL
DATES (yy-mm-dd): DUG	14-12-1 14-12-1
WATER LEVEL	N/A
TEST PIT No.	TP-02
PROJECT No.	121618147
DATUM	Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	OTHER TESTS
0	19.60						
	19.30	FILL: grey rockfill - sub-angular to angular			BS	1	
1	18.38	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till					
	18.08	Dense grey poorly graded sand with silt and gravel (SW-SM) to silty sand with gravel (SM) TILL - with occasional boulders, - with some to frequent cobbles - sub-rounded to sub-angular					
		End of Test Pit Surface water infilling test pit					

UNDRAINED SHEAR STRENGTH - kPa ★

20 40 60 80

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

10 20 30 40 50 60 70 80

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Stantec

TEST PIT RECORD

CLIENT Harbourside Engineering Consultants
PROJECT Northwest River Bridge Reconstruction
LOCATION Terra Nova National Park, NL
DATES (yy-mm-dd): DUG 14-12-1 14-12-1 WATER LEVEL N/A

TEST PIT No. TP-03
PROJECT No. 121618147
DATUM Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	OTHER TESTS
0	21.80				
	21.50	FILL: grey rockfill - sub-angular to angular	BS	1	
	20.89	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till	BS	2	S
	20.28	Dense grey poorly graded sand with silt and gravel (SW-SM) to silty sand with gravel (SM) TILL - with occasional boulders, - with some to frequent cobbles - sub-rounded to sub-angular			

End of Test Pit

Surface water infilling test pit

UNDRAINED SHEAR STRENGTH - kPa ★ (20, 40, 60, 80)

WATER CONTENT & ATTERBERG LIMITS W_P W W_L (10, 20, 30, 40, 50, 60, 70, 80)

Depth scale 0–15 m.

Stantec

TEST PIT RECORD

CLIENT Harbourside Engineering Consultants
PROJECT Northwest River Bridge Reconstruction
LOCATION Terra Nova National Park, NL
DATES (yy-mm-dd): DUG 14-12-1 14-12-1 WATER LEVEL N/A

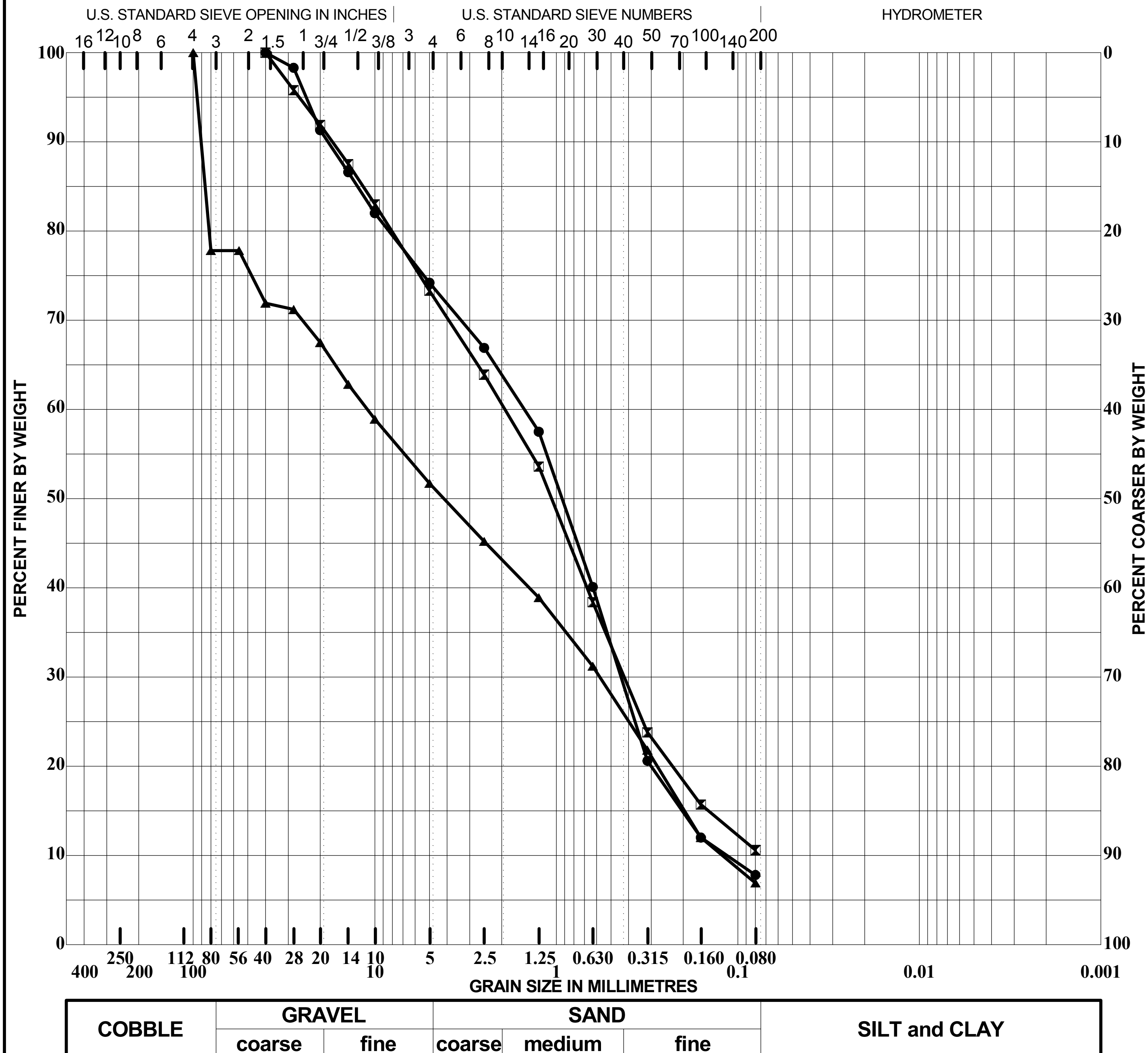
TEST PIT No. TP-04
PROJECT No. 121618147
DATUM Geodetic (Approx.)

DEPTH (m)	ELEVATION (m)	DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES OTHER TESTS
0	20.50						
	20.20	FILL: grey rockfill - sub-angular to angular					
					BS	1	
	19.29	FILL: brown sand with silt and gravel - with occasional cobbles and boulders - sub-angular to rounded gravel - reworked till					
		End of Test Pit					

UNDRAINED SHEAR STRENGTH - kPa ★ (20, 40, 60, 80)

WATER CONTENT & ATTERBERG LIMITS W_P W W_L (10, 20, 30, 40, 50, 60, 70, 80)

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15



	Sample	Depth (m)	Description	W%	W_L	W_P	I_P
●	BH-04 SS1	0.76	FILL: well-graded sand with silt and gravel	3.1			
⧗	BH-04 SS4	2.28	FILL: well-graded SAND with silt and gravel	3.8			
▲	TP-01 BS2	1.52	Poorly graded sand with silt and gravel (SP-SM) TILL	14.0			

	Sample	Depth (m)	D100	D60	D30	D10	%Gravel	%Sand	%Silt / Clay
●	BH-04 SS1	0.76	40.00	1.50	0.440	0.1150	25.8	66.4	7.8
⧗	BH-04 SS4	2.28	40.00	1.92	0.423		26.7	62.7	10.6
▲	TP-01 BS2	1.52	100.00	11.00	0.577	0.1219	26.1	44.8	6.9

REMARKS:

Stantec

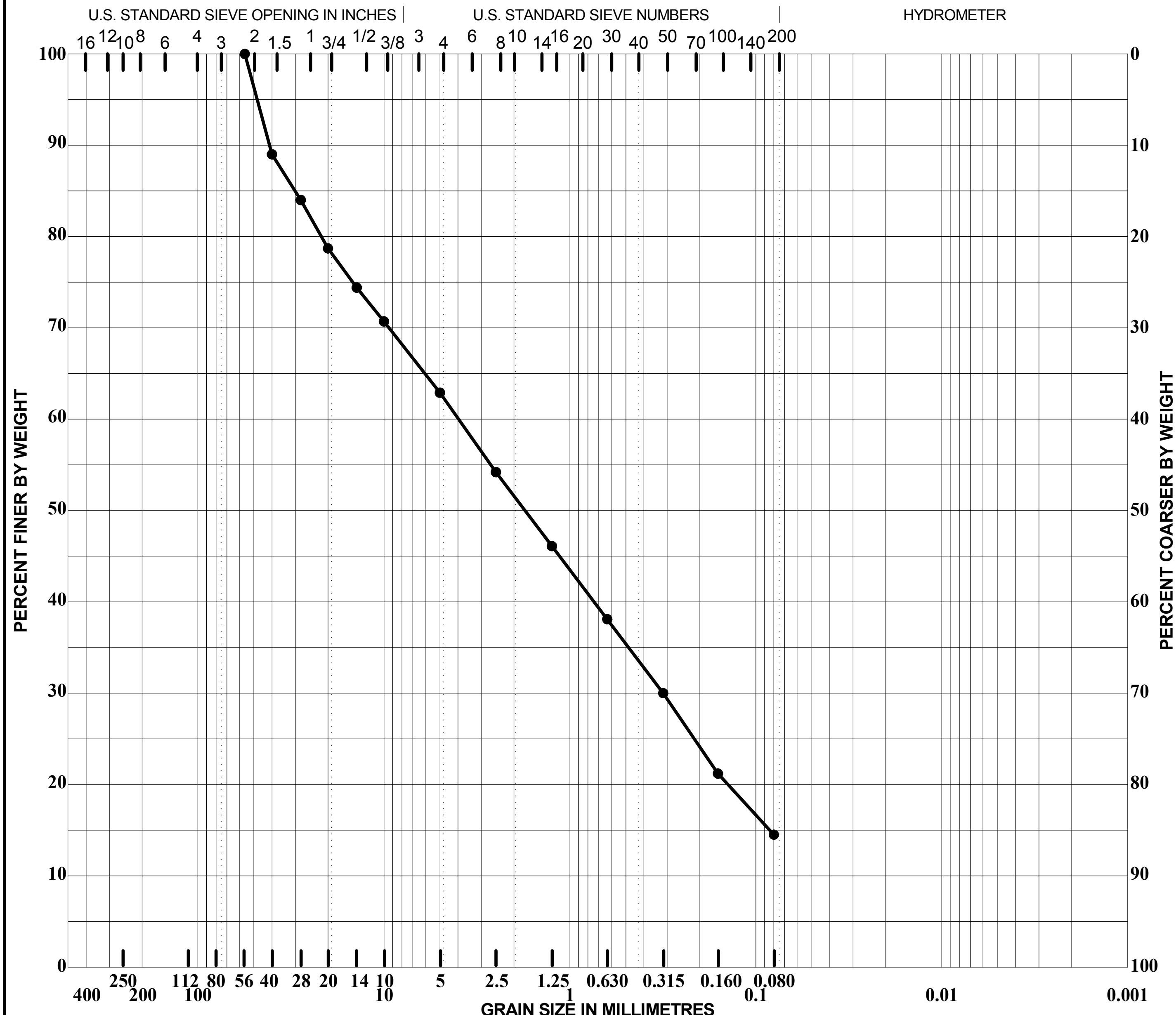
Client: **Harbourside Engineering Consultants**

Project: **Northwest River Bridge Reconstruction**

Project No.: **121618147**

Location: **Terra Nova National Park, NL**

GRADATION CURVES



COBBLE	GRAVEL		SAND			SILT and CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (m)	Description	W%	W_L	W_P	I_P
●	TP-03 BS2	1.21	Silty sand with gravel (SM) TILL	8.9			

	Sample	Depth (m)	D100	D60	D30	D10	%Gravel	%Sand	%Silt / Clay
●	TP-03 BS2	1.21	56.00	3.97	0.315		37.1	48.4	14.5

REMARKS:

Stantec

Client: **Harbourside Engineering Consultants**

Project: **Northwest River Bridge Reconstruction**

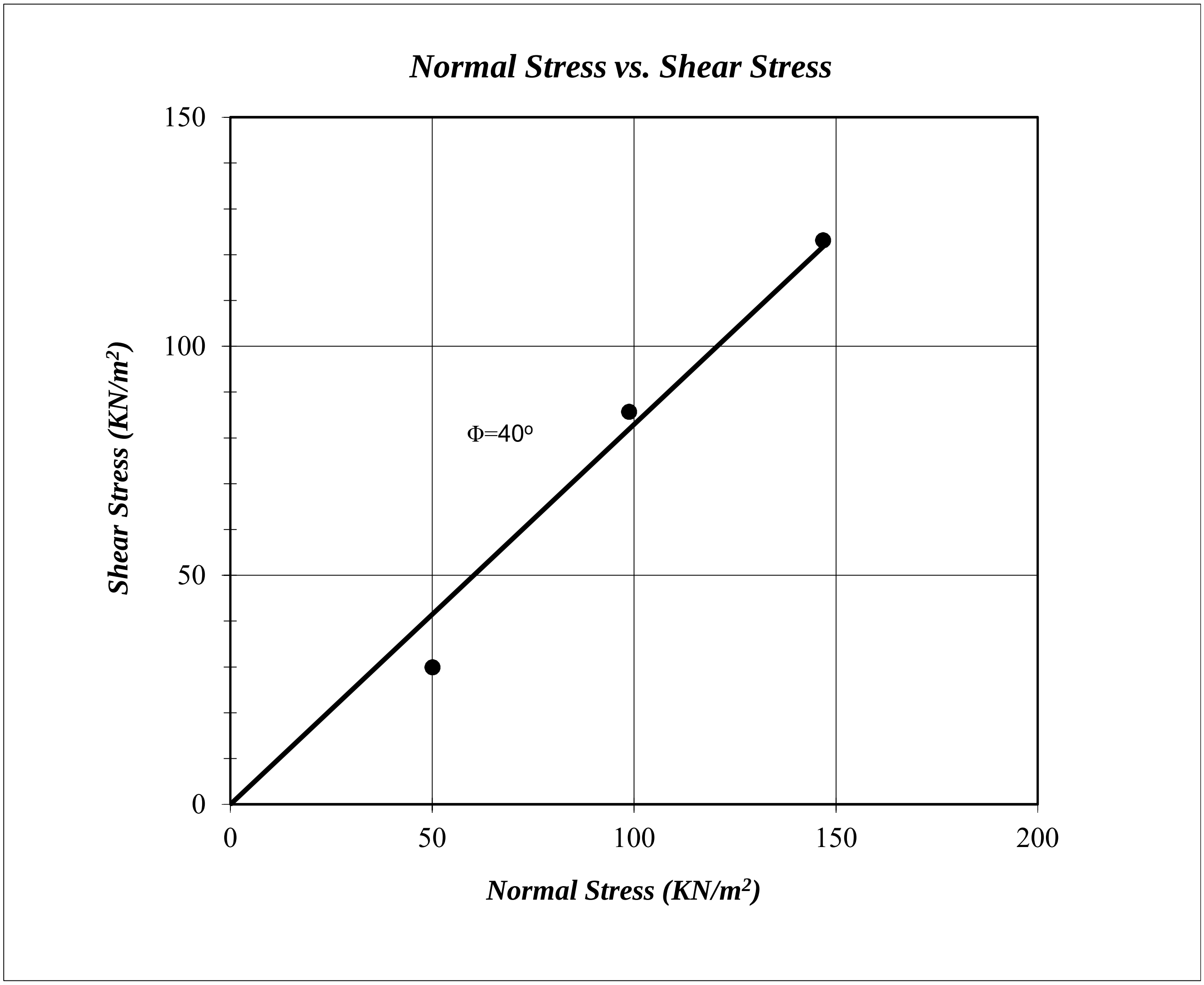
Project No.: **121618147**

Location: **Terra Nova National Park, NL**

GRADATION CURVES

Direct Shear Test

Northwest River Bridge, Harbourside Engineering Consulting (25 mm minus)
Stantec Project No: 121618147
Test date: Dec 18/2014



Normal Stress	Bulk Unit Weight	Dry Unit Weight
50 kPa	22.6 kN/m^3	21.3 kN/m^3
99 kPa	23.4 kN/m^3	22.0 kN/m^3
147 Kpa	23.4 kN/m^3	22.0 kN/m^3

19-Dec-14

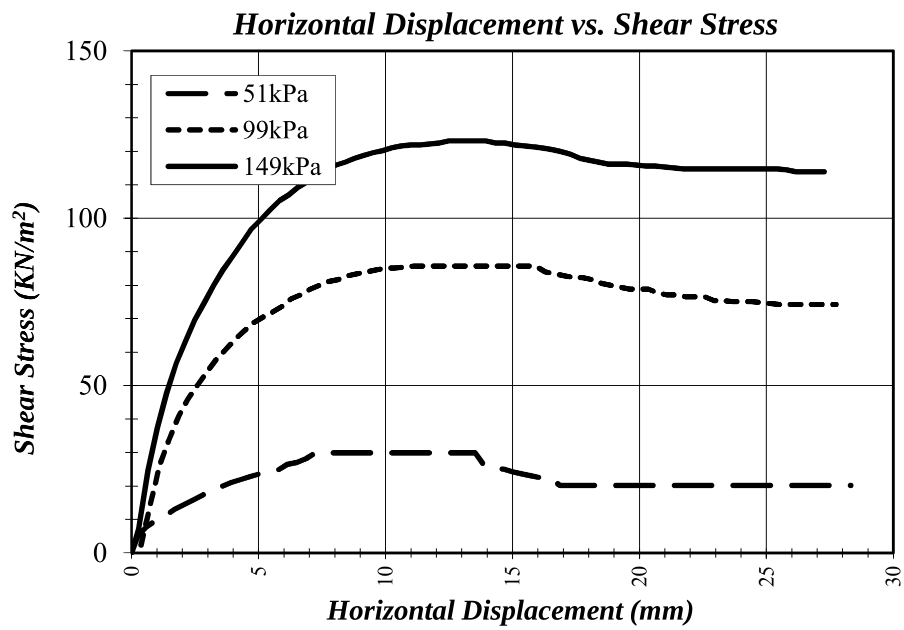
19-Dec-14

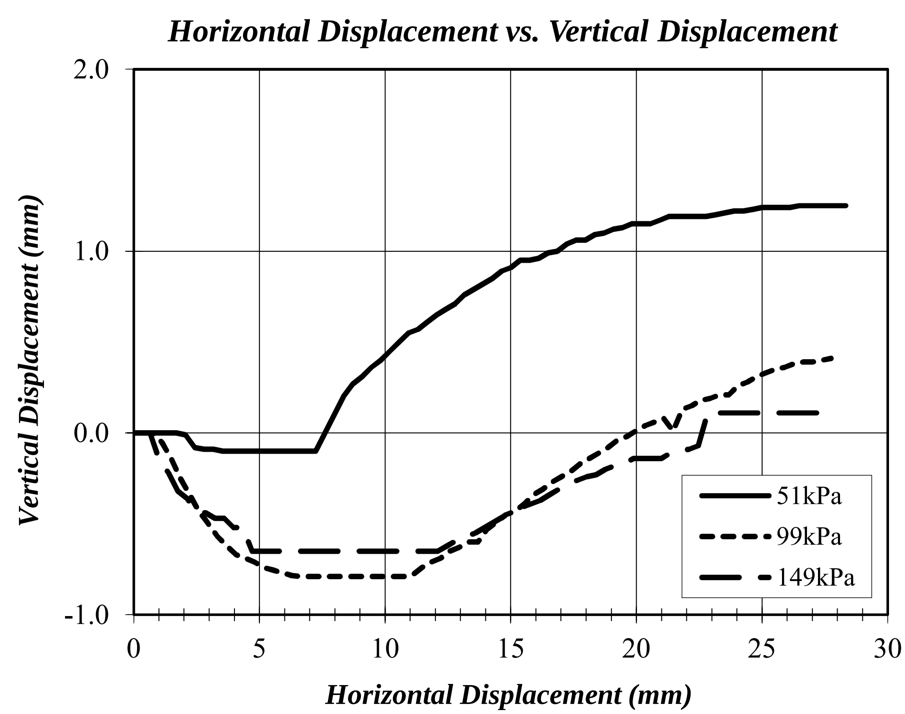
Direct Shear Test

Northwest River Bridge, Harbourside Engineering Consulting (25 mm minus)

Stantec Project No: 121618147

Test date: Dec 18/2014





STANTEC CONSULTING LTD
COMPRESSIVE STRENGTH OF ROCK CORES
ASTM D7012-10

PROJECT: Northwest River Bridge Reconstruction
CLIENT: Harbourside Engineering Consultants
CORE LOCATION:
DATE TESTED: December 9, 2014
DATE SAMPLED:

STANTEC PROJECT #: 121618147
CORE CONDITIONING:
DIRECTION DRILLED:
CORE DESCRIPTION: See report

BOREHOLE #	SAMPLE #	DIAMETER (mm)	Length (mm) TRIMMED	L/D RATIO	AREA (mm^2)	WEIGHT IN AIR (g)	WEIGHT IN WATER (g)	DENSITY (kg/m^3)	GAUGE LOAD (kN)	CORRECTION L/D	COMPRESSIVE STRENGTH, MPa ACTUAL	COMPRESSIVE STRENGTH, MPa CORRECTED MPa
BH-02 (7.9m)	HQ-1	63.0	145.0	2.30	3117	1245.2	790.5	2739	136.0	1.0000	43.6	43.6
BH-02 (10.1m)	HQ-2	63.0	143.0	2.27	3117	1236.8	786.7	2748	321.0	1.0000	103.0	103.0
BH-03 (7.0m)	HQ-1	63.0	141.0	2.24	3117	1201.1	756.0	2698	413.0	1.0000	132.5	132.5
BH-03 (7.2m)	HQ-2	63.0	143.0	2.27	3117	1221.7	768.6	2696	399.0	1.0000	128.0	128.0
							Average:	2720.3			Average:	101.8
											Range 43.6	to 132.5

Remarks:
1) Cores were tested in accordance with ASTM D7012-10.

APPROVED BY: VCG