DRAFT GEOTECHNICAL INVESTIGATION REPORT

PROPOSED WHARF REPLACEMENT LOWER NICHOLSON LOCKSTATION ANDREWSVILLE, ONTARIO

Prepared for:

Parks Canada

Date: September 2016

Project No. 161-08318-00

WSP Canada Inc.
2611 Queensview Drive,
Ottawa, ON K2B 8K2 Canada

Phone: 613-829-2800
Fax: 613-829-8299
www.wspgroup.com

TABLE OF CONTENTS

APPENDICES

1 INTRODUCTION

1.1 CONTEXT

WSP Canada Inc. (WSP) was retained by Parks Canada to conduct a geotechnical investigation as part the design of a proposed wharf replacement at the Lower Nicholson Lock Station, located near Andrewsville ON.

The purpose of the geotechnical investigation was to obtain subsurface information at the site by means of exploratory boreholes. This report presents the findings of the investigation and provides comments and recommendations related to the geotechnical aspects of the project.

1.2 PROJECT AND SITE DESCRIPTION

The project site is located at the Lower Nicholson Lock Station, on the east side of the Rideau River Valley, downstream of Merrickville, ON. The project includes the removal and replacement of an existing wharf structure located on the east side of the river, downstream of the lower of two locks.

The Wharf and surrounding ground are located on the east side of the Rideau River valley at the beginning of an outside bend in the river. The River valley in the general area is relatively steep and heavily vegetated, with a flat to gently sloping upland. At the wharf location, the wharf and locks are built into a relatively flat area which could have been a natural terrace in the river valley, filled in during construction of the locks, or a combination of both. This terrace area is relatively wide behind the wharf extending 15 m or more from the back of the structure. It is likely that some portions of the ground behind the wharf have been constructed by filling in slow moving portions of the river channel.

Photo 1 – Existing Wharf Structure

The wharf is a rock-filled timber crib structure approximately 40 m in length. The structure is approximately 2.5 m in height, of which approximately 2 m was below water at the time of the field investigation. The structure is topped with a concrete deck approximately 2.4 m in width. The existing timber cribbing is showing signs of distress, and the wharf appears to be tilting towards the water.

It is understood the replacement wharf will be similar in size to the existing, and constructed in approximately the same location. Several types of structure are being considered at this time including:

- A simple gravity structure (for example a similar crib structure, or a series of steel cells filled with granular material;
- A segmental retaining wall structure (pre-cast concrete blocks with tie-backs and anchors as required); or
- A structural steel wall (such as H-piles with some form of lagging) set into rock, with or without tie-backs and anchors.

All of the proposed options will incorporate a concrete deck similar to the existing structure.

1.3 LIMITATIONS

The current report was prepared at the request and for the sole use of Parks Canada and according to the specific terms of the mandate given to WSP. The use of this report by a third party, as well as any decision based upon this report, is under this party's sole responsibility. WSP may not be held accountable for any possible damages resulting from third party decisions based on this report.

Furthermore, any opinions regarding conformity with laws and regulations expressed in this report are technical in nature; the report is not and shall not, in any case, be considered as a legal opinion.

Additional Limitations of this Report are presented in Appendix E, and form an integral part of this document.

2 SITE INVESTIGATION

2.1 SCOPE OF WORK

The scope of work for this assignment included:

- A desk study and review of existing geotechnical information in the general area;
- Laying out the boreholes and obtaining utility locates at the project site;
- Drilling of three exploratory boreholes at the project site;
- In-situ soil sampling and testing, including Standard Penetration (SPT) Testing;
- Obtaining soil and rock samples for additional review and laboratory testing;
- Laboratory testing;
- Geotechnical analysis; and
- Preparation of this report which presents the results of the investigation and provides geotechnical recommendations related to the design and construction of the proposed new wharf structure.

2.2 INVESTIGATION PROCEDURES

The geotechnical investigation was carried out in June, 2016.

2.2.1 DESKTOP STUDY

Surficial geology maps indicate that the area is underlain by modern alluvial deposits (clay, silt, sand and gravel which may contain organic material) as well as glacio-marine deposits (sand and gravel with minor silt and clay). Bedrock geology maps indicate the rock in the general area includes dolostone and sandstone of the Beekmantown Formation.

2.2.2 FIELD INVESTIGATION

The field investigation was carried out on June 21, 2016 and included the drilling of three boreholes in the area around the existing wharf, as shown on Drawing No. 2.

The boreholes were advanced using a track-mounted drill rig supplied and operated by George Downing Estates Drilling (Downing) of Hawkesbury, Ontario. The boreholes were advanced using hollow-stem augers to auger refusal at a maximum depth of 4.2 m below the existing ground surface. Upon meeting auger refusal in Borehole BH16-1, drilling was extended a depth of 6.2 m by means of NQ size rotary diamond coring equipment. Soil and rock samples retrieved during drilling were logged and visually classified in the field by a member of WSP's geotechnical staff. In-situ tests including Standard Penetration (SPT) Testing were carried out at regular intervals.

Water level observations were made during drilling and in the open boreholes at the completion of the drilling operations. A standpipe piezometer was installed in Borehole BH16-2 to allow for measurement of stabilized groundwater levels.

Borehole logs are included in Appendix B of this report.

2.2.3 LABORATORY TESTING

Upon completion of drilling and in-situ testing, soil samples were returned to WSP's laboratory for further examination, classification and testing. A laboratory testing program, carried out on selected representative soil samples, included the determination of natural water content, grain size distribution, and chemical analyses of soil corrosivity.

The results of natural water content tests are included on the relevant borehole logs in Appendix B. The results of the grain size distributions are summarized on the individual borehole logs and presented in Appendix C. Chemical testing to determine sulphate content, chloride content, pH and resistivity was also carried out on selected soil samples obtained during drilling. The results of these tests are included in Appendix C.

3 SUBSURFACE GEOTECHNICAL CONDITIONS

The subsurface soil profile at the site generally consists of a topsoil layer overlying a mixture of granular and cohesive soils, underlain by Limestone bedrock at relatively shallow depth.

Specific descriptions of the various soils encountered are presented below, as well as in the individual borehole logs.

3.1 SOIL CONDITIONS

3.1.1 TOPSOIL

A layer of topsoil was encountered at all three borehole locations (which were drilled in the grassed area adjacent to the wharf). The topsoil thickness ranged from 270 mm to 310 mm at the borehole locations.

3.1.2 SILTY CLAY/CLAYEY SILT

Fine-grained cohesive soils were encountered in Boreholes BH16-1 and BH16-3 comprising silty clay and clayey silt. Based on SPT ‘N’ values, the silty clay immediately behind the wharf (in BH16-1) was found to be stiff in consistency (with SPT ‘N’ values of 10 and 11 blows per 300 mm penetration). At Borehole BH16-3, located further away from the wall the cohesive soils were found to be soft in consistency (with ‘N’ values of 2 to 4 blows per 300 mm of penetration). These cohesive soils extended to a depth of 1.2 m and 2.1 m at BH16-1 and BH16-3, respectively.

3.1.3 SILTY SAND AND GRAVEL/SILTY SAND

Underlying the cohesive soils in BH16-1 and BH16-3, and the topsoil in BH16-2 silty sand, as well as silty sand and gravel were encountered. The silty sand and gravel immediately behind the wall (in BH16-1 and BH16-2 ranged from loose to compact, with SPT ‘N’ values of 2 to 24 blows per 300 mm of penetration, and was somewhat coarser in texture. This material may have been placed as backfill behind the existing wharf structure. The silty sand in Borehole BH16-3 was found to be finer in texture, very loose (SPT ‘N’ value of 2 blows per 300 mm) and contained organic material. Granular soils extended to a depth of 2.1 m and 2.2 m in Boreholes BH16-1 and BH16-2 located near the wharf. At Borehole BH16-3, located to the north of the structure, the silty sand extended to a depth of 3.1 m below the existing ground surface.

3.1.4 GLACIAL TILL

A layer of glacial till was encountered in all three of the boreholes. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of clay, silt and sand. This deposit extended to the depth of refusal ranging 2.7 m to 4.2 m below the existing ground surface. Standard penetration test ‘N’ values within the glacial till ranged from 16 to 25 blows per 300 mm of penetration indicating a compact state of packing.

Grain size curves for selected samples of the glacial till are presented in Appendix C. A summary of these grain size distributions is also presented in the table below.

Table 1 – Results of Grain Size Analyses for Glacial Till

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH16-1	SS4	8	21	53	18
BH16-2	SS4	8	41	39	12
BH16-3	SS5	7	36	41	16

3.1.5 AUGER REFUSAL/BEDROCK

Auger refusal was encountered in all three boreholes at depths ranging from 2.7 m to 4.2 m below the existing ground surface. Borehole BH16-1 was extended beyond auger refusal depth and the bedrock was cored using “N” sized diamond coring equipment.

The rock encountered in Borehole BH16-1 includes fresh to slightly weathered, thinly bedded limestone with moderately closely spaced horizontal joints. Rock Quality Designation (RQD) ranged 48% to 87% indicating a rock quality ranging from “poor” to “good” with the majority of the values in the “good” range.

3.2 GROUNDWATER CONDITIONS

A standpipe piezometer was installed in Borehole BH16-1 during the geotechnical investigation to allow for the measurement of stabilized groundwater levels. Groundwater levels were measured on six days after drilling (in June, 2016) and were found to be 0.46 m below the existing ground surface, which was approximately coincident with the level of the river at the time of the measurement.

Groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events. Given the proximity of the site to the Rideau River, the groundwater level should also be expected to fluctuate with the river level.

3.3 SUMMARY

A summary of the subsurface conditions encountered during drilling are presented in the table below.

Table 2 –Simplified Stratigraphy and Groundwater Elevations

BH No.	Simplified Stratigraphy (Depth in metres)						Notes
	Topsoil	Silty Clay & Clayey Silt	Silty Sand/Silty Sand and Gravel	Glacial Till	Auger Refusal	Bedrock (Cored)	
BH16-1	0.0 – 0.3	0.3 – 1.2	1.2 – 2.1	2.1 – 2.7	2.7	2.7 – 6.2	--
BH16-2	0.0 – 0.3	--	0.0 – 2.2	2.2 – 3.1	3.1	--	Piezometer installed. Measured GWL at 0.46 m (June 2016)
BH16-3	0.0 – 0.3	0.3 – 2.1	2.1 – 3.1	3.1 – 4.2	4.2	--	--

4 RECOMMENDATIONS

4.1 GENERAL

This section of the report provides engineering guidelines related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and our understanding of the project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

4.2 FROST PROTECTION

The depth of frost penetration for the site is 2.1 m. Foundations of unheated structures placed on soil should be provided with a minimum of 2.1 m of earth cover (or equivalent insulation). The rock present at the site generally would not be considered susceptible to frost heave, but this assumption should be reviewed and confirmed during construction by means of localized drilling and inspection to confirm there are no frost-susceptible layers within the rock mass.

If the foundations will be permanently submerged (i.e. the river level does not fluctuate enough to expose the foundations to sub-zero temperatures, and the water around the structure foundations will remain unfrozen) then frost protection would not be required.

4.3 SEISMIC SITE CLASSIFICATION

The site may be assumed to be Class ‘C’ for seismic site response. It is possible that a more favourable site classification (‘A’ or ‘B’) could be used, but this would require site specific measurement of shear wave velocities.

4.4 FOUNDATIONS

It is understood that there are several types of structure being considered. For conventional gravity structures, such as crib structures, segmental retaining walls or granular-filled steel bin wall type structures conventional shallow foundations may be used. For an ‘H’-pile type structure (either cantilevered or tied-back) rock sockets would likely be required.

4.4.1 SHALLOW FOUNDATIONS ON ROCK

Shallow foundations for the new wharf structure could be placed on rock, which was found to be present at 2.7 m below the existing ground surface at Borehole BH16-1 and was inferred to be at approximately 3.1 m depth at Borehole BH16-2. It is understood the existing wharf is approximately 2.5 m in height, and therefore likely founded on or just above rock. Given that the existing structure will need to be removed it is likely the simplest approach will be to found the new structure on rock.

The unfactored bearing resistance of foundations placed on the existing rock may be taken as 2.5 MPa. A geotechnical resistance factor of 0.5 should be applied to this, for a factored resistance at ULS of 1.25 MPa.

The settlement of rock associated with these bearing pressures is typically significantly less than the 25 mm normally accepted and therefore SLS conditions generally do not govern the design of foundations constructed on rock.

It should be noted that the rock surface is unlikely to be even, and some filling/levelling with concrete, as well as potentially excavation of high points is likely to be required in order to provide a flat base for the new structure. In the event the exposed rock surface is found to be sloping dowels can be installed to provide additional sliding resistance (if required).

All bearing surfaces should be checked, evaluated and approved at the time of construction by a geotechnical engineer who is familiar with the findings of this investigation and the design and construction of similar structures prior to placement of any concrete.

4.4.2 ROCK SOCKETED CAISSONS OR PILES

COMPRESSIVE RESISTANCE

The compressive resistance of drilled and cast-in-place piles (caissons or steel piles set in pre-drilled and concreted sockets) which incorporate rock sockets will be a function of the shaft resistance of the socket. For design purposes, the unfactored shaft resistance of a socket in sound rock may be taken as 3.0 MPa. A geotechnical resistance factor of 0.4 should be applied to this value resulting in a factored resistance at ULS of 1.2 MPa. Because of the difficulty in ensuring a clean base, end resistance is typically ignored in assessing the compressive resistance of small-diameter caissons.

The displacements required to reach the ULS condition in a properly constructed rock socket are typically small and therefore SLS considerations do not generally govern the design of caissons socketed in sound rock.

The above resistances assume a minimum centre-to-centre caisson spacing of 3 times the caisson diameter. If caisson groups are constructed with more closely spaced caissons then the individual caisson capacities should be reduced to account for overlap of vertical caisson loads. Additional assistance during detailed design if closely spaced caissons are required.

UPLIFT RESISTANCE

For the purposes of determining the uplift capacity of drilled, rock-socketed caissons the unfactored ultimate shaft resistance within the rock sockets may be assumed to be 3.0 MPa. A geotechnical resistance factor of 0.3 should be applied to this value, resulting in a factored resistance at ULS of 0.9 MPa. The dead weight of the caisson itself (with an appropriate structural resistance factor for dead weight) may also be added to the geotechnical resistance in calculating the total uplift resistance.

The total uplift resistance of a caisson group is the lesser of the sum of the individual caisson resistances as described above, or the resistance of a single “block” of soil and rock with a perimeter equal to the perimeter of the caisson group (the mass of the soil and rock inside the “block” may be included in the calculation; use a unit weight of 19 kN/m3 for soil and 26 kN/m3 for rock).

WSP should review the preliminary pile design geometry and design and provide additional comments as appropriate.

LATERAL RESISTANCE

The ultimate geotechnical resistance to lateral loading for a caisson embedded in sound rock may be estimated by limiting the horizontal bearing stress at the top and toe of the theoretical rock socket to an unfactored value of 2.0 MPa and a factored value of 1.0 MPa.

As with vertical loads, the displacements required to reach the ULS condition in a properly constructed rock socket are relatively small and therefore SLS considerations do not generally govern the lateral resistance of rock sockets.

CONSTRUCTION CONSIDERATIONS

In the event the rock sockets are installed prior to removal of the existing structure, the rock sockets will be drilled through overburden soils and potentially the existing structure which is likely to contain cobbles and boulders (rock fill), as well as potentially concrete, metal, timber, portions of the cribbing, and other obstructions. If drilled prior to dewatering, the bore will pass through sands and gravels below the water table which should be assumed to have virtually no “stand-up” time and will behave as flowing soils if left unsupported. Temporary steel casing will be required to prevent collapse of the sidewalls during drilling through overburden and the existing crib wall fill.

Groundwater should be expected during drilling of any rock sockets (irrespective of whether or not the site has been dewatered, or the level of water in the river). It is anticipated that groundwater inflow can be handled by pumping from the sockets provided the flow through the any overburden (if present) is appropriately cut off. There may, however, be locations where jointing of the rock mass results in higher groundwater flows and contractors should be prepared to deal with additional flow (for example by extending casing, pumping at an increased rate, placement of concrete by tremie, etc.) during construction.

The capacity of rock sockets is highly dependent upon the construction quality of the socket, which must be appropriately cleaned prior to concreting. It is recommended that contractors be required to submit their construction methodology (including type of equipment, drilling procedure, procedure for cleaning the socket, etc.) for review prior to beginning installation.

All deep foundation construction should be inspected on a full-time basis by qualified staff under the supervision of a geotechnical engineer.

4.5 EARTH PRESSURES

4.5.1 STATIC EARTH PRESSURES

The lateral earth pressures acting on the new wharf structure walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning earth pressures for the design of the new wharf structure. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the new structure (as exists currently). If design changes require sloping ground then the earth pressure coefficients should be adjusted accordingly, or the sloping ground added as a surcharge loading.

Table 3 – Earth Pressure Coefficients

Parameter	**Value (Unfactored)**	
Material	Granular A or Granular B	Existing Fill Material
Angle of Internal Friction (ϕ)	32 degrees	28 degrees
Unit Weight	22.0 kN/m^3 above the groundwater table; 12.2 kN/m^3 below the groundwater table	20.0 kN/m^3 above the groundwater table; 10.2 kN/m^3 below the groundwater table
Coefficient of Active Earth Pressure (k_a)	0.31	0.36
Coefficient of Earth Pressure at Rest (k_0)	0.47	0.53
Coefficient of Passive Earth Pressure (k_p)	1.9	1.5
Coefficient of Sliding Along Wall Base	0.35	

For gravity walls where some amount of movement can occur during backfilling active earth pressures may be used in the geotechnical design of the structure. For stiffer walls, or where deflection of the structure must be limited (for example deflection of cantilevered piles), at rest earth pressures should be assumed for design. The above values represent unfactored values.

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for design (i.e. where the calculated earth pressure is less than 12 kPa, use 12kPa). Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design, as required.

Free draining, non-frost-susceptible granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the new wharf structure.

4.5.2 SEISMIC EARTH PRESSURES

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the seismic earth pressures may be assumed to be:

$$P_{AE} = {}^{1}/_{2}\gamma H^{2}(1-k_{v})K_{AE}$$

and

$$P_{PE} = {}^{1}/_{2}\gamma H^{2}(1-k_{v})K_{PE}$$

Where P_{AE} = Seismic Active Earth Pressure (kN);

H = the total height of the wall (m);

k_v = vertical acceleration coefficient (use 0.2);

K_{AE} = the seismic active earth pressure coefficient (use 0.5);

P_{PE} = Seismic Active Earth Pressure (kN);

K_{PE} = the seismic passive earth pressure coefficient (use 4);

The above earth pressure values (both static and seismic) are unfactored values.

The seismic earth pressure component (P_{AE} – P_A) should be assumed to act at a height of 0.6H above the base of the wall (i.e. higher than the non-seismic earth pressure component, P_A, which is typically assumed to act at 0.33H).

4.6 ROCK ANCHORS

The ultimate geotechnical pull-out resistance provided by an anchor in limestone should be taken as the lesser of:

- The capacity of the anchor calculated using an unfactored bond stress of 1,500 kPa along the grout/rock interface for the upper 2 m of rock, and 3,000 kPa below.
- The buoyant weight of a cone of rock (and overlying soil) having an angle of 60 degrees from horizontal with the apex located at the tip of the anchor. The unit weight of the rock may be assumed to be 26 kN/m^3 above the water table and 16 kN/m^3 below the water table. For soil, a unit weight of 19 kN/m^3 and 9 kN/m^3 can be used above and below the water table, respectively.

Where multiple anchors are to be installed the total resistance of the group must consider the potential overlap of the theoretical cones of the rock masses stressed by individual anchors, in which case the weight should be the weight of the truncated cones. Further guidance can be provided during the detailed design phase if required based on the actual foundation and anchor geometry.

A geotechnical resistance factor of 0.4 should be applied to the total resistance obtained using the above calculations.

Typically, the displacement required to mobilize the full bond stress in rock is relatively small, and therefore for preliminary design the displacement of the grouted portion of the anchor at SLS can be assumed to be similarly small (typically less than 5 mm). WSP can confirm this assumption in the detailed design phase based on the actual anchor details.

The actual capacity of the soil and rock anchors should be confirmed, in accordance with PTI and CFEM guidelines. Permanent soil/rock anchors should be double-corrosion protected (Class I). Where a full-scale load test is completed the geotechnical resistance factor may be increased to 0.55.

4.7 CONSTRUCTION CONSIDERATIONS

4.7.1 EXCAVATIONS

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). If required, SPL can provide additional guidance based on preliminary excavation plans, depths, etc. during the detailed design phase of the project.

4.7.1.1 EXCAVATIONS IN SOIL

The soils at the site range from silt and clay to sand and gravel. Glacial till is present at the site which may contain cobbles and boulders. Fill material is present within the existing wharf structure (as well as potentially behind and around it) which may also contain cobbles, boulders, concrete, timber and other obstructions.

The soils immediately behind the wall were found to be a combination of stiff clay and compact sand and gravel, and may have been placed or compacted during construction of the wharf. The soils at a distance (in Borehole BH16-3) were found to include soft silt, and very loose silty sand as well as organic material. These soils may include a combination of unconsolidated sediment, colluvial material from the valley wall and fill material which may have historically been used to fill in a portion of the site.

For preliminary planning purposes these soils should be assumed to be Type 4 Soil and constructing a stable open excavation (particularly given the high groundwater level which will exist) may be difficult. Excavation support such as shoring or trench boxes may be required to maintain stable excavation within the soft clayey silt and very loose silty sand. These classifications must be reviewed and confirmed by a qualified person during excavation.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to additional loading of braced excavations or slope instability of unsupported excavations.

4.7.1.2 EXCAVATIONS IN ROCK

If bedrock excavation is required, shallow excavations in weaker or more heavily jointed rock may be feasible with mechanical excavating (i.e. hoe-ramming). Deeper excavations in more intact or competent rock are typically more economically made by blasting.

Excavations cut into the bedrock can be on a near-vertical face (say 10V:1H). The face of the excavation, however, must be scaled of any loose rock to protect the workers working in the excavation. Line drilling may be required to adequately define and control the extent of rock excavation.

Deep excavations in weathered, heavily jointed or previously disturbed rock may require temporary support to ensure stability and worker safety. All rock faces should be reviewed by a qualified person as excavated.

4.7.2 GROUNDWATER CONTROL

Groundwater levels at the site were found to be approximately 0.5 m below grade at the time of the investigation.

Excavations below the groundwater table (or river level) will have the potential to generate significant quantities of groundwater. If temporary excavation support includes coffer dams or cut-off walls which restrict groundwater flow to the underlying rock, then the inflow can likely be managed by pumping from sumps in the rock (because disturbance of the base will not be an issue). If open-cut excavations through soil are undertaken, or soil remains in place at the base of the excavation an active dewatering system (such as pumping from closely-spaced well points) will likely be required. Unsupported excavations below the water table will likely behave as “flowing” soils if sufficient dewatering is not in place during construction. A specialist dewatering contractor may be required to design and operate a dewatering system.

If required, WSP can provide additional guidance during detailed design with respect to groundwater quantities based on the size, depth and anticipated support conditions of proposed excavations.

4.8 CORROSION AND CEMENT TYPE

Samples of the existing soils were submitted to Exova Accutest for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix III and summarized in the table below.

Table 4 – Results of Soil Corrosivity Testing

Borehole No.	Chloride (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)	Sulphate (%)
BH16-2	0.004	0.18	7.6	5,560	0.02
BH16-3	< 0.002	0.13	8.5	7,690	< 0.01

The soil resistivity values measured in the fill suggest a low to moderate corrosivity environment for buried steel elements.

The test results also indicate low soluble sulphate content in the existing soils. For these values, sulphate-resistant cement is not required.

5 CLOSURE

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

WSP Canada Inc.

Prepared By:
Chris Hendry, M.Eng., P.Eng.
Senior Geotechnical Engineer

Appendix A

DRAWINGS

Client:	Parks Canada			Title:	Site Location Plan
Project#:	161-08318-00	DWG #:	1	Project:	Geotechnical Investigation Lower Nicholson's Lockstation
Drawn:	DW	Approved:	CH		
Date:	July 2016	Scale:	N. T. S.		
Size:	Letter	Rev:	0		

Client:	Parks Canada		
Project#:	161-08318-00	DWG #:	2
Drawn:	DW	Approved:	CH
Date:	July 2016	Scale:	N. T. S.
Size:	Letter	Rev:	0

Title:	Borehole Location Plan
Project:	Geotechnical Investigation Lower Nicholson's Lockstation

WSP

Appendix B

BOREHOLE LOGS AND CORE PHOTOGRAPHS

LOG OF BOREHOLE BH16-1

Project: Lower Nicholson Lockstation
Client: Parks Canada
Project Location: Lower Nicholson Lockstation
Datum: n/a
BH Location: See borehole location plan

DRILLING DATA
Rig Type: CME 55
Method: Hollow Stem Auger Drilling
Borehole Diameter: 203 mm
Core Diameter: 76 mm

Project No.: 161-08318-00
Date Started: 6/21/2016
Supervisor: DR
Reviewer: CH

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	Natural moisture content w (%)	Remarks and grain size distribution (%) GR SA SI CL
0.0	**TOPSOIL - 300 mm**					
0.3	**SILTY CLAY** some sand, trace gravel, brown, wet, stiff	1	SS	10	○	
		2	SS	11	○	
1.2	**SILTY SAND AND GRAVEL** brown, wet, compact	3	SS	20	○	
2.1	**SILTY CLAY** some sand, trace gravel, brown grey, wet, compact (GLACIAL TILL)	4	SS	25	w_P–w–w_L	8 21 53 18
2.7	**LIMESTONE** fresh, close to moderately closely spaced horizontal joints, thinly bedded, grey Run 1: 2.7 m to 3.1 m TCR - 100% SCR - 87% RQD - 87%	1	CORE			
	Run 2: 2.7 m to 4.7 m TCR - 100% SCR - 73% RQD - 48%	2	CORE			
	Run 3: 4.7 m to 6.2 m TCR - 100% SCR - 98% RQD - 80%	3	CORE			
6.2	**END OF BOREHOLE** 1) Auger refusal at 2.7 meters below existing elevation, switched to NQ coring. 2) Borehole was dry upon completion of sampling.					

Other column headings: STRATA PLOT; GROUND WATER CONDITIONS; ELEVATION; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE (25, 50, 75, 100, 125); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) (25, 50, 75); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (KN/m³); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

WSP SOIL LOG - OTTAWA NICHOLSON LOCKSTATION.GPJ SPL.GDT 9/22/16

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-2

Project: Lower Nicholson Lockstation
Client: Parks Canada
Project Location: Lower Nicholson Lockstation
Datum: n/a
BH Location: See borehole location plan

DRILLING DATA
Rig Type: CME 55
Method: Hollow Stem Auger Drilling
Borehole Diameter: 203 mm
Core Diameter: N/A

Project No.: 161-08318-00
Date Started: 6/21/2016
Supervisor: DR
Reviewer:CH

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL - 270 mm**						
0.3	**SILTY SAND AND GRAVEL** trace organics, brown, wet, compact	1	SS	5	Cuttings	○	
		2	SS	24	Bentonite; W. L. 1.1 mBGL Jun 29, 2016; Sand	○	
	- very loose below 1.5 m	3	SS	2		○	
2.2	**SAND AND SILT** trace gravel, trace to some clay, brown grey, wet, compact (GLACIAL TILL)	4	SS	16	Screen	○	8 41 39 12
3.1	**END OF BOREHOLE**						

1) Auger refusal at 3.1 meters below existing elevation.
2) 25 mm dia. piezometer was installed in the borehole upon completion.
3) Date Depth-groundwater

06/27/2016 0.46 m

Column headings also shown: STRATA PLOT; ELEVATION; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (KN/m³)

GROUNDWATER ELEVATIONS
Shallow/ Single Installation Deep/Dual Installation

GRAPH NOTES + 3 , × 3 : Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-3

Project: Lower Nicholson Lockstation
Client: Parks Canada
Project Location: Lower Nicholson Lockstation
Datum: n/a
BH Location: See borehole location plan

DRILLING DATA
Rig Type: CME 55
Method: Hollow Stem Auger Drilling
Borehole Diameter: 203 mm
Core Diameter: N/A

Project No.: 161-08318-00
Date Started: 6/21/2016
Supervisor: DR
Reviewer: CH

ELEV (m) DEPTH	DESCRIPTION	STRATA PLOT	SAMPLE NUMBER	SAMPLE TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	GR	SA	SI	CL
0.0	**TOPSOIL - 310 mm**														
0.3	**CLAYEY SILT** some gravel, some organics, brown, moist, soft		1	SS	4										
			2	SS	3										
			3	SS	2										
2.1	**SILTY SAND** trace organics, brown, moist, very loose		4	SS	2										
3.1	**SILTY SAND** some clay, some gravel, grey, moist, compact (GLACIAL TILL)		5	SS	19							7	36	41	16
3.7	**SILTY CLAY** some sand, some gravel, grey, moist (GLACIAL TILL)		6	SS	50/ 100 mm										
4.2	**END OF BOREHOLE**														

END OF BOREHOLE

1) Auger refusal at 4.2 meters below existing elevation.
2) Standing water 2.3 m below the existing surface elevation at end of drilling.

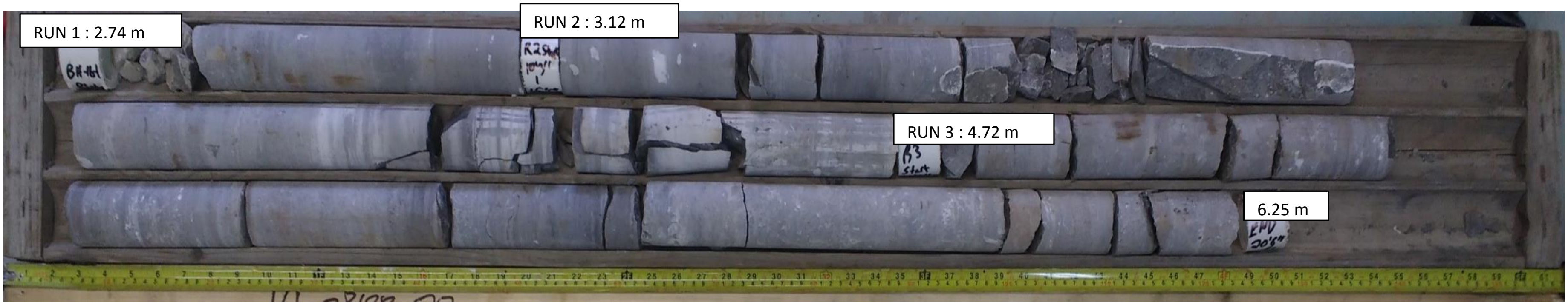
Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100. SHEAR STRENGTH (kPa): ○ UNCONFINED; + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL; × LAB VANE; 25 50 75 100 125. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L. WATER CONTENT (%): 25 50 75. REMARKS AND GRAIN SIZE DISTRIBUTION (%).

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

Borehole BH16-1



Client:	Parks Canada			Title:	Core Photograph
Project#:	161-08318-00	DWG #:		Project:	Geotechnical Investigation Lower Nicholson Lock Starion
Drawn:	DMR	Approved:	CH		
Date:	June 21/16	Scale:	N. T. S.		WSP
Size:	Letter	Rev:	0		

Appendix C

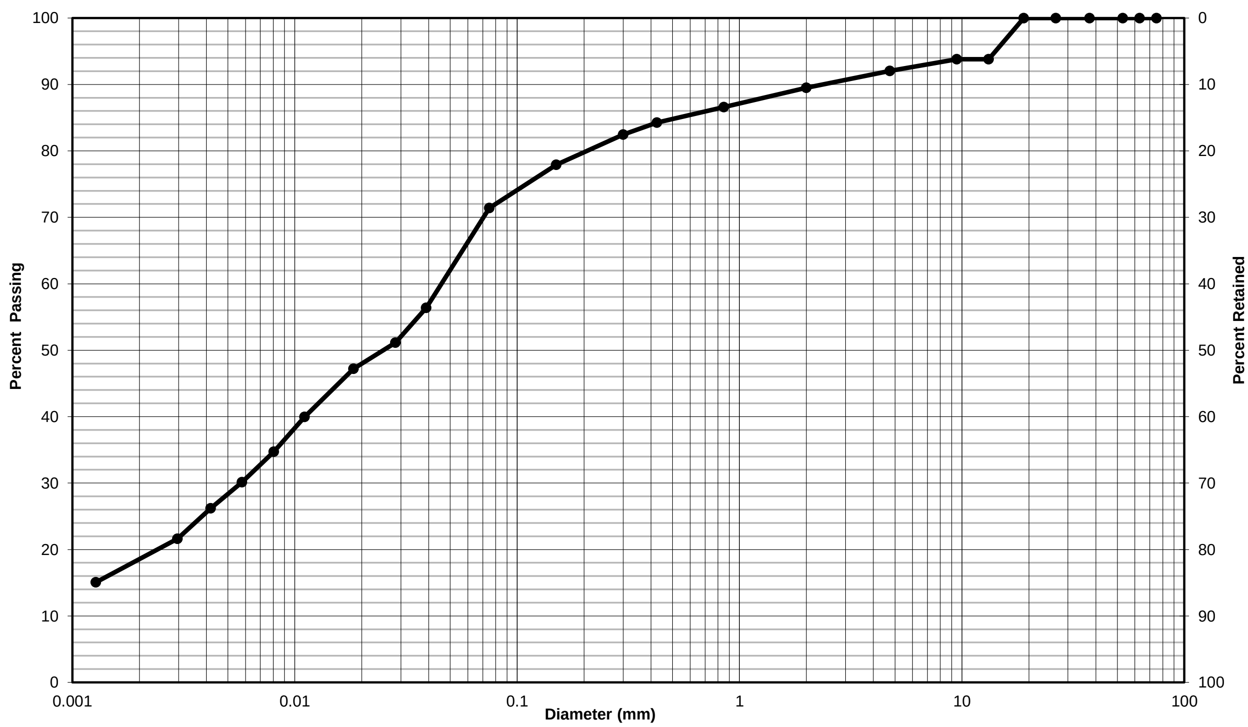
LABORATORY TESTING RESULTS

WSP

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL37-1
Project/Site:	Lower Nicholson	**Project no.:**	161-08318-00

Borehole no.:	16-1	Sample no.:	SS4
Depth:	2.25-2.85m	Location:	Lower Nicholson



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	8	21	71	53	18

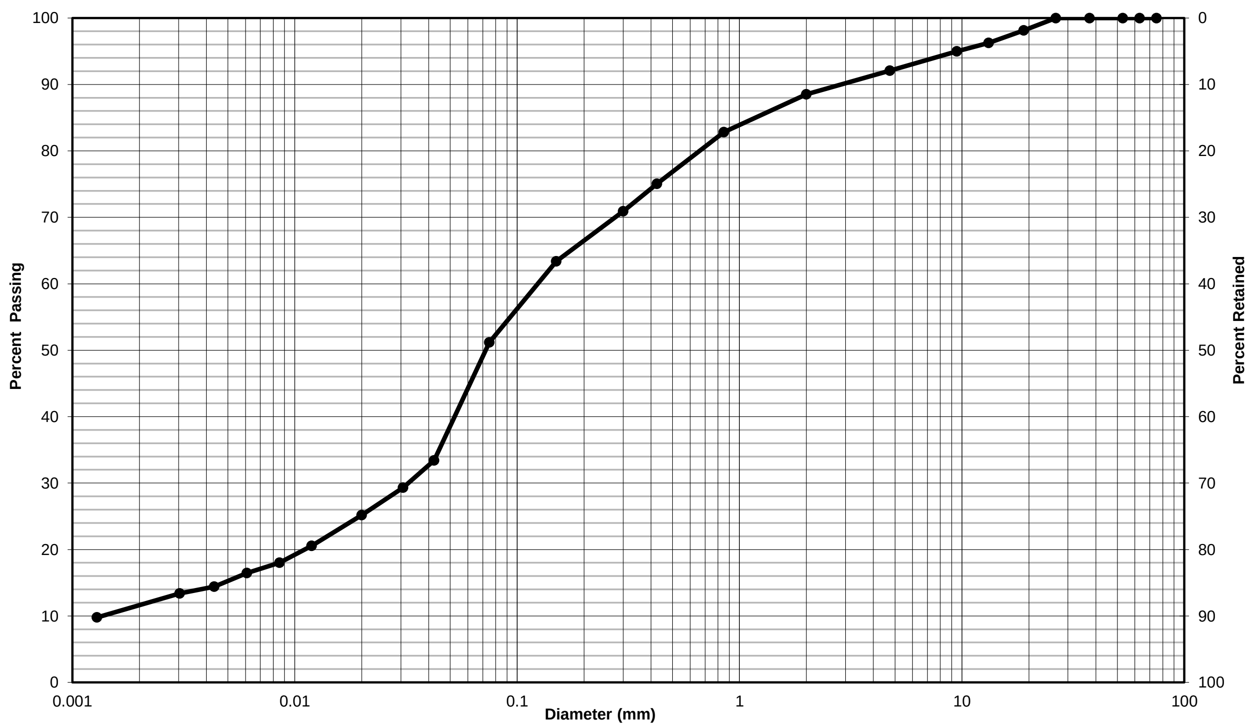
Remarks:

Performed by:	J.Meehan	**Date:**	July 8, 2016
Verified by:	N.Krebs	**Date:**	July 13, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL37-2
Project/Site:	Lower Nicholson	**Project no.:**	161-08318-00

Borehole no.:	16-2	Sample no.:	SS4
Depth:	2.25-2.85m	Location:	Lower Nicholson



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

	Gravel	Sand	Clay & Silt	Silt	Clay
Percent Retained	8	41	51	39	12

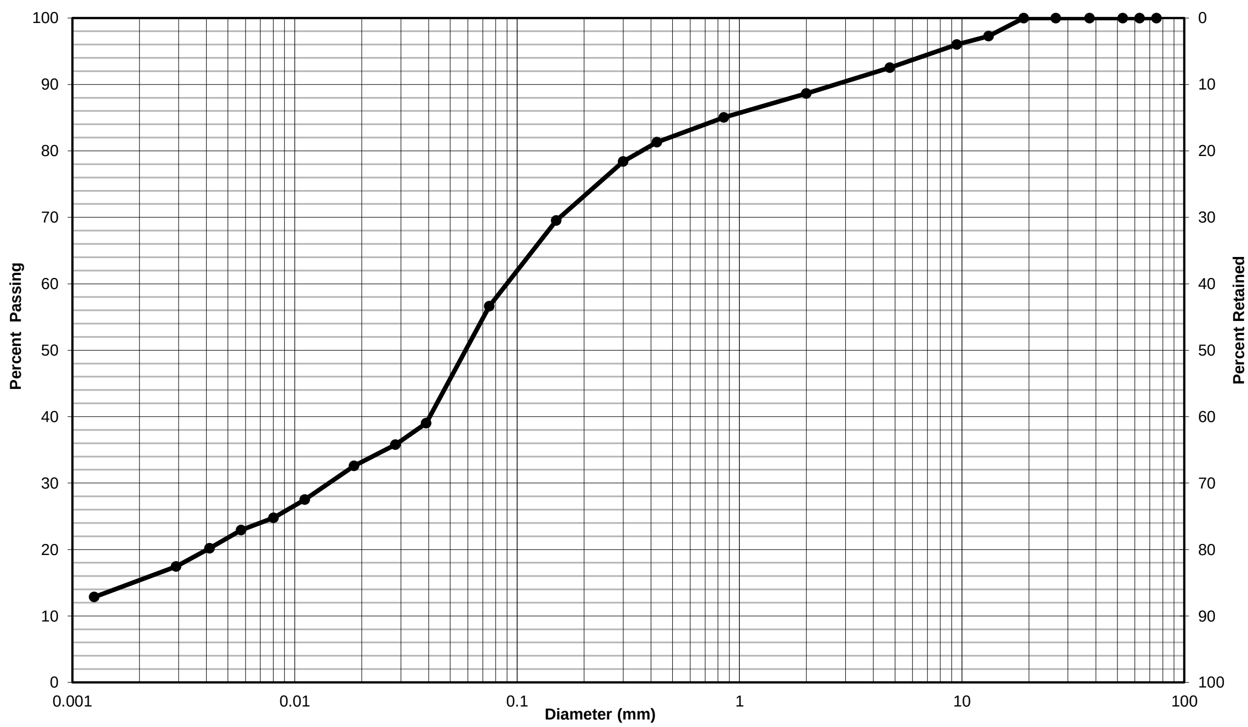
Remarks:

Performed by:	J.Meehan	**Date:**	July 8, 2016
Verified by:	N.Krebs	**Date:**	July 13, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL37-3
Project/Site:	Lower Nicholson	**Project no.:**	161-08318-00

Borehole no.:	16-3	Sample no.:	SS5
Depth:	3.0-3.7m	Location:	Lower Nicholson



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	7	36	57	41	16

Remarks:

Performed by:	J.Meehan	**Date:**	July 8, 2016
Verified by:	N.Krebs	**Date:**	July 13, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Parks Canada	**Lab no.:**	OL37-1
Project/Site:	Lower Nicholson	**Project no.:**	161-08318-00

Borehole no.:	16-1	Sample no.:	4	Depth:	2.25 - 2.85m
Soil description:	CL - Low plasticity inorganic clay			Date sampled:	June 21, 2016

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	6
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	40	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	35	27	19
Water Content:			
Tare no.	32	133	62
Wet soil+tare, g	31.32	32.22	28.17
Dry soil+tare, g	28.57	29.13	25.70
Mass of water, g	2.75	3.09	2.47
Tare, g	14.18	13.96	14.25
Mass of soil, g	14.39	15.17	11.45
Water content %	19.1%	20.4%	21.6%

Plastic Limit (PL) - Water Content:

Tare no.	9	55
Wet soil+tare, g	22.58	22.02
Dry soil+tare, g	21.83	21.22
Mass of water, g	0.75	0.80
Tare, g	14.83	14.09
Mass of soil, g	7.00	7.13
Water content %	10.7%	11.2%
Average water content %	11.0%	

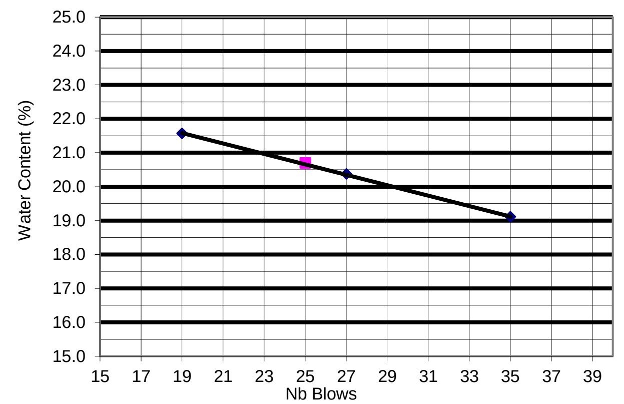
Natural Water Content (W^n):

Tare no.	B79	
Wet soil+tare, g	890.20	
Dry soil+tare, g	806.30	
Mass of water, g	83.90	
Tare, g	154.90	
Mass of soil, g	651.40	
Water content %	12.9%	

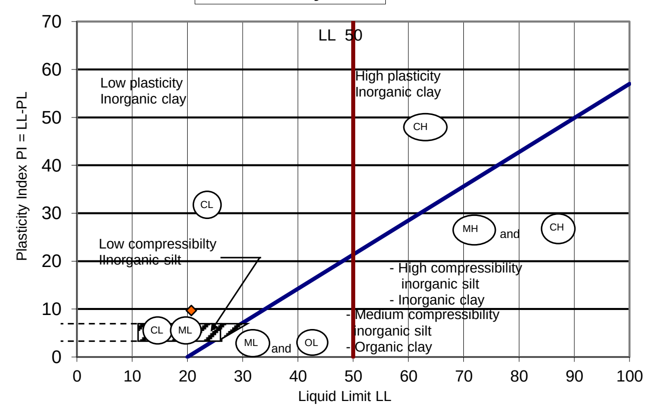
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation
- ☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
20.7	11	10	13

Remarks:

Performed by:	J.Meehan	**Date:**	July 12, 2016
Verified by:	N.Krebs	**Date:**	July 13, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Parks Canada	**Lab no.:**	OL37-4
Project/Site:	Lower Nicholson	**Project no.:**	161-08318-00

Borehole no.:	16-3	Sample no.:	6	Depth:	3.85 - 4.45m
Soil description:	CL / ML - Low compressibility inorganic silt			Date sampled:	June 21, 2016

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	5
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	40	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	34	20	15
Water Content:			
Tare no.	19	82	67
Wet soil+tare, g	31.16	31.36	28.17
Dry soil+tare, g	28.93	29.10	26.15
Mass of water, g	2.23	2.26	2.02
Tare, g	13.63	14.13	13.96
Mass of soil, g	15.30	14.97	12.19
Water content %	14.6%	15.1%	16.6%

Plastic Limit (PL) - Water Content:

Tare no.	6	41
Wet soil+tare, g	22.05	22.51
Dry soil+tare, g	21.32	21.73
Mass of water, g	0.73	0.78
Tare, g	14.58	13.92
Mass of soil, g	6.74	7.81
Water content %	10.8%	10.0%
Average water content %	10.4%	

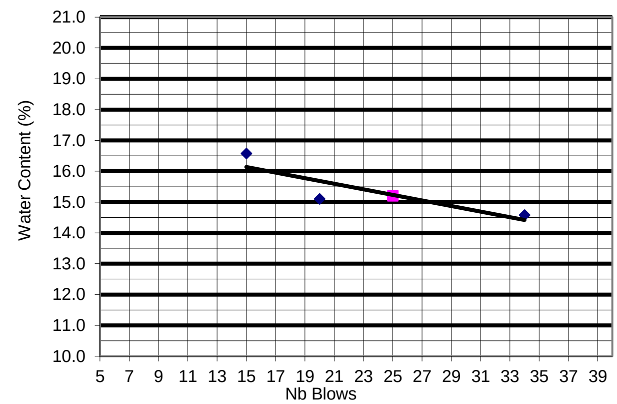
Natural Water Content (W^n):

Tare no.	B7	
Wet soil+tare, g	506.70	
Dry soil+tare, g	473.10	
Mass of water, g	33.60	
Tare, g	91.50	
Mass of soil, g	381.60	
Water content %	8.8%	

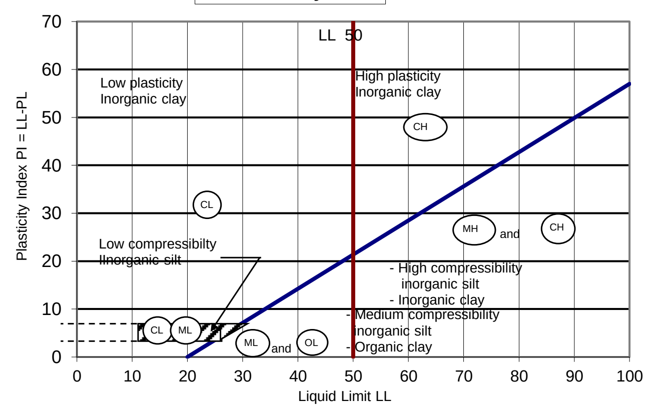
Soil Preparation:

- ☑ Cohesive <425 µm
- ☐ Cohesive >425 µm
- ☐ Non-cohesive
- ☑ Dry preparation
- ☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
15.2	10	5	9

Remarks:

Performed by:	J.Meehan	**Date:**	July 12, 2016
Verified by:	N.Krebs	**Date:**	July 13, 2016

Certificate of Analysis

Client:	WSP Canada Inc.(SPL) 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1	Report Number:	1610586
Attention:	Mr. Chris Hendry	Date Submitted:	2016-06-27
PO#:		Date Reported:	2016-06-30
Invoice to:	WSP Canada Inc.	Project:	161-08318-00, Lower Nicholson
		COC #:	183042

Dear Chris Hendry:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Shyla Monette
Team Leader, Inorganics

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Exova Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Exova (Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Exova (Mississauga) is accredited for specific parameters by SCC, Standards Council of Canada (to ISO 17025)

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Exova recommends consulting the official provincial or federal guideline as required.

EXOVA ENVIRONMENTAL ONTARIO

Certificate of Analysis

Client:	WSP Canada Inc.(SPL) 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1
Attention:	Mr. Chris Hendry
PO#:	
Invoice to:	WSP Canada Inc.

Report Number:	1610586
Date Submitted:	2016-06-27
Date Reported:	2016-06-30
Project:	161-08318-00, Lower Nicholson
COC #:	183042

Group	Analyte	MRL	Units	Guideline	1247335	1247336
				Lab I.D.	1247335	1247336
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2016-06-21	2016-06-21
				Sample I.D.	16-2	16-3
Agri. - Soil	pH	2.0			7.6	8.5
General Chemistry	Cl	0.002	%		0.004	<0.002
	Electrical Conductivity	0.05	mS/cm		0.18	0.13
	Resistivity	1	ohm-cm		5560	7690
	SO4	0.01	%		0.02	<0.01

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Appendix D

EXPLANATION OF TERMS USED IN REPORT

Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

WH – Samples sinks under "weight of hammer"

Dynamic Cone Penetration Resistance, N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	> 200 mm
Cobbles	75 mm - 200 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction
1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

Appendix E

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Incorporated (WSP) at the time of preparation. Unless otherwise agreed in writing by WSP, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.