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GEOTECHNICAL REPORT
RICHMOND LANDING SHORELINE ACCESSS FEASIBILITY STUDY
OTTAWA, ONTARIO

Prepared For:
NATIONAL CAPITAL COMMISSION

SPL Project No.: 10001599
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1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by the National Capital Commission (NCC) to conduct a geotechnical investigation in Support of the Richmond Landing Shoreline Access Feasibility Study, in Ottawa, Ontario.

The Terms of Reference for this investigation are outlined in SPL's Proposal No. P-15.02.126 dated March, 2015 and subsequent project correspondence.

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 engineering guidance related to the geotechnical aspects of the design.

2. PROJECT AND SITE DESCRIPTION

The overall project will include a variety of improvements to the site. The major components of the project are:

- Construction of two new pedestrian bridges (for the purposes of this report referred to as the North Bridge and the South Bridge). The North Bridge crosses from Victoria Island to Richmond Landing. The South Bridge crosses from Richmond Landing to the south shore of the Ottawa River.
- A new Ceremonial Landing and Dock which will be located at the northeast end of Richmond Landing. The Ceremonial dock will include an in-water structure founded on/anchored to the shore with a slab-on-grade section on shore;
- Four new monuments which will be constructed at various locations (two on Richmond Landing and two on the south shore of the Ottawa River). These monuments will be raised off the existing ground and rest on concrete pedestals;
- Various new landscaping works (pathways, minor re-grading, plantings, etc.) throughout the project area.

The areas around Richmond Landing, Victoria Island and the south shore of the Ottawa River have been extensively developed in the past. Victoria Island is generally flat to gently sloping with steep slopes (approximately 5 m to 6 m in height) dropping down to the river level. The ground surface at Richmond Landing slopes towards the northeast (from the west end of the project near the Portage Bridge towards the northeast tip of Richmond Landing near the location of the proposed new Belvedere). At the two bridge abutments on Richmond Landing the situation is similar to the Victoria Island abutment with relatively flat upland areas dropping steeply 5 m to 6 m into the Ottawa River.

It is anticipated that the majority of the soils at Victoria Island, Richmond Landing and the south Shore of the Ottawa River are primarily fill material, as the areas have been raised extensively over time. Erosion protection and rip rap is present in numerous locations in the project area which also suggests that erosion has been a problem in the past.

3. 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 a total of twenty boreholes in the project area;
- In-situ soil sampling and testing, including Standard Penetration Testing (SPT);
- Rock coring at selected borehole locations;
- Excavation of two test pits in the area of the Ceremonial Landing and Dock;
- Obtaining soil and rock samples for additional review and laboratory testing;
- Laboratory testing;
- Completion of a geophysical survey at the locations of proposed bridge abutments;
- 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 new pedestrian bridges and structures.

4. INVESTIGATION PROCEDURES

The geotechnical investigation was carried out in July 2015 through December 2015 in several stages.

4.1 Desk Study

Bedrock geology maps indicate the bedrock in the general area is limestone, dolostone, shale, arkose and sandstone of the Ottawa Formation. Surficial geology maps indicate the area is underlain by Paleozoic bedrock (i.e. no or minimal natural soil cover over the rock surface).

4.2 Field Investigation

4.2.1 July 2015 Field Investigation

The initial field investigation was carried out in July 2015 and included the drilling of 13 boreholes (BH15-1 through BH15-13) at various locations as shown in Drawing No. 2 and described below.

Table 1 – Borehole Locations (July 2015)

Borehole	Project Element	Elevation (Top of Borehole)	Borehole Depth
BH15-1	North Bridge – north abutment	48.3 m	4.4 m
BH15-2	North Bridge – south abutment	46.0 m	7.9 m
BH15-3	South Bridge – north abutment	45.1 m	9.1 m
BH15-4	South Bridge – south abutment	45.7 m	9.7 m
BH15-5	Proposed monument site	52.2 m	7.4 m

Borehole	Project Element	Elevation (Top of Borehole)	Borehole Depth
BH15-6	Proposed monument site	50.7 m	8.2 m
BH15-7	Proposed monument site	46.4 m	7.0 m
BH15-8	Proposed monument site	47.1 m	7.8 m
BH15-9	Belvedere Location	42.4 m	5.6 m
BH15-10	Landscaped Area	44.4 m	1.3 m
BH15-11	Landscaped Area	45.1 m	1.5 m
BH15-12	Landscaped Area	55.7 m	1.8 m
BH15-13	Landscaped Area	54.7 m	1.8 m

The boreholes were advanced using track-mounted and portable drilling equipment supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. Soil and rock samples retrieved during drilling were logged and visually classified in the field by a member of SPL's geotechnical staff. In-situ tests including Standard Penetration Testing (SPT) were carried out at regular intervals.

Piezometers were installed in Boreholes BH15-1 through BH15-4 to allow for subsequent measurement of stabilized groundwater levels and long-term groundwater monitoring at the site.

Borehole logs are included in Appendix I of this report.

4.2.2 December 2015 Field Investigation

A supplemental field investigation was completed in December, 2015. The supplemental field investigation included six additional boreholes (BH15-14 through BH15-19) in the vicinity of the proposed bridge abutments, as well as one borehole (BH15-20) at the location of the new Ceremonial Landing and Dock.

Table 2 – Borehole Locations (December 2015)

Borehole	Project Element	Elevation (Top of Borehole)	Borehole Depth
BH15-14	North Bridge – north abutment	45.0 m	2.1 m
BH15-15	North Bridge – south abutment	44.7 m	5.0 m
BH15-16	North Bridge – south abutment	42.8 m	5.0 m
BH15-17	South Bridge – north abutment	44.3 m	7.8 m
BH15-18	South Bridge – south abutment	45.1 m	7.7 m
BH15-19	South Bridge – south abutment	43.1 m	7.9 m
BH15-20	Ceremonial Landing and Dock	43.4 m	7.7 m

The boreholes were advanced using portable drilling equipment supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. Soil and rock samples retrieved during drilling were logged and visually classified in the field by a member of SPL's geotechnical staff. In-situ tests including Standard Penetration Testing (SPT) were carried out at regular intervals in soil. At all borehole locations, rock was cored using "N" size core barrels.

In addition to the additional boreholes, two test pits (TP15-1 and TP15-2) were excavated near the site of the proposed Ceremonial Landing and Dock to visually observe the nature and composition of the fill material at this location. The test pits were excavated using a hydraulic excavator supplied and operated by Landraulics Inc. of Ottawa, ON. The two test pits were excavated to a depth of 1.6 m to 1.7 m below the existing ground surface.

Borehole logs and test pit records for the December 2015 investigation are also included in Appendix I of this report.

4.2.3 Geophysical Survey

Concurrent with the December 2015 field investigation, a geophysical survey was completed to provide additional information related to the uniformity of the bedrock surface in the general area of the bridge abutments. The results of the geophysical survey are included in Appendix IV of this report.

4.3 Laboratory Testing

Upon completion of drilling and in-situ testing, soil and rock samples were returned to SPL's laboratory for further examination, classification and testing. A laboratory testing program, which was carried out on selected representative soil samples included the determination of natural water content, grain size distribution, and chemical analyses of soil corrosivity. The unit weight and unconfined compressive strength of select rock core samples were also determined.

The results of natural water content tests are included on the relevant borehole logs in Appendix I. The results of determination of grain size distribution are summarized on the individual borehole logs and are included in Appendix II. 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 III.

5. SUBSURFACE CONDITIONS

The subsurface conditions encountered within the boreholes at the site are discussed in the following sections. Detailed descriptions of the soil and groundwater conditions encountered at each of the borehole locations are included in the individual borehole logs in Appendix I.

5.1 Soil Conditions

5.1.1 Topsoil and Asphalt

Topsoil or asphalt was encountered at all of the borehole locations (asphalt where the boreholes were drilled through existing pathways and asphalt-covered areas; topsoil where drilled on soft landscaped areas).

5.1.2 Fill

The sub-surface conditions encountered at the site generally consists of a variable thickness of fill material overlying rock.

The fill material is a heterogeneous mix of silt, sand and gravel with cobbles and boulders as well as fragments of metal, brick, concrete, slag, glass, etc. Layers of clay and organic soil were also encountered at some locations. Angular rip rap/rock fill was encountered at one location (BH15-9). A relatively thick layer of wood and timber was encountered at some locations (Boreholes BH15-2, BH15-3, BH15-5, BH15-7 and BH15-8) near the interface between the fill and underlying rock on Richmond Landing.

Native soils, consisting of sand and gravel, silty sand and silty clay may have been encountered in small quantities at selected locations (at BH15-4 from a depth of 4.6 m to 6.5 m and BH15-8 from 6.8 m to 7.8 m). The majority of the boreholes, however, encountered rock (or auger refusal) immediately below the fill. While it is possible that other localized zones of natural soils are present, it is anticipated that the majority of the soils present on site are fill.

The consistency of the fill material ranges from very loose to dense, based on SPT “N” values.

Grain size curves for selected samples of the fill are included in Appendix II and summarized in the table below. It should be noted that grain size distribution testing was carried out on a samples obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles. Because of this the grain size distribution obtained through drilling may be finer overall than some portions of the material in the field.

Table 2 – Results of Grain Size Analyses for Fill

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	%Fines
BH15-1	1	34	55	11
BH15-2	1	17	47	36
BH15-3	1	18	63	20
BH15-3	2	12	81	8
BH15-3	3	24	51	26
BH15-4	4	54	33	13
BH15-5	1	64	29	7
BH15-5	3	9	80	11
BH15-6	3	42	41	17
BH15-7	3	39	47	15
BH15-8	3	11	40	49
BH15-11	1	34	45	21

It should be noted that fill material is, by nature, a highly variable material and other soil types or obstructions may be encountered during construction which were not encountered during drilling.

5.1.3 Auger Refusal/Bedrock

Auger refusal was encountered in the majority of the boreholes (not including BH15-11 through BH15-13 which were terminated at shallow depth). Auger refusal was encountered at various depths ranging from 1.5 m to 7.8 m (of the deeper holes, only Borehole BH15-6 did not meet auger refusal).

At Boreholes BH15-1 though BH15-4, BH15-9 and BH15-14 through BH15-20 bedrock was cored using “N” size coring equipment. The bedrock is generally described as fresh to slightly weathered, limestone with closely spaced shale partings. Rock Quality Designation (RQD) values range from 7% to 100% in the cores retrieved indicating very poor to excellent quality. The RQD values typically increase with depth, which is typical of rock in the area.

Unconfined Compressive Strength (UCS) testing was completed on selected rock cores and yielded the following values:

Table 3 – Results of UCS Testing

Borehole No.	Depth (m)	Unit Weight (kN/m^3)	UCS (MPa)
15-1	2.9	26.3	75.8
15-1	4.3	26.2	55.6
15-2	7.2	26.6	79.6
15-2	7.5	26.4	79.7
15-3	6.3	26.2	60.1
15-3	8.9	26.3	76.9
15-4	7.5	26.4	57.5
15-9	4.0	26.4	63.7
15-9	5.0	26.2	71.8
15-15	5.8	26.6	95.1
15-16	2.2	26.5	95.4
15-18	6.7	26.4	102.2
15-19	5.2	25.9	97.0
15-20	2.3	26.6	100.3
15-20	7.0	26.7	73.5
Average		26.4	78.9

5.2 Groundwater Conditions

Piezometers were installed in Boreholes BH15-1 through BH15-4 during the field investigation. The groundwater levels within the piezometers were measured and found to be between 2.8 m and 5.5 m below the existing ground surface as presented below at the time of the original investigation in the summer of 2015. A subsequent measurement was taken at accessible piezometers in February, 2016 at which time the groundwater level on Richmond Landing had risen to 42.8 m to 42.9 m.

Table 4 – Simplified Stratigraphy and Groundwater Elevations

Borehole No.	Elevation (Top of Borehole)	Measured Groundwater Depth and Elevation	
		(August 10, 2015)	(February 8, 2016)
BH15-1	48.3 m	2.3 m/46.0 m	N/A
BH15-2	46.0 m	4.6 m/41.4 m	3.2 m/42.8 m
BH15-3	45.1 m	3.9 m/41.2 m	2.2 m/42.9 m
BH15-4	45.7 m	4.3 m/41.4 m	N/A

At Borehole BH15-1 the measured groundwater level was significantly higher than the adjacent river (by several metres). This may indicate the presence of a shallow perched water table.

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events. At this site, it should also be anticipated that groundwater levels would fluctuate with the level of the river.

5.3 Summary

A summary of the soil and groundwater conditions encountered within the boreholes near the proposed bridge abutment locations is presented below.

Table 5 – Simplified Stratigraphy and Groundwater Elevations

Borehole No. (Elevation)	Simplified Stratigraphy (Depth in metres)			Measured Ground Water Depth (m)	Notes
	Topsoil or Asphalt	Overburden (Fill)	Bedrock (Cored)		
BH15-1	0 – 0.13	0.13 – 1.5	1.5 – 4.4	2.8	--
BH15-2	0 – 0.09	0.09 – 4.8	4.8 – 7.9	5.8	--
BH15-3	0 – 0.09	0.09 – 5.8	5.8 – 9.1	5.5	--
BH15-4	0 – 0.27	0.27 – 6.5	6.5 – 9.7	3.5	--
BH15-5	0 – 0.05	0.05 – 7.4	--	--	Auger refusal at 7.4 m
BH15-6	0 – 0.11	0.11 – 8.2	--	--	Borehole terminated at 8.2 m
BH15-7	0 – 0.10	0.10 – 7.0	--	--	Auger refusal at 7.0 m
BH15-8	0 – 0.14	0.14 – 7.8	--	--	Auger refusal at 7.8 m
BH15-9	--	0 – 3.8	3.8 – 5.6	--	--
BH15-10	0 – 0.05	0.05 0 1.3	--	--	Auger refusal at 1.3 m
BH15-11	0 – 0.13	0.13 – 1.5	--	--	Borehole terminated at 1.5 m

BH15-12	0 – 0.10	0.10 – 1.8	--	--	Borehole terminated at 1.8 m
BH15-13	0 – 0.08	0.08 – 1.8	--	--	Borehole terminated at 1.8 m
BH15-14	0 – 0.15	0.15 – 1.6	1.6 – 2.1[1]	--	Borehole terminated at 2.1
BH15-15	--	0 – 3.4	3.4 – 5.0	--	--
BH15-16	--	0 – 0.7	0.7 – 5.0	--	--
BH15-17	--	0 – 5.6	5.6 – 7.8	--	--
BH15-18	--	0 – 6.3	6.3 – 7.7	--	--
BH15-19	--	0 – 4.9	4.9 – 7.9	--	--
BH15-20	--	0 – 5.1	5.1 – 7.7	--	--
TP15-1	0 – 0.24	0.24 – 1.7	--	1.7 m	Test pit terminated due to excessive seepage
TP15-2	0 – 0.20	0.20 – 1.6	--	1.6 m	Test pit terminated due to excessive seepage

6. DISCUSSION AND RECOMMENDATIONS

6.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. At this time general arrangements of the proposed bridges and preliminary details regarding the structures and monuments are available. Once the detailed design has started and additional information is available, SPL should review the recommendations provided within this report. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

Reference should be made to the Limitations of this Report which follows the text but forms an integral part of the report.

6.2 Frost Protection

The depth of frost penetration for the site is 1.8 m. All foundation elements should therefore have a permanent soil cover of at least 1.8 m (or its thermal equivalent if artificial insulation is used).

[1] Borehole only cored to 0.5 m depth due to inability to continue accessing the site at the time of the investigation.

6.3 Seismic Site Classification

The site is located in an area of moderate seismic activity. For the purposes of seismic design, the Site Classification for Seismic Site Response may be assumed to be Site Class B (Rock) at the bridge abutment locations where the elevation of the abutment is 3 m or less from the underlying rock. Where the depth of soil below the final abutment level is greater than three metres the Site Classification may be assumed to be Site Class C.

The Peak Horizontal Ground Acceleration (PHA) for an earthquake with 2% chance of exceedance in 50 years (2,475 years return period event) is 0.323 g according to the Earthquakes Canada seismic hazard values calculator. The corresponding spectral accelerations are $S_a(0.2) = 0.635$ and $S_a(1.0) = 0.137$. These values correspond to a Seismic Performance Category of 2 or 3 depending upon the fundamental period of the bridge structures and Importance Category (See Table 4.10 of the CHBDC).

6.4 Seismic Liquefaction

Based on the results of the field investigation it is considered that there is no significant risk of liquefaction under the design earthquake.

6.5 Bridge Foundations

At the north abutment of the North Bridge (BH15-1 and BH15-14) the overburden depth was found to be relatively shallow (1.5 m and 1.6 m respectively, dipping slightly towards the river). Excavations to the proposed footing depth would be above the indicated 2 year flood level, and at this location excavations to construct a shallow foundation on rock may be feasible. At the remaining abutments (BH15-2 through BH15-4 and BH15-15 through BH15-19) the rock surface was encountered at depths of 4.8 m to 6.5 m below the existing grade at these locations it is unlikely that excavations for shallow foundations will be a preferred option, given the requirements for dewatering.

Environmental investigations are beyond SPL's current scope of work. It is, however, known based on previous reports provided by the NCC that impacts exist within the fill material which is present over most of the site. If large excavations are carried out on Victoria Island it is important to consider the requirements for disposal of excess soils and protection of the adjacent River. For this reason, deep excavations, or excavations immediately adjacent to the River may not be a preferred option, even though technically feasible.

6.5.1 Shallow Foundations

If construction of a shallow foundation on rock at BH15-1 is a preferred option, the unfactored bearing resistance of foundations placed on the existing rock may be taken as 3.0 MPa. A geotechnical resistance factor of 0.5 should be applied to this, for a factored resistance at ULS of 1.5 MPa. If higher bearing resistances are required then the rock may be excavated to sound, undisturbed rock (1 m to 1.5 m below the rock surface) in which case the geotechnical bearing resistance may be increased to 6 MPa and 3 MPa at ULS (unfactored and factored, respectively).

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.

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.

6.5.2 Deep Foundations

Deep foundations are likely the most feasible foundation option for three of the four bridge abutments (accounting for environmental considerations and depending on geometry perhaps all four).

Based on the preliminary designs for the two bridges it is possible that driven piles will be very short (in several cases shorter than the 3 m minimum that is typically required) because of the relatively shallow bedrock.

Where foundation depths are too short for driven steel piles drilled and cast-in-place piles (caissons) with rock sockets can be considered. Where sufficient depth exists for driven steel piles then either deep foundation type may be used.

6.5.2.1 Rock-Socketed Caissons

6.5.2.1.1 Compressive Resistance

The compressive resistance of drilled and cast-in-place piles (caissons) 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 upper portion of the rock was typically found to be highly fractured, with relatively low RQD values, which is normal for bedrock in the area. RQD values and rock quality generally increase with depth. Typical design practice is to simply ignore the resistance in the upper poor quality rock when assessing the overall socket capacity.

For design purposes the following table provides typical depths to sound rock (and therefore the theoretical top of the rock socket for design purposes).

Table 6 – Depth to Sound Rock and Depth to Top of Rock Socket

Location	Borehole No.	Approximate BH Elevation	Rock Surface Depth/Elevation (m)	Depth of Highly Fractured or Poor Quality Rock (m)	Top of Theoretical Socket Depth/Elevation (m)
North Bridge, North Abutment	BH15-1	48.3 m	1.5/46.8	1.4	2.9/45.4
	BH15-14	45.0 m	1.6/43.4	*	*/42.0
North Bridge, South Abutment	BH15-2	46.0 m	4.8/41.2	0.8	5.6/40.4
South Bridge, North Abutment	BH15-3	45.1 m	5.8/39.3	1.4	7.2/37.9
South Bridge, South Abutment	BH15-4	45.7 m	6.5/39.2	0.9	7.4/38.3

* *Rock could not be completely cored at BH15-14 due to site restrictions. For preliminary design it is assumed the depth to sound rock is similar to nearby BH15-1*

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. SPL can provide additional assistance during detailed design if closely spaced caissons are required.

6.5.2.1.2 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/m^3 for soil and 26 kN/m^3 for rock).

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

6.5.2.1.3 Lateral Resistance

If caissons are used lateral loads would typically be resisted by the lateral resistance of the vertical caissons (and adjacent soil).

The lateral resistance of long caissons is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine lateral deflection of caissons in soil is the coefficient of horizontal subgrade reaction (k_s). For this site k_s may be assumed to be:

k_s = 6.6 (z/d) above the water table

and

k_s = 4.4 (z/d) below the water table

Where: k_s = the modulus of subgrade reaction (MPa/m);
z = depth below final grade;
d = caisson or pile diameter

Due to the permeable nature of the soils at the site the groundwater level for design may be assumed to be at the higher of the encountered groundwater levels or the design flood elevation of the river (i.e. it should be assumed that should the river rise, groundwater levels will also rise).

The use of a modulus of subgrade reaction provides a linear "spring" coefficient for modelling soil-caisson response. The total resistance applied by the spring should be limited to the passive pressure of the soil (see Section 6.8); i.e. the soil resistance will increase linearly with deflection up to the passive pressure, beyond which the lateral resistance of the soil will remain constant even with increasing deflection.

This parameter is associated with acceptable deflections, and is an unfactored SLS value. The corresponding geotechnical resistance factor (ϕ_{gs}) may be taken as 0.8 and should be applied the above values for design.

The value above is for a single pile. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal subgrade reaction (k_s) by an appropriate factor as follows:

Table 7 – Coefficient of Horizontal Subgrade Reaction Reduction Factors

Pile Spacing in Direction of Loading (d = pile diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other spacings may be interpolated from the above. No reduction is required for the first row of piles (i.e. the row which bears against undisturbed soil with no piles in front).

The lateral resistance of the rock-socketed portions of the piles (i.e. below the overburden) is likely to be significantly higher than the resistance of the existing soil overburden or new backfill above. For the purposes of determining the resistance of the portions of the piles in rock, the weathered, fractured portion of the rock may be conservatively modelled as soil down to sound rock (see Table 6 above). The caissons may be assumed to be essentially fixed at the top of the theoretical rock socket provided in

Table 6, as the deflection of sound rock under lateral load will be very small compared with the deflection of the portion in soil.

Based on experience with similar projects in the past, it is likely that should lateral resistances be mobilized to depths of 6 m to 7 m below grade, the lateral deflections will have often already become unacceptable regardless of the higher lateral resistances at depth.

If the lateral deflection of the foundations is a critical issue then it is recommended that a more rigorous non-linear method (such as the method of p-y curves) be used to model the soil-pile system. These more rigorous methods require detailed understanding of the proposed soil-pile system (pile type, size, spacing, depths, etc.) and are best undertaken in detailed design.

In the event the lateral resistance of the soil/rock is insufficient to limit deflections to an appropriate level, then it is understood the foundations may be anchored to resist lateral forces. Discussion of rock anchors is provided in Section 6.10 below.

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 (provided in Table 6) to an unfactored value of 3.0 MPa and a factored value of 1.5 MPa.

6.5.2.1.4 Construction Considerations

The caissons will be drilled through overburden soils which are known to contain cobbles, boulders, concrete, metal, timber, and other obstructions. In addition 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.

Groundwater should be expected during drilling of the rock sockets. It is anticipated that groundwater inflow can be handled by pumping from the caisson provided the flow through the overburden 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.

6.5.2.2 Driven Steel Piles

If the depth to rock is sufficient (greater than 3 m to 4 m) then driven steel piles may be considered as a foundation option.

6.5.2.2.1 Compressive Resistance

Steel piles would be driven to bedrock which was encountered at depths of up to 7.4 m below the existing ground surface at the proposed bridge abutment locations.

Piles driven to sound rock typically generate high ultimate geotechnical capacities, generally equal to or in excess of the structural capacity of the steel section. For the purposes of design, the ultimate geotechnical resistance may be assumed to be equal to the ultimate structural resistance of the steel section. A resistance factor of 0.4 should be applied to this value to obtain the factored geotechnical resistance of a pile driven to sound rock.

As an example, an HP310x79 has an ultimate structural resistance of 3,490 kN (based on the cross-sectional area and assuming 350 MPa yield strength, and ignoring buckling, bending, lateral loads, etc. or any other more complex situations which may reduce the structural capacity). The factored geotechnical resistance of an HP310x79 driven to sound rock can therefore be assumed to be 1,395 kN (0.4 x 3,490).

Settlements for piles driven to sound rock are generally negligible, and the geotechnical resistance mobilized at 25 mm of settlement (SLS) would normally exceed the factored axial resistance at ULS. Geotechnical SLS considerations therefore do not generally govern the design of piles driven to sound rock.

6.5.2.2.2 Uplift Resistance

The uplift resistance of a pile will be as a result of skin friction acting along the surface area of the embedded pile.

The unfactored shaft resistance (q_s) is equal to:

$$q_s = \beta\sigma_v' = \beta(\gamma' h)$$

where:

q_s = the unfactored shaft resistance (in kPa)
β = the shaft resistance factor based on pile and soil type (use 0.4)
σ_v' = the effective stress at a given depth equal to $\gamma'h$
γ' = the effective soil unit weight at a given depth (use 19 kN/m^3 above the water table and 9 kN/m^3 below the water table; for design purposes the water table should be taken as the higher of the encountered groundwater level or the design flood level in the river);
h = the depth below the (final) ground surface

A resistance factor of 0.3 should be applied to this value, to obtain the factored geotechnical uplift resistance. The dead weight of the pile 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 pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single "block" of soil with a perimeter equal to the perimeter of the pile group (the mass of the soil inside the "block" may be included in the calculation; use 19 kN/m^3).

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

It should be noted that the uplift resistance is highly dependent upon the installation of the piles as well as the layout of the pile groups. If the piles are used to resist significant uplift loads (and uplift governs the overall design) consideration may be given to carrying out a tension test to confirm the uplift capacity.

6.5.2.2.3 Lateral Resistance

As with caissons, the lateral resistance of long piles is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine lateral deflection of caissons in soil is the coefficient of horizontal subgrade reaction (k_s) which may be assumed as recommended in Section 6.5.1.2.3 for Caissons above.

The ultimate lateral resistance of a slender pile typically does not govern the overall design because of the high displacements required to reach ULS. However, should a geotechnical ULS value be required one can be calculated using the Broms method referenced in the Canadian Foundation Engineering Manual or the method of P-Y curves as discussed above for caissons. These methods, however, both require knowledge of geometry of the foundation and therefore cannot be completed until the detailed design phase.

6.5.2.2.4 Negative Skin Friction

The raising of the grade and/or permanent lowering of the groundwater table will cause settlement of the existing soils which will in turn cause negative friction or down drag on the piles. Under either of these conditions the potential exists to develop negative skin friction along the piles and this should be considered in the final design.

The magnitude of negative skin friction depends on the pile loading, dimensions and the final configuration of the site, as well as the details of the permanent below-grade portions of the building (in particular drainage) and will need to be confirmed during detailed design based on these factors. For preliminary design, however, the negative skin friction can be assumed to be equal to the shaft friction as calculated for uplift resistance above (the resistance factor of 0.3 should not be applied).

Negative friction is typically only considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the pile. Negative friction does not impact the geotechnical resistance of the piles.

6.5.2.2.5 Construction Considerations

The piles will be driven to bedrock (which is expected to be up to 7.5 m below the existing ground surface through ground which is expected to contain cobbles, boulders and other similar obstructions. Some allowance should be made for wasting of piles which become damaged or for reduced design capacities for piles which cannot be successfully driven to rock.

Appropriate piling equipment and hammers capable of generating sufficient driving energy will be required to drive the piles to rock and mobilize the full geotechnical resistance of the pile. Allowance should also be made for re-striking a portion of the piles a minimum of 2 days after initial driving to confirm that relaxation has not occurred. The rock quality is generally good and significant penetration into the bedrock is not expected.

The piling specifications should be reviewed by SPL prior to tender, as should the contractor's submission (i.e. shop drawings, equipment, procedures and preliminary set criteria) prior to construction. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic testing (PDA Testing) carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses.

All piling operations should be supervised on a full-time basis by SPL to monitor pile locations, plumbness, pile set, re-striking, etc. and to confirm that the design and construction of the piles is as anticipated in preparing the recommendations included in this report.

6.6 Ceremonial Landing and Dock

It is understood the proposed ceremonial dock and landing structure will be a relatively lightly loaded structure similar to a concrete slab-on-grade, to which a section of floating dock will be connected/anchored.

Borehole BH15-20 as well as Test Pits TP15-1 and TP15-2 were advanced at this location and encountered fill material which included silty sand, cobbles, boulders, brick, asphalt, concrete, cut stone blocks, etc. This fill extended to a depth of 5.1 m. Test pitting within the fill material at this location revealed the material is primarily soil mixed with a significant amount of large debris down to 1.6 m to 1.7 m. Below this depth (which coincided with the level of the river at the time of the test pitting, further excavation was not possible due to excessive seepage).

For preliminary planning and design the following is recommended for construction of the sub-grade at the dock location:

- Strip the existing topsoil and fill material to a minimum of 450 mm below the proposed base of the slab on grade;
- Place 150 mm of Granular B Type II over the exposed fill material as a levelling course and compact to 95% SPMDD, filling any voids exposed on the surface of the coarse fill material;
- Place a layer of geotextile and geogrid (such as a Combigrid or similar product);
- Place 300 mm of Granular A and compact to 98% SPMDD;
- Construct the concrete slab on grade

The factored geotechnical bearing resistance at ULS for an appropriately prepared foundation may be assumed to be 150 kPa.

The geotechnical resistance of the slab-on-grade at SLS will depend on the settlement characteristics of the soil below the slab, as well as the magnitude and geometry of loading. The geotechnical parameter typically used for analysis of settlement below a raft or slab is the vertical modulus of subgrade reaction. Based on the field investigation, a modulus of subgrade reaction (k_v) of 40 MPa/m may be used for a subgrade prepared as recommended above.

The modulus of subgrade reaction is not a fundamental soil property, but is dependent upon the size and shape of the loaded area, soil type, relative stiffness of the raft and soil, duration of loading, etc. As a result, the modulus for a 300 mm square footing is typically used as a standard basis. For loaded areas greater than 300 mm square the above value should be multiplied as follows:

$$kvb = kv1\left[\frac{b+0.3}{2b}\right]^2$$

where

k_{vb} = the modulus for actual loaded area of b
b = width of the loaded area; for b > 4 m the corresponding value for b = 4 m may be used with no further reduction

It should be noted that fill material can be highly variable and adjustments may need to be made to the subgrade preparation in light of uncovered conditions as construction proceeds. The area should be reviewed during excavation and proof-rolling and prior to placement of any engineered fill, and if required the above recommendations adjusted as appropriate in the field.

In the event caisson foundations are required to support/anchor portions of the dock structure the recommendations in Section 6.5 above may be followed.

6.7 Monument Foundations

There are four proposed new monuments located throughout the project area (near Boreholes BH15-5 through BH15-8). The current plans incorporate a single monument at BH15-6 through BH15-8 and a slab-on-grade at BH15-5 to be used as a support for a future light monument(s).

6.7.1 Shallow Foundations (BH15-6 through BH15-8)

It is understood the structures at BH15-6 through BH15-8 will include statues or similar monuments on concrete pedestals. It is further understood that these monuments would be relatively tolerant of settlement.

The existing sub-grade in an area 1 m larger than the foundation (on all sides) should be proof rolled and compacted to 95 % SPMDD with a smooth drum vibratory roller. Any loose or soft areas exposed in the sub-grade should be removed and replaced with compacted granular fill (Granular A or Granular B Type II) compacted to 95% SPMDD. All foundation subgrades should be reviewed and approved by a geotechnical engineer

For a properly prepared subgrade, the factored geotechnical bearing resistance may be assumed to be 150 kPa at ULS. The geotechnical bearing resistance at SLS may be assumed to be 100 kPa. It should be noted that the soils at the area are primarily fill material which is highly variable in nature. It is possible that settlement of the foundations will occur. Settlements for foundations with a width of up to 3 m may be assumed to be on the order of 25 mm. For foundations larger than 3 m the proposed loading and size should be reviewed by SPL during detailed design.

6.7.2 Slab-on-Grade Foundation (BH15-5)

It is understood the proposed monument near BH15-5 has yet to be defined, but will include a large concrete slab-on-grade supporting the eventual monument(s).

New concrete slabs-on-grade (if required) should be supported on at least 300 mm of compacted, free-draining, well graded crushed sand and gravel (Granular “A”). All topsoil and deleterious material should be removed and the prepared surface recompacted and proof-rolled prior to placement of the sand and gravel. The crushed sand and gravel should be placed over the properly prepared subgrade or engineered fill and compacted to 98% of the materials Standard Proctor Maximum Dry Density (SPMDD) using a heavy vibratory roller.

For the purposes of the eventual design of the slab to support various imposed loads, a modulus of subgrade reaction (k_v) of 30 MPa/m may be used for a subgrade prepared as recommended above.

The modulus of subgrade reaction is not a fundamental soil property, but is dependent upon the size and shape of the loaded area, soil type, relative stiffness of the raft and soil, duration of loading, etc. As a result, the modulus for a 300 mm square footing is typically used as a standard basis. For loaded areas greater than 300 mm square the above value should be multiplied as follows:

$$kvb = kv1 \left[\frac{b + 0.3}{2b}\right]^2$$

where:

k_{vb} = the modulus for actual loaded area of b
b = width of the loaded area;

It should be noted that fill material can be highly variable and adjustments may need to be made to the subgrade preparation in light of uncovered conditions.

6.8 Earth Pressures

6.8.1 Static Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining 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 new foundation walls, retaining walls, etc. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. The current proposed alignments include approximately flat ground behind the foundation walls. If design changes require sloping ground then the earth pressure coefficients should be adjusted accordingly.

Table 8 – Earth Pressure Coefficients for Granular Fill (Existing and New)

Parameter	Value (Unfactored)	
Material	Granular A or Granular B	Existing Fill Material
Angle of Internal Friction (ϕ)	34 degrees	30 degrees
Unit Weight	22.0 kN/m^3 above the groundwater table	20.0 kN/m^3 above the groundwater table
Coefficient of Active Earth Pressure (k_a)	0.27	0.33
Coefficient of Earth Pressure at Rest (k_0)	0.43	0.5
Coefficient of Passive Earth Pressure (k_p)	3.8	3.0

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design.

The geotechnical resistance factor for passive pressure should be taken as 0.5. Even with this factor it should be noted that a displacement or deflection of the wall on the order of 1% to 2% of the retained height will still be required to mobilize the passive resistance. If these deflections cannot be tolerated then passive earth pressure (even factored) should not be relied upon to resist lateral loads. Active and at-rest earth pressures represent loads, and should be factored accordingly for structural design.

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.

Select free draining 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 walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

The granular fill should be placed in a zone behind the back of the wall defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or wall (see Figure C6.20 of the Commentary to the CHBDC).

6.8.2 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).

6.9 Embankment Design and Construction

The current preliminary design does not require any significant changes to the site grading. Should changes in grade be required, SPL should review the new designs and provide additional recommendations as appropriate.

It is understood the currently proposed design will not significantly alter the slopes along the Ottawa River, and because the bridge abutments will most likely be on deep foundations or rock, they will not be impacted by (or contribute to) any potential instability of the existing slopes. SPL should review the final design geometry to ensure this assumption remains valid and, if necessary complete a slope stability assessment.

6.10 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 3,000 kPa along the grout/rock interface. The upper portion of weathered or highly fractured rock should be ignored above the depths provided in Table 6 above.

- 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 rock socket at SLS can be assumed to be similarly small (typically less than 5 mm). SPL 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 by at least two full scale “performance” tests at an early stage of construction in accordance with Post-Tensioning Institute (PTI) and CFEM guidelines taken to 200% of working load. In the field, each installed anchor should be proof loaded to 1.33 times the design working load for the anchor, in accordance with PTI and CFEM guidelines. The soil/rock anchors should be double-corrosion protected (Class I).

At abutments where a full-scale load test is completed the geotechnical resistance factor may be increased to 0.55, and therefore consideration may be given to increasing the number of load test locations if increased anchor capacities are desirable.

6.11 Construction Considerations

6.11.1 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.

6.11.1.1 Excavations in Soil

The soils at the site are primarily fill material which includes silts, sands and gravels as wells as cobbles, boulders, concrete, timber and other obstructions. For preliminary planning purposes these soils above the water table (or depth of de-watering) can be classified as a Type 3 Soil. These soils should be classified as a Type 4 Soil below the water. 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 basal instability of braced excavations or slope instability of unsupported excavations.

Environmental investigations are beyond the scope of this current report. It is noted, however, that previous investigations have identified impacts in the fill material, which will need to be accounted for when planning excavations, disposal of excess soil, etc. Care should also be taken to protect the adjacent river from any potential impacts during excavation.

6.11.1.2 Excavations in Rock

Bedrock excavation may be required for the north bridge (particularly on the north abutment, but also potentially on the south) depending upon the depth of pile caps, abutment walls, etc.

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.

6.11.2 Groundwater Control

Groundwater levels at the site were found to be 2.3 m below grade (approximately 46.0 m elevation) at Victoria Island. The two piezometers installed on Richmond Landing (BH15-2 and BH15-3) encountered groundwater at 4.6 m and 3.9 m below the existing grade (approximately 40.4 m and 40.6 m elevation). The piezometer on the south shore of the Ottawa River (BH15-4) encountered groundwater at a depth of 3.5 m (42.5 m elevation).

In shallow excavations above the groundwater level seepage is likely to be manageable by pumping from properly filtered sumps. If deeper excavations are required below the water table then an active dewatering system, as well as cut-off walls will likely be required. In particular, should excavations be extended below the level of the river at the time of construction they would require a major dewatering effort.

It is anticipated that shallow excavations above the water table will likely not require a Permit To Take Water (PTTW) provided the initial dewatering and any dewatering after rainfall events is kept to below 50,000 liters per day. If substantially deeper excavations below the water table or the level of the river are required then this assumption is likely invalid and the requirements for a PTTW should be reviewed during design.

6.12 Corrosion and Cement Type

Samples of the existing fill and native silty clay 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 9 – Results of Soil Corrosivity Testing

Borehole/ Sample No.	Soil Type	Chloride (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)	Sulphate (%)
BH15-1 GS1	Fill	<0.002	0.18	8.6	5,560	0.03
BH15-2 GS1	Fill	0.003	0.27	8.5	3,700	0.02
BH15-3 SS2	Fill	<0.002	0.18	8.2	5,560	0.02
BH15-4 SS3	Fill	0.003	0.35	9.7	2,860	0.07

The soil resistivity values measured in the fill suggest a moderately corrosive 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 (both for caissons as well as other buried concrete components).

7. ADDITIONAL RECOMMENDATIONS

Environmental investigations are beyond the scope of SPL's current assignment. There is, however, enough existing information to suggest that there is a potential for any soils excavated from the site to have environmental impacts. Similarly, groundwater pumped from the site may also have impacts.

It is therefore recommended that the NCC begin developing a soil and groundwater management plan to decide how excess soil and groundwater will be managed and disposed of during construction.

8. CLOSURE

The Limitations of this Report, as presented in Appendix V, are an integral part of this report.

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

SPL CONSULTANTS LIMITED

Chris Hendry, M.Eng., P.Eng.
Senior Geotechnical Engineer

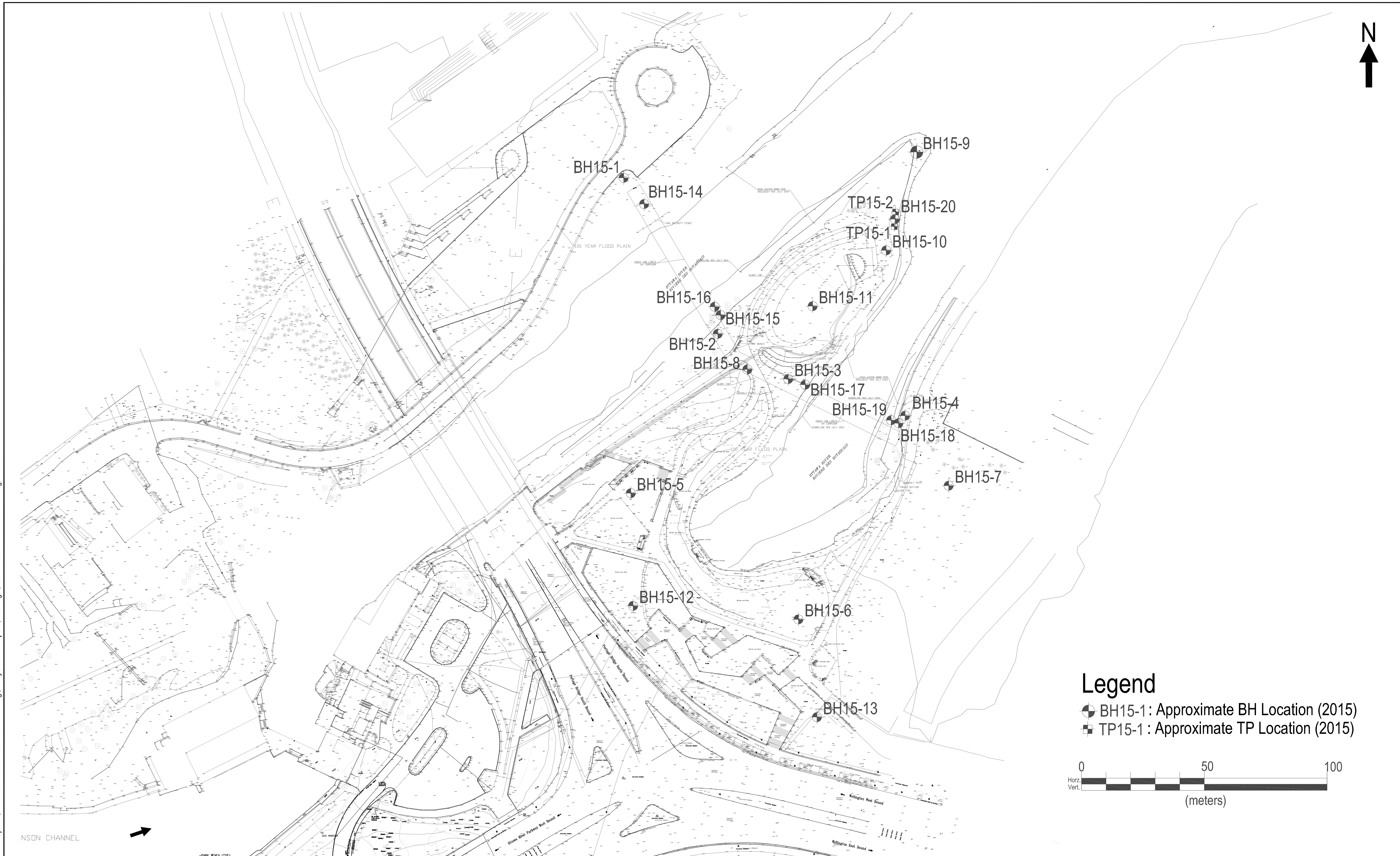
Fanyu Zhu, Ph.D., P.Eng.

Drawings

Client:	National Capital Commission, Capital Planning Branch		
Project#:	10001599	DWG #:	1
Drawn:	DW	Approved:	CH
Date:	August 2015	Scale:	N. T. S.
Size:	Letter	Rev:	0

Title:	Site Location Plan
Project:	Geotechnical Investigation Richmond Landing Shoreline Access Feasibility Study

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology



P:\Projects\SPL Projects and WSP 2015 Projects\10001599 NCC Richmond Landing\Analysis and Report\Drawings\ACAD-15023-RichLandMill.dwg

Client:	National Capital Commission, Capital Planning Branch			Title:	Borehole Location Plan
Project No.:	10001599	Drawing No.:	2	Project:	Geotechnical Investigation - Richmond Landing Shoreline Access Feasibility Study
Drawn:	DW	Approved:	CH		
Date:	January, 2016	Scale:	As Shown		SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology
Original	Tabloid	Rev:	1		

Appendix I

Borehole Logs (Record of Borehole Sheets)

LOG OF BOREHOLE BH15-01

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/27/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
48.3								
48.0 / 0.0	**TOPSOIL - 125 mm**							
48.1 / 0.1	**GRAVELLY SAND** trace to some silt, brown, moist (FILL)	1	GRAB		Cuttings	48, 47	○	34 55 (11)
46.8 / 1.5	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings,grey, with close to moderately closely spaced horizontal joints Run 1: 1.5m - 2.9m TCR: 86% SCR: 63% RQD: 7% Run 2: 2.9m - 4.4m TCR: 98% SCR: 86% RQD: 42%	2	CORE		Bentonite W. L. 46.0 m Aug 10, 2015	46		
		3	CORE		Sand Screen	45, 44		
43.9 / 4.4	**End of Borehole**							

Column headings also printed: SOIL PROFILE; STRATA PLOT; SAMPLES; 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/m^3).

Notes:
1) Auger refusal encountered at 4.4 m below the existing ground surface.
2) Borehole was dry upon completion of augering.
3) Borehole was dry upon completion of coring.
4) 19 mm dia. piezometer was installed in the borehole upon completion.
5) Date Depth-groundwater

Date	Depth-groundwater
08/10/2015	2.3 m

- the piezometer was noted to be clogged at 2.3 m below the surface elevation

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 BH15-02

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/27/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE

ELEV (m)	DEPTH (m)	DESCRIPTION
46.0		Ground surface
46.0	0.1	**Topsoil - 90 mm**
		SILTY SAND some crushed gravel, brown, moist (FILL)
44.5	1.5	SILTY SAND some gravel, some red brick fragments, brown, moist (FILL)
44.2	1.8	**SILTY SAND** brown, moist, loose to compact (FILL) - shale fragments - mixed with organics
41.4		
41.2	4.8	**WOOD** mixed with silty sand and gravel with a strong PHC odour.
		LIMESTONE fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 4.75m - 5.61m TCR: 71% SCR: 51% RQD: 0% Run 2: 5.61m - 7.11m TCR: 100% SCR: 86% RQD: 42% Run 3: 7.11m - 7.85m TCR: 98% SCR: 97% RQD: 59%
38.1	7.9	**End of Borehole**

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m
1	GRAB	
2A 2B	SS	25
3A 3B	SS	5
4	SS	17
5	SS	20
6	SS	50/ 50 mm
7	CORE	
8	CORE	
9	CORE	

GROUND WATER CONDITIONS

Sand
Bentontie
Cuttings
W. L. 41.4 m
Aug 10, 2015
Bentontie
Sand
Screen

ELEVATION: 45, 44, 43, 42, 41, 40, 39

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/m^3)

REMARKS AND GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
1	17	47	(36)	

Notes:
1) Auger refusal encountered at 4.75 m below the existing ground surface.
2) Borehole was dry upon completion of augering.
3) Water level upon completion of coring is 4.5 m below the existing surface elevation .
4) 19 mm dia. piezometer was installed in the borehole upon completion.
5)

Date	Depth-groundwater
8/10/2015	4.6 m

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 BH15-03

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand portable and coring
Diameter: 50 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m)	DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	NATURAL MOISTURE CONTENT	GR	SA	SI CL
45.1									
45.0	0.1	**Topsoil - 90 mm**							
		SILTY SAND some crushed gravel, brown, mosit (FILL)	1	SS	21	○	18	63	(20)
44.5	0.6	**SAND** trace silt to silty, trace to some gravel, trace red brick fragments, brown, moist, loose to compact (FILL)	2	SS	9		22	81	(8)
			3A 3B	SS	10	○	24	51	(26)
43.3	1.8	**SILTY SAND AND GRAVEL** with cobble and boulder sized rock fagments, brown, mosit (FILL)							
40.2	4.9	**Wood**							
39.3	5.8	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 5.79m - 7.16m TCR: 94% SCR: 84% RQD: 34% Run 2: 7.16m - 9.07m TCR: 100% SCR: 92% RQD: 82%	4	CORE					
			5	CORE					
36.0	9.1	**End of Borehole** Notes: 1) Sampler refusal encountered at 1.8 m below the existing ground surface, switch to NQ coring.							

Ground water conditions: Bentonite; W. L. 41.2 m Aug 10, 2015; Sand; Screen

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (45, 44, 43, 42, 41, 40, 39, 38, 37, 36); 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/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

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

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand portable and coring
Diameter: 50 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE: (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" BLOWS/0.3 m	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/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	2) Borehole was dry upon completion of sampling. 3) Bedrock encountered at 5.79 m below the existing ground surface. 4) 19 mm dia. piezometer was installed in the borehole upon completion. 5) Date Depth-groundwater -- 8/10/2015 3.9 m											

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 BH15-04

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE

ELEV (m) / DEPTH	DESCRIPTION
45.7 / 0.0	**Topsoil - 270 mm**
45.5 / 0.3	**SILTY SAND AND GRAVEL** brown, mosit, compact (FILL) - concrete fragments
41.2 / 4.6	**SAND** some silt, some gravel, brown, wet, loose to compact - black
39.2 / 6.5	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 5.79m - 7.39m TCR: 93% SCR: 81% RQD: 23% - fresh Run 2: 7.39m - 8.86m TCR: 100% SCR: 95% RQD: 55% Run 3: 8.86m - 9.68m TCR: 100% SCR: 100% RQD: 100%
36.1 / 9.7	**End of Borehole**

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	Natural moisture content	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	GRAB			
2	SS	26	○	54 33 (13)
3	SS	10		
4	SS	8	○	
5	SS	21		
6	SS	11	○	- wet spoon
7	SS	6		
8A / 8B	SS	50/ 50 mm	○	
9	CORE			
10	CORE			
11	CORE			

GROUND WATER CONDITIONS: Sand; Bentonite; W. L. 41.4 m Aug 10, 2015; Bentonite; Sand; Screen

ELEVATION: 45, 44, 43, 42, 41, 40, 39, 38, 37

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/m^3)

Continued Next Page

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

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

LOG OF BOREHOLE BH15-04

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) (25 50 75 100 125) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE	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/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Notes: 1) Auger refusal encountered at 6.5 m below the existing ground surface. 2) Water level upon completion of augering was 4.3 m below the existing ground surface . 3) Water level upon completion of coring is 4.3 m below the existing surface elevation . 4) 19 mm dia. piezometer was installed in the borehole upon completion. 5) Date Depth-groundwater -- 8/10/2015 4.3 m											

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

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

LOG OF BOREHOLE BH15-05

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/28/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION	WATER CONTENT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
52.2							
52.1 / 0.1	**Asphalt - 50 mm**						
	SILTY SAND AND GRAVEL brown, moist, loose to compact (FILL)	1	GRAB		52	○	64 29 (7)
		2	GRAB		51		
	- trace gravel at 1.5 m	3	SS	7		○	9 80 (11)
		4	SS	10	50		
		5	SS	9	49	○	
	- Limestone fragments at 4.1 m	6A 6B	SS	27	48	○	
47.6 / 4.6	**Burned wood and organic material with red brick fragments mixed with silty sand and gravel**	7	SS	54	47	○	
		8	SS	24			
		9	SS	21	46	○	
44.8 / 7.4	**End of Borehole**				45		

Notes:
1) Auger refusal encountered at 7.4 m below the existing ground surface.
2) Borehole was dry upon completion of augering and open to 4.7 m.

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; 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

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LOG OF BOREHOLE BH15-06

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/28/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
50.7						
50.6 / 0.1	**Topsoil - 110 mm**					
	SILTY SAND some crushed gravel, brown, moist, loose to compact (FILL)	1	GRAB			
		2	SS	20	○	
48.6 / 2.1	**SILTY SAND AND GRAVEL** with red brick fragments, brown, moist, compact (FILL)	3	SS	16	○	42 41 (17)
	- trace clay	4	SS	12		
	- shale fragments	5	SS	10	○	
		6	SS	10		
45.5 / 5.2	**SILTY SAND** trace gravel, brown, moist, loose to compact (FILL)	7	SS	9	○	
		8	SS	29		
		9	SS	11	○	
42.5 / 8.2	**End of Borehole**					

Notes:
1) Borehole was dry upon completion of augering.

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (50, 49, 48, 47, 46, 45, 44, 43); 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

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LOG OF BOREHOLE BH15-07

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/28/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GR	SA	SI	CL
46.4								
46.3 / 0.0	**Topsoil - 100 mm**							
0.1	**SILTY SAND AND GRAVEL** brown, mosit (FILL)	1	GRAB					
44.9 / 1.5	**SILTY SAND AND GRAVEL** dark brown, moist, compact (FILL)	2	SS	20				
	- asphalt fragments	3	SS	18	39	47	(14)	
	- red brick fragments - loose below 3.0 m	4	SS	7				
42.7 / 3.7	**SILTY SAND AND GRAVEL** black, moist, very loose (FILL) - contains slag particles	5	SS	1				
		6	SS	1				
41.1 / 5.3	**SILTY SAND** some gravel, dark brown, mosit, very loose (FILL)	7A 7B	SS	2				
40.3 / 6.1	**Wood**	8	SS	11				
39.6 / 6.8	**SILTY SAND AND GRAVEL** mixed with wood fragments, trace clay, grey, mosit (FILL)	9	SS	50/ 50mm				
39.4 / 7.0	**End of Borehole**							

Notes:
1) Borehole was dry upon completion of augering.

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (46, 45, 44, 43, 42, 41, 40); 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; STRATA PLOT

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

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

LOG OF BOREHOLE BH15-08

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/29/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
47.1 / 0.0	**Topsoil - 140 mm**
0.1	**SILTY SAND AND GRAVEL** brown, mosit (FILL)
45.3 / 1.8	**SILTY CLAYEY SAND** trace to some gravel, compact, moist (FILL) - contains woods, brick and mortar fragments
43.2 / 4.0	**SAND TO SILTY SAND** brown, compact to dense, moist (FILL) - contains organics, wood, brick fragments etc. - glass fragments
41.0 / 6.1	**Wood**
40.3 / 6.8	**SILTY CLAY** some gravel, trace sand, grey, wet, stiff (Glacial Till)
39.3 / 7.8	**End of Borehole**

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	GRAB		
2	SS	46	
3	SS	15	11 40 28 22
4	SS	9	
5	SS	12	
6	SS	21	
7	SS	76	
8	SS	9	
9	SS	7	
10	SS	50/ 125 mm	

Column headings: STRATA PLOT; GROUND WATER CONDITIONS; ELEVATION (47, 46, 45, 44, 43, 42, 41, 40); 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/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Notes:
1) Auger refusal encountered at 7.8 m below ground surface.
2) Water level upon completion of augering was 6.2 m below ground surface and borehole open to 7.7 m

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

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

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LOG OF BOREHOLE BH15-09

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand portable and coring
Diameter: 50 mm
Date: Jul/28/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION
42.4	
0.0	**COBBLES AND BOULDERS** (rock fill, rip rap, etc.) - angular limestone - sampler refusal from surface
38.6 3.8	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 3.8m - 5.0m TCR: 96% SCR: 85% RQD: 36% Run 2: 5.0m - 5.6m TCR: 100% SCR: 85% RQD: 25%
36.8 5.6	**End of Borehole**

Notes:
1) Bedrock was encountered at 3.8 m below the ground surface.
2) Water level on completion of coring was 1.2 m below ground surface.

SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m
1	CORE	
2	CORE	
3	CORE	
4	CORE	
5	CORE	

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES (NUMBER, TYPE, "N" BLOWS/0.3 m); GROUND WATER CONDITIONS; ELEVATION (42, 41, 40, 39, 38, 37); 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/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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 BH15-10

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) (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/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
44.4												
44.4 0.1	**Topsoil - 50 mm**											
	SILTY SAND some crushed gravel, brown, moist, loose to compact (FILL)		1	SS	7		44		○			
			2	SS	15							
43.1	- with brick and mortar fragments		3	SS	50/ 100 mm				○			
1.3	**End of Borehole** Notes: 1) Borehole was dry upon completion of sampling.											

SHEAR STRENGTH legend: ○ UNCONFINED; ● QUICK TRIAXIAL; + FIELD VANE & Sensitivity; × LAB VANE

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

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

LOG OF BOREHOLE BH15-11

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Jul/29/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
45.1								
45.0 / 0.0	**Topsoil - 130 mm**					45		
0.1	**SILTY SAND** gravelly, brown, mosit (FILL)	1	GRAB				○	34 45 (21)
						44		
43.6 / 1.5	**End of Borehole**							

Notes:
1) Borehole was dry upon completion of augering.
2) Steel cable wrapped around auger cutting bit on removal from the borehole.

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/m^3)

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 BH15-12

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	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/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
55.7												
55.6 / 0.1	**Topsoil - 100 mm**											
	SILTY SAND trace gravel, brown, moist, compact (FILL)		1	SS	12							
	- trace clay		2	SS	18		55		○			
			3	SS	23		54		○			
53.9 / 1.8	**End of Borehole** Notes: 1) Borehole was dry upon completion of sampling.											

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

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

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

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LOG OF BOREHOLE BH15-13

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Jul/30/2015

REF. NO.: 10001599
ENCL NO.:

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	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
54.7												
54.6 / 0.1	**Topsoil - 75 mm**											
	SILTY SAND some gravel, brown, moist, loose to compact (FILL)		1	SS	7							
							54					
	- trace clay		2	SS	16				○			
	- dense											
			3	SS	31				○			
52.9							53					
1.8	**End of Borehole**											
	Notes: 1) Borehole was dry upon completion of sampling.											

GROUNDWATER ELEVATIONS

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

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

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Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH15-14

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/11/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	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/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
45.0											
44.9 / 0.2	**TOPSOIL - 150 mm**										
	SILTY SAND some gravel, trace organics, trace glass, brown, moist, loose to compact (FILL)	1	SS	12							
		2	SS	6		44					
		3	SS	17							
43.4 / 1.6	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 1.6 m - 2.1 m TCR: 100% SCR: 62.5% RQD: 0%	1	CORE			43					
42.9 / 2.1	**End of Borehole**										

Notes:
1) SPT sampler refusal encountered at 1.6 m below the existing ground surface.
2) Borehole was dry upon completion of drilling.
3) NQ coring terminated at 2.1 m below the existing ground surface.

Plot scales: DCPT 20 40 60 80 100; Shear strength 25 50 75 100 125; ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; Water content 25 50 75

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 BH15-15

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/15/2015

REF. NO.: 10001599
ENCL NO.:

SOIL PROFILE: (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" BLOWS/0.3 m	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/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
44.7 / 0.0	**SILTY SAND** brown, moist (FILL)						44					
							43					
							42					
41.4 / 3.4	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 3.4 m - 5.0 m TCR: 100% SCR: 81% RQD: 44%						41					
							40					
39.7 / 5.0	**End of Borehole** Notes: 1) SPT sampler refusal encountered at 3.4 m below the existing ground surface. 2) Borehole was dry upon completion of drilling. 3) NQ coring terminated at 5.0 m below the existing ground surface.						39					
							38					
							37					

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

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

SPL Consultants Limited
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LOG OF BOREHOLE BH15-16

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/15/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
42.8 / 0.0	**GRAVEL** black, damp, compact (FILL)	1	SS	17			
42.1 / 0.7	**LIMESTONE** slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 0.7 m - 1.8 m TCR: 83% SCR: 38% RQD: 23%	1	CORE			42 41	
	Run 2: 1.8 m - 3.4 m TCR: 58% SCR: 51% RQD: 23%	2	CORE			40	
	Run 3: 3.4 m - 5.0 m TCR: 100% SCR: 100% RQD: 85%	3	CORE			39 38	
37.8 / 5.0	**End of Borehole** Notes: 1) SPT sampler refusal encountered at 0.7 m below the existing ground surface. 2) Borehole was dry upon completion of drilling. 3) NQ coring terminated at 5.0 m below the existing ground surface.						

Plot columns: 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/m^3); STRATA PLOT

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 BH15-17

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/14/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m)	DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m
44.3	0.0	**SILTY SAND** trace to some organics, dark brown, moist (FILL)	1	SS	4
43.6	0.8	**SILTY SAND** some gravel, brown, moist (FILL)	2	SS	50/0 mm
43.4	1.0	**CONCRETE**	1	CORE	
42.2	2.1	**SAND AND GRAVEL WITH COBBLES AND BOULDERS**	2	CORE	
40.4	4.0	**WOOD**	3	CORE	
38.7	5.6	**LIMESTONE** slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 5.6 m - 6.6 m TCR: 66% SCR: 34% RQD: 15% Run 2: 6.6 m - 7.8 m TCR: 68% SCR: 22% RQD: 0%	4 5	CORE CORE	
36.5	7.8	**End of Borehole**			

Notes:
1) SPT sampler refusal encountered at 0.95 m below the existing ground surface.
2) Borehole was dry upon completion of drilling.
3) NQ coring terminated at 7.8 m below the existing ground surface.

Other column headings: STRATA PLOT; GROUND WATER CONDITIONS; ELEVATION (37–44); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa) (25, 50, 75, 100, 125): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; 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/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

LOG OF BOREHOLE BH15-18

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/17/2015

REF. NO.: 10001599
ENCL NO.:

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)	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT (WATER CONTENT (%))	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
45.3 / 0.0	**SILTY SAND** brown, moist (FILL)						45, 44, 43, 42, 41, 40					
39.0 / 6.3	**LIMESTONE** slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 6.3 m - 7.7 m TCR: 100% SCR: 78% RQD: 50%		1	CORE			39, 38					
37.6 / 7.7	**End of Borehole** Notes: 1) Washbore to 6.3 m below the existing ground surface. 2) NQ coring terminated at 7.7 m below the existing ground surface.											

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
W_P w W_L — WATER CONTENT (%): 25 50 75

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

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

LOG OF BOREHOLE BH15-19

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/17/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS
43.1					
43.0 / 0.0	**TOPSOIL - 150 mm**				
0.2	**SILTY SAND** some gravel, brown, moist, compact (FILL)	1	SS	6	
		2	SS	12	
	- dense	3	SS	35	
41.6 / 1.5	**BOULDERS AND CONCRETE**	1	CORE		
38.3 / 4.9	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 4.85 m - 6.3 m TCR: 100% SCR: 99% RQD: 82%	2	CORE		
	Run 2: 6.3 m - 7.9 m TCR: 100% SCR: 87.5% RQD: 79%	3	CORE		
35.2 / 7.9	**End of Borehole** Notes: 1) SPT sampler refusal at 1.5 m below the existing ground surface. 2) Borehole is dry upon completion of drilling.				

Chart column headings: STRATA PLOT; ELEVATION (43, 42, 41, 40, 39, 38, 37, 36); 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/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

LOG OF BOREHOLE BH15-20

PROJECT: NCC Richmond Landing
CLIENT: National Capital Commission, Capital Planning Branch
PROJECT LOCATION: Richmond Landing, Ottawa, ON
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hand Portable
Diameter: 50 mm
Date: Dec/18/2015

REF. NO.: 10001599
ENCL NO.:

ELEV (m) / DEPTH	DESCRIPTION	STRATA PLOT	SAMPLE NUMBER	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)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
43.4 0.0	**SILTY SAND** brown, moist (FILL)						43 42 41 40 39					
38.3 5.1	**LIMESTONE** fresh to slightly weathered, strong to very strong, very closely bedded with close to very closely spaced shale partings, grey, with close to moderately closely spaced horizontal joints Run 1: 5.1 m - 6.1 m TCR: 100% SCR: 59% RQD: 53% Run 2: 6.1 m - 7.7 m TCR: 100% SCR: 89% RQD: 57%		1 2	CORE CORE			38 37 36					
35.7 7.7	**End of Borehole** Notes: 1) Washbored to 5.1 m below the existing ground surface 2) NQ coring terminated at 7.7 m below the existing ground surface.											

Plot scales: Dynamic cone 20 40 60 80 100; Shear strength 25 50 75 100 125; ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; Water content 25 50 75.

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

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

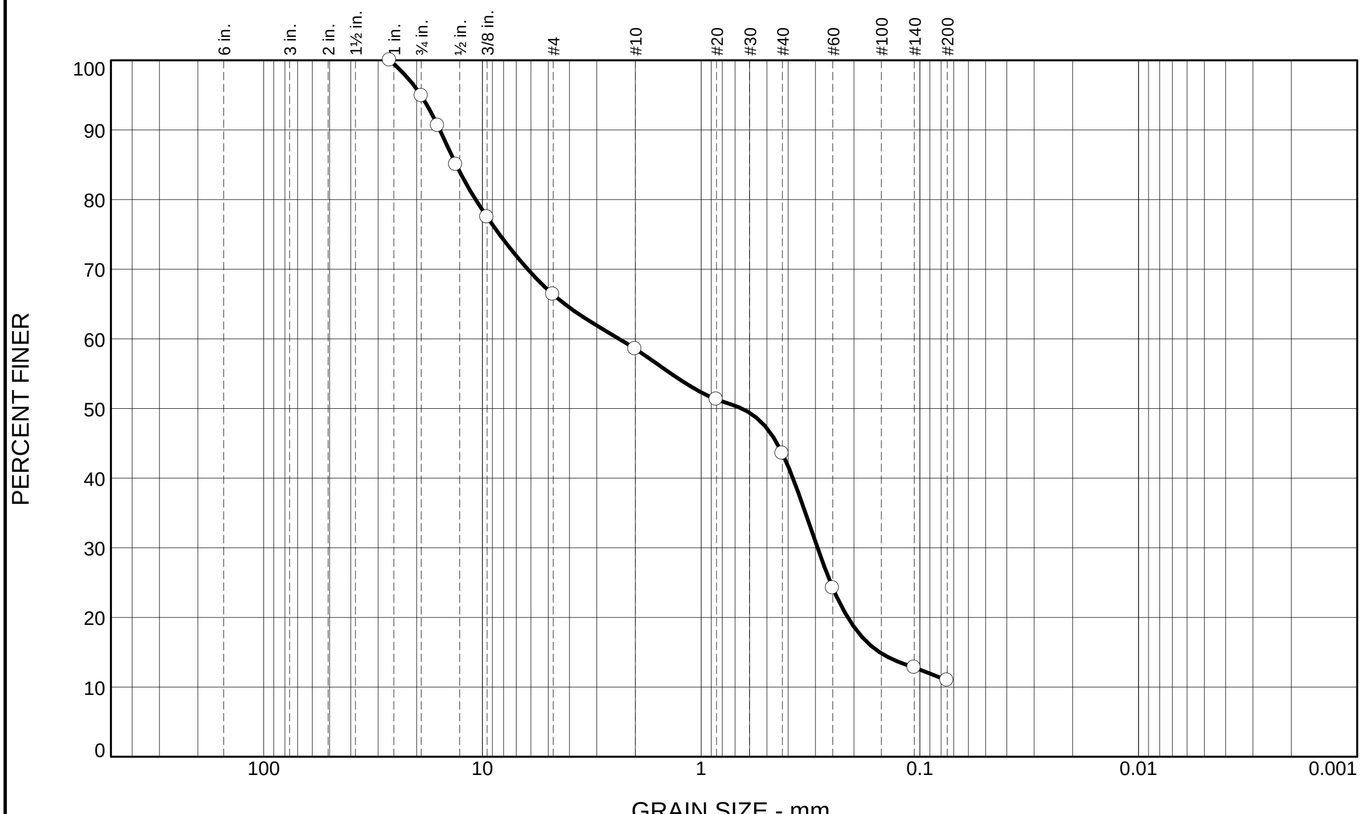
Location: NCC Richmond Landing TP Number: 15-1		Date:12/18/2015
Stratigraphy		**Samples**
Depth (m)	**Material Description**	**Sample No. & Depth (m)**
0 - 0.24	Topsoil - 240 mm	
0.24 - 1.7 m	Silty Sand with boulders, cobbles, red brick, asphalt, concrete and masonry block, brown grey, moist (FILL)	GS:1 0.5 - 0.7
Notes: Testpit terminated at 1.7 m when seepage began entering the testpit		

Location: NCC Richmond Landing TP Number: 15-2		Date:12/18/2015
Stratigraphy		**Samples**
Depth (m)	**Material Description**	**Sample No. & Depth (m)**
0 - 0.20	Topsoil - 200 mm	
0.20 - 1.6 m	Silty Sand with boulders, cobbles, red brick and concrete, brown grey, moist (FILL)	GS:1 0.8 - 1.0
Notes: Testpit terminated at 1.6 m when seepage began entering the testpit		

Appendix II

Results of Laboratory Testing

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.1	28.5	7.8	15.1	32.5	11.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
26.5mm	100.0		
19mm	94.9		
16mm	90.6		
13.2mm	85.0		
9.5mm	77.5		
4.75mm	66.4		
2.00mm	58.6		
0.850mm	51.3		
0.425mm	43.5		
0.250mm	24.2		
0.106mm	12.8		
0.075mm	11.0		

* (no specification provided)

Soil Description

Gravelly sand, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.6494 D_{85}= 13.1844 D_{60}= 2.3716
D_{50}= 0.6516 D_{30}= 0.2936 D_{15}= 0.1526
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

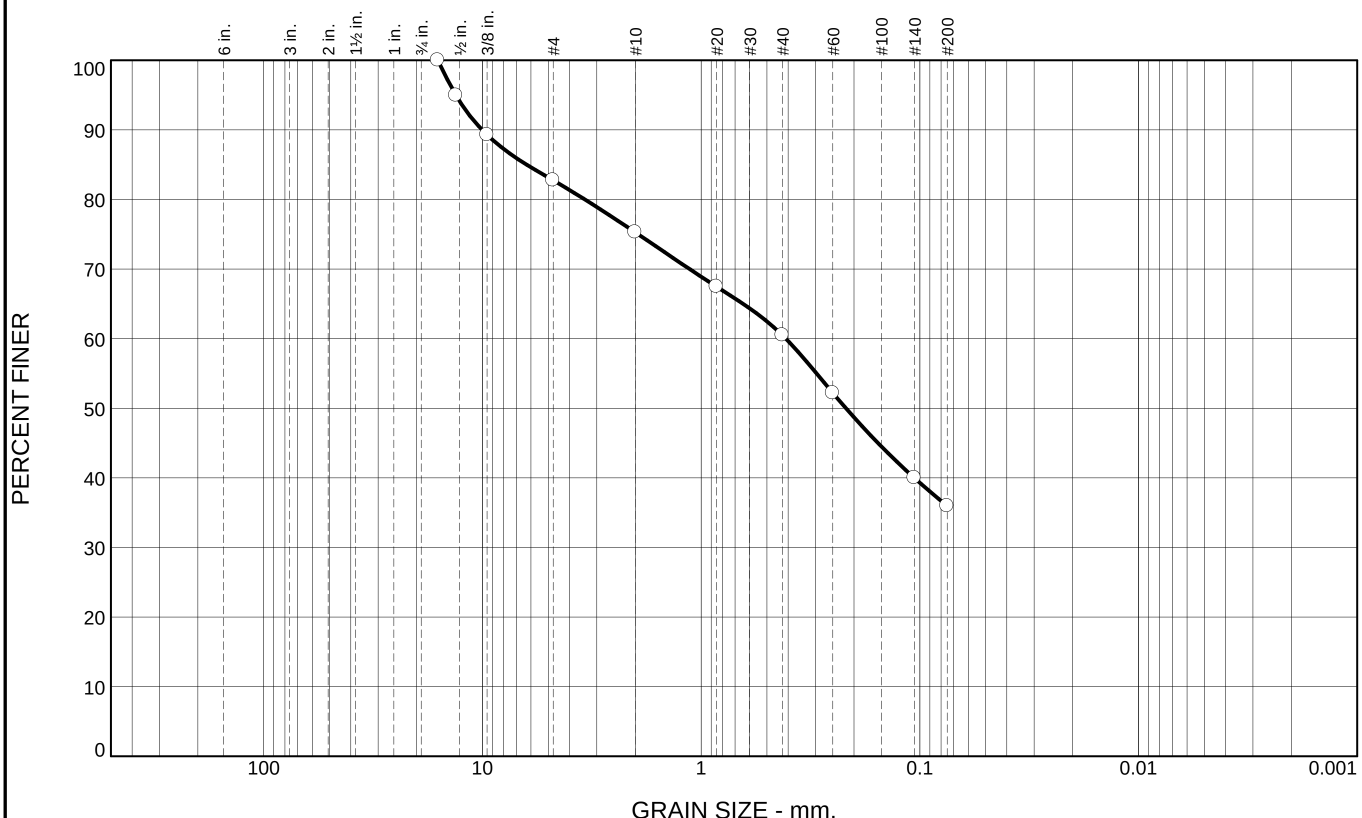
Location: BH15-1 GS1
Sample Number: MM-1977
Date:

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	17.2	7.5	14.8	24.5	36.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
16mm	100.0		
13.2mm	95.0		
9.5mm	89.3		
4.75mm	82.8		
2.00mm	75.3		
0.850mm	67.5		
0.425mm	60.5		
0.250mm	52.2		
0.106mm	40.0		
0.075mm	36.0		

* (no specification provided)

Soil Description

Silty sand, some gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.0322 D_{85}= 6.2815 D_{60}= 0.4097
D_{50}= 0.2177 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

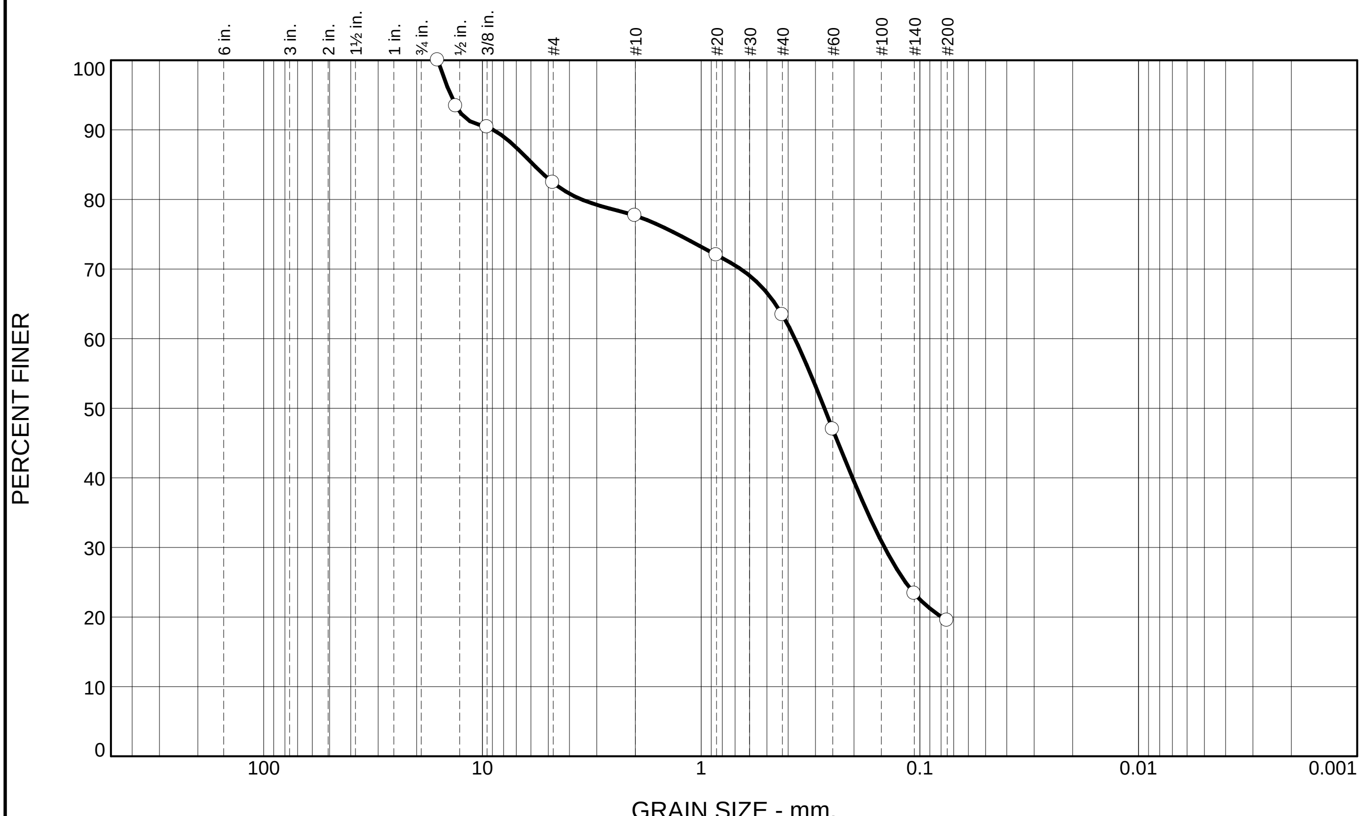
Sampled by D.Wall on July 27-30, 2015

Location: BH15-2 GS1
Sample Number: MM-1978
Date:

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.
Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	17.6	4.7	14.3	43.9	19.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
16mm	100.0		
13.2mm	93.4		
9.5mm	90.4		
4.75mm	82.4		
2.00mm	77.7		
0.850mm	72.0		
0.425mm	63.4		
0.250mm	47.0		
0.106mm	23.4		
0.075mm	19.5		

* (no specification provided)

Soil Description

Sand, some gravel, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 8.9083 D_{85}= 5.8390 D_{60}= 0.3725
D_{50}= 0.2730 D_{30}= 0.1449 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

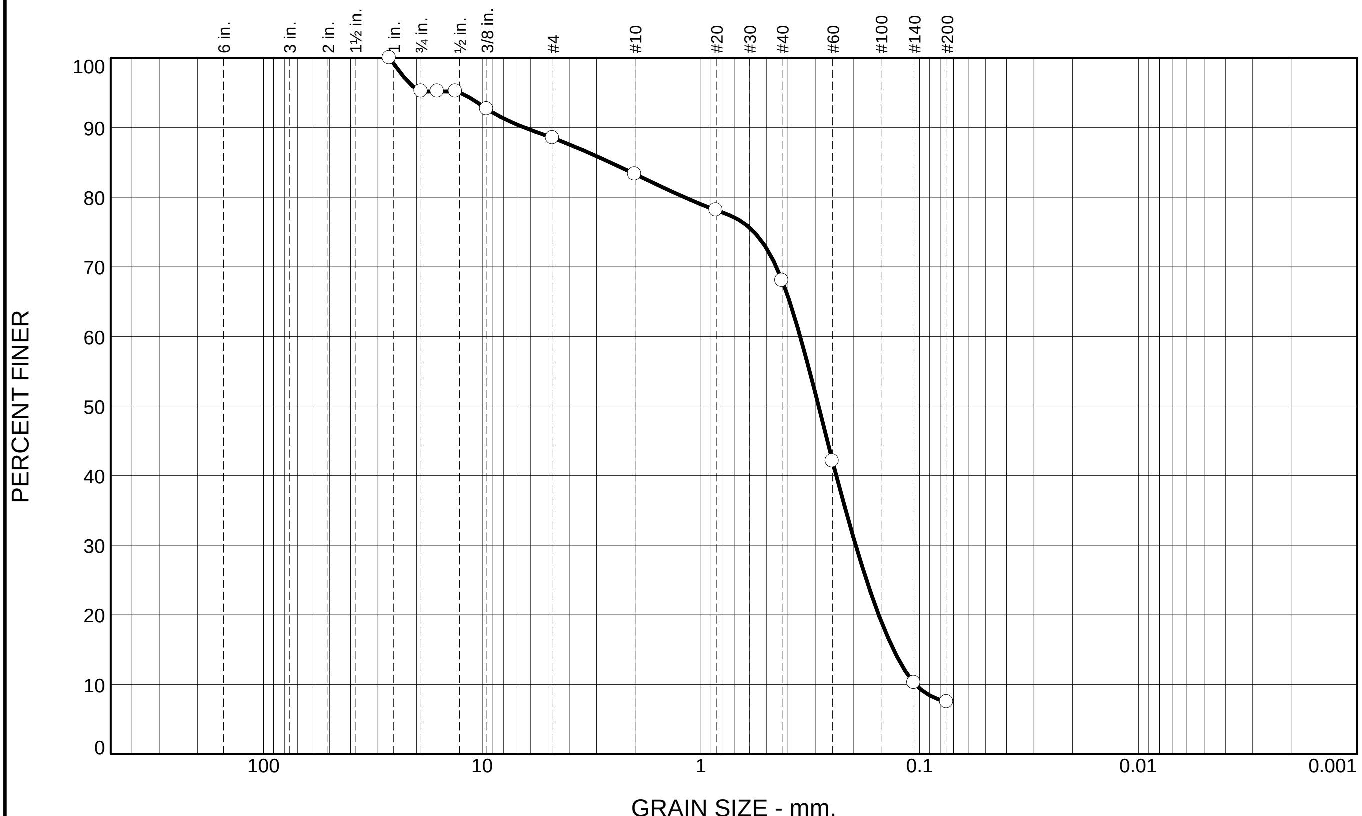
Sampled by D.Wall on July 27-30, 2015

Location: BH15-3 SS1
Sample Number: MM-1979
Date:

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.
Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.8	6.7	5.2	15.3	60.5	7.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
26.5mm	100.0		
19mm	95.2		
16mm	95.2		
13.2mm	95.2		
9.5mm	92.7		
4.75mm	88.5		
2.00mm	83.3		
0.850mm	78.1		
0.425mm	68.0		
0.250mm	42.1		
0.106mm	10.2		
0.075mm	7.5		

* (no specification provided)

Soil Description

Sand, some gravel, trace fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 6.3962 D_{85}= 2.6002 D_{60}= 0.3515
D_{50}= 0.2897 D_{30}= 0.1960 D_{15}= 0.1316
D_{10}= 0.1044 C_u= 3.37 C_c= 1.05

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

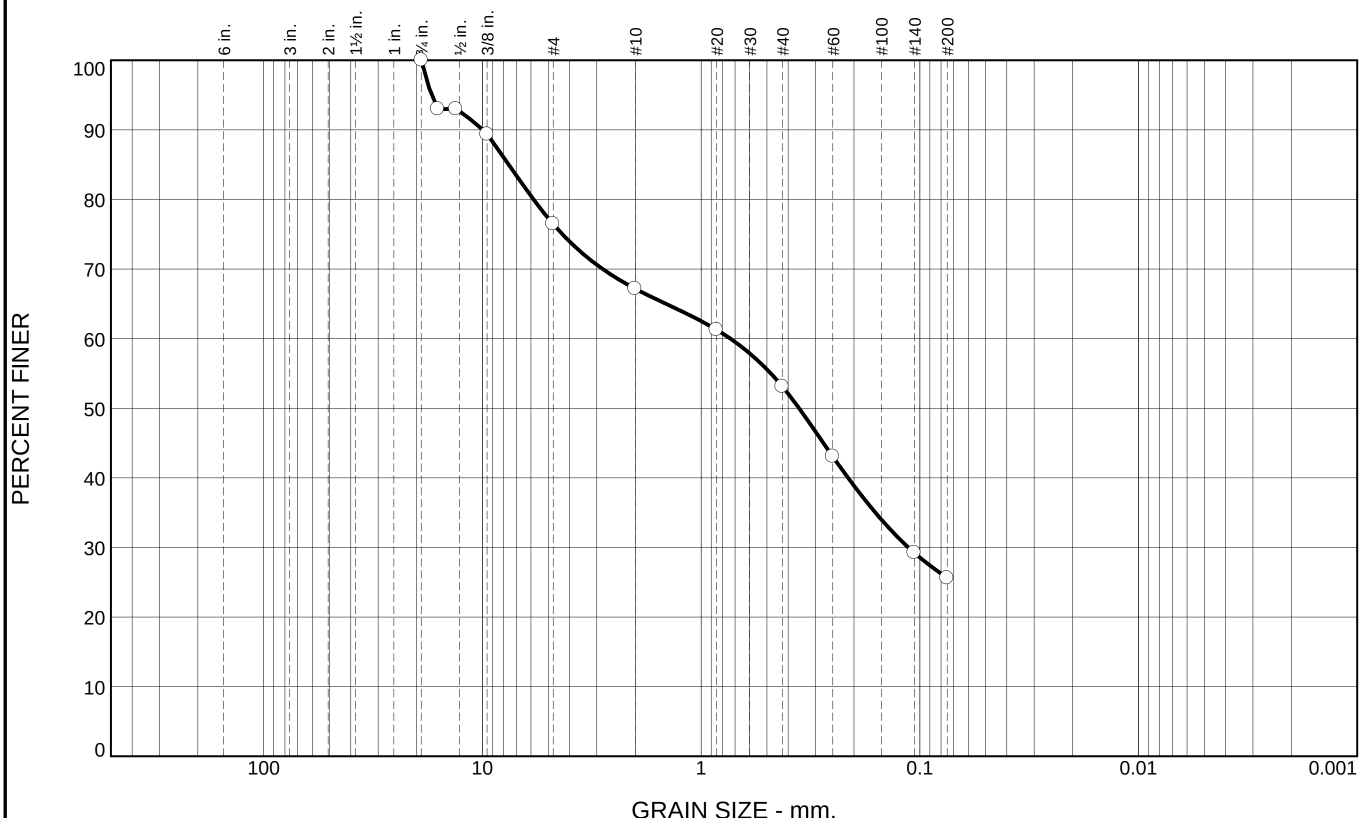
Location: BH15-3 SS2
Sample Number: MM-1989 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	23.5	9.3	14.1	27.5	25.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
19mm	100.0		
16mm	93.0		
13.2mm	93.0		
9.5mm	89.4		
4.75mm	76.5		
2.00mm	67.2		
0.850mm	61.3		
0.425mm	53.1		
0.250mm	43.1		
0.106mm	29.2		
0.075mm	25.6		

* (no specification provided)

Soil Description

Silty sand, gravelly to some

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.9826 D_{85}= 7.5849 D_{60}= 0.7345
D_{50}= 0.3565 D_{30}= 0.1129 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

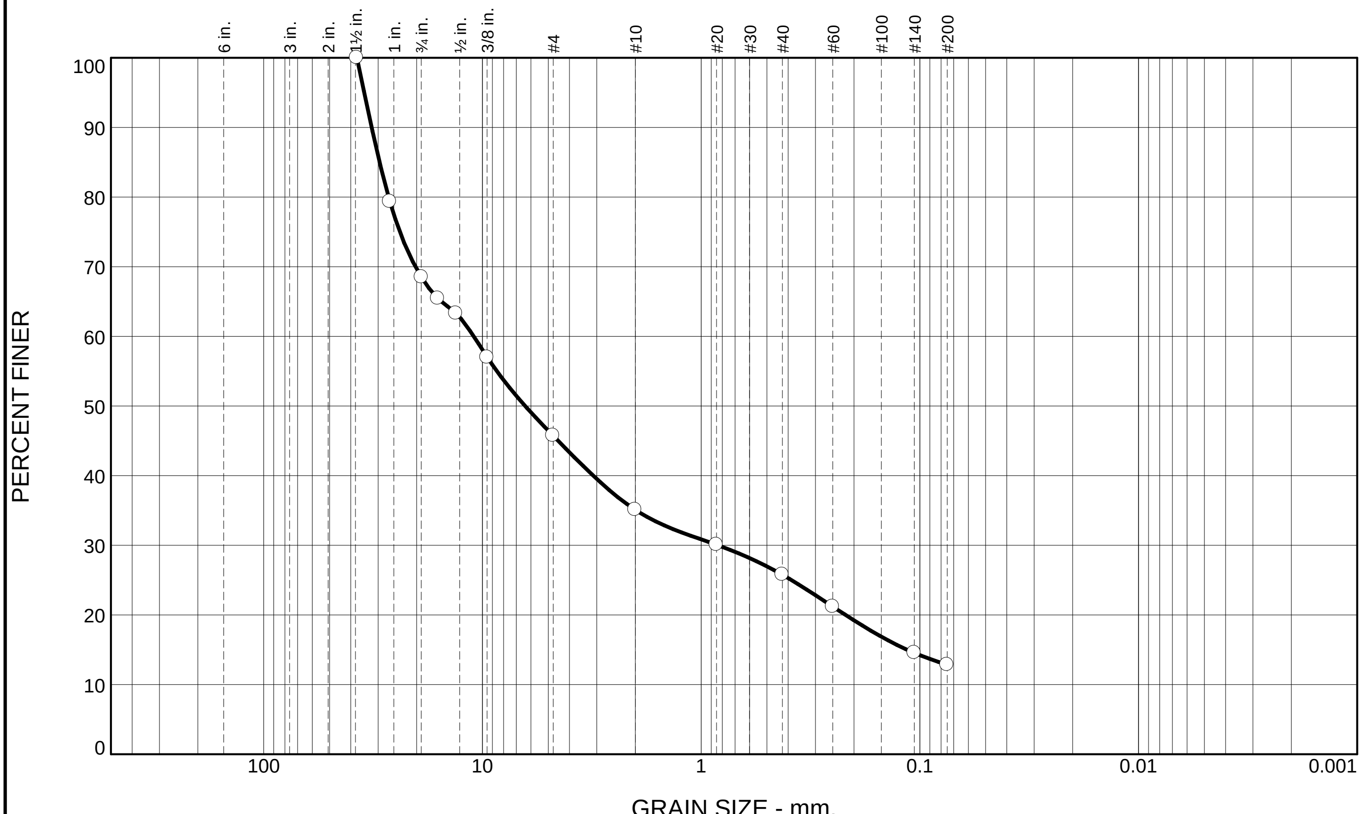
Location: BH15-3 SS3A
Sample Number: MM-1980 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	31.4	22.8	10.7	9.3	13.0	12.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
37.5mm	100.0		
26.5mm	79.4		
19mm	68.5		
16mm	65.5		
13.2mm	63.3		
9.5mm	57.0		
4.75mm	45.8		
2.00mm	35.1		
0.850mm	30.1		
0.425mm	25.8		
0.250mm	21.2		
0.106mm	14.6		
0.075mm	12.8		

* (no specification provided)

Soil Description

Sandy gravel, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 32.1136 D_{85}= 29.5288 D_{60}= 10.9536
D_{50}= 6.3767 D_{30}= 0.8362 D_{15}= 0.1142
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

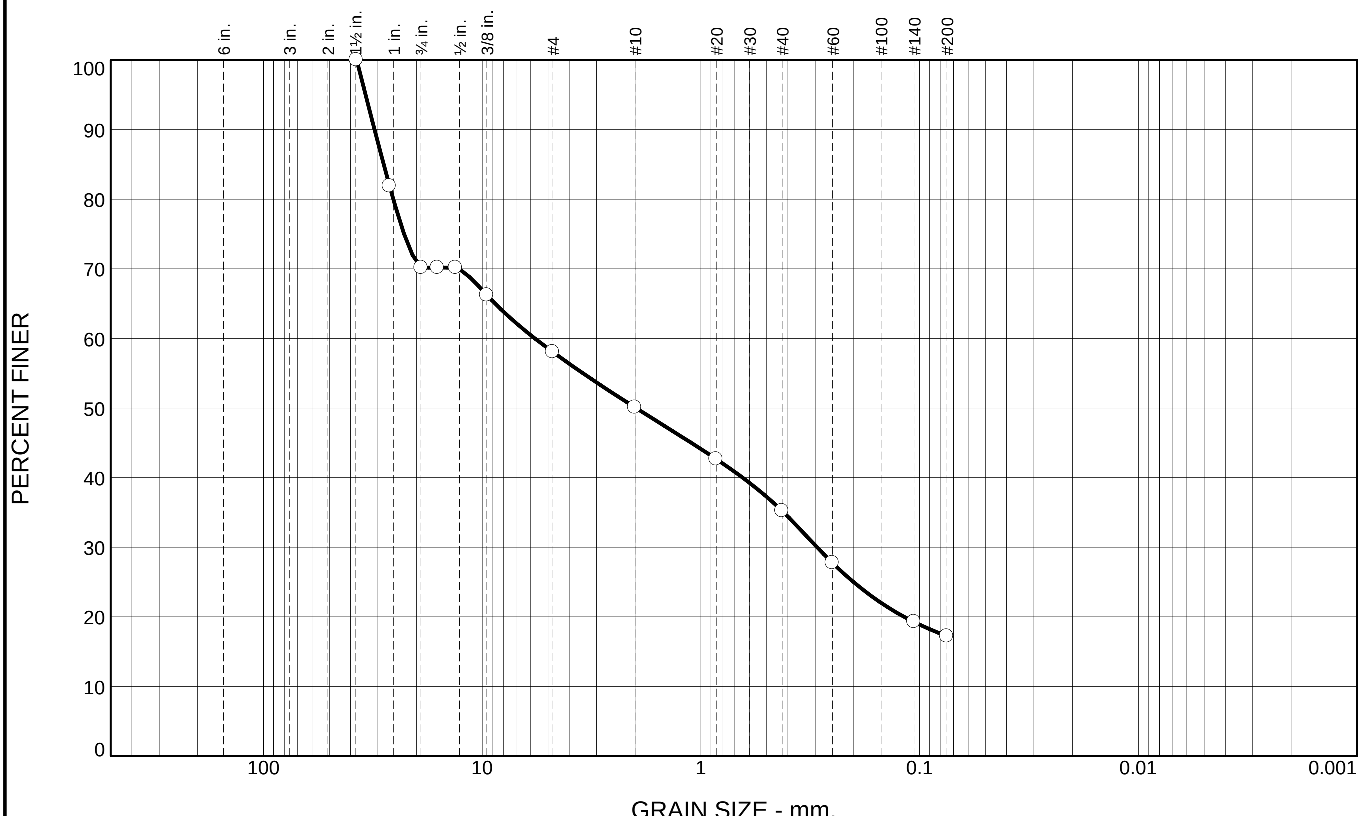
Location: BH15-4 SS2
Sample Number: MM-1981 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	29.8	12.1	8.0	14.9	18.0	17.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
37.5mm	100.0		
26.5mm	81.9		
19mm	70.2		
16mm	70.2		
13.2mm	70.2		
9.5mm	66.2		
4.75mm	58.1		
2.00mm	50.1		
0.850mm	42.7		
0.425mm	35.2		
0.250mm	27.7		
0.106mm	19.3		
0.075mm	17.2		

* (no specification provided)

Soil Description

Sand and gravel, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 31.0759 D_{85}= 28.2071 D_{60}= 5.7387
D_{50}= 1.9811 D_{30}= 0.2941 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

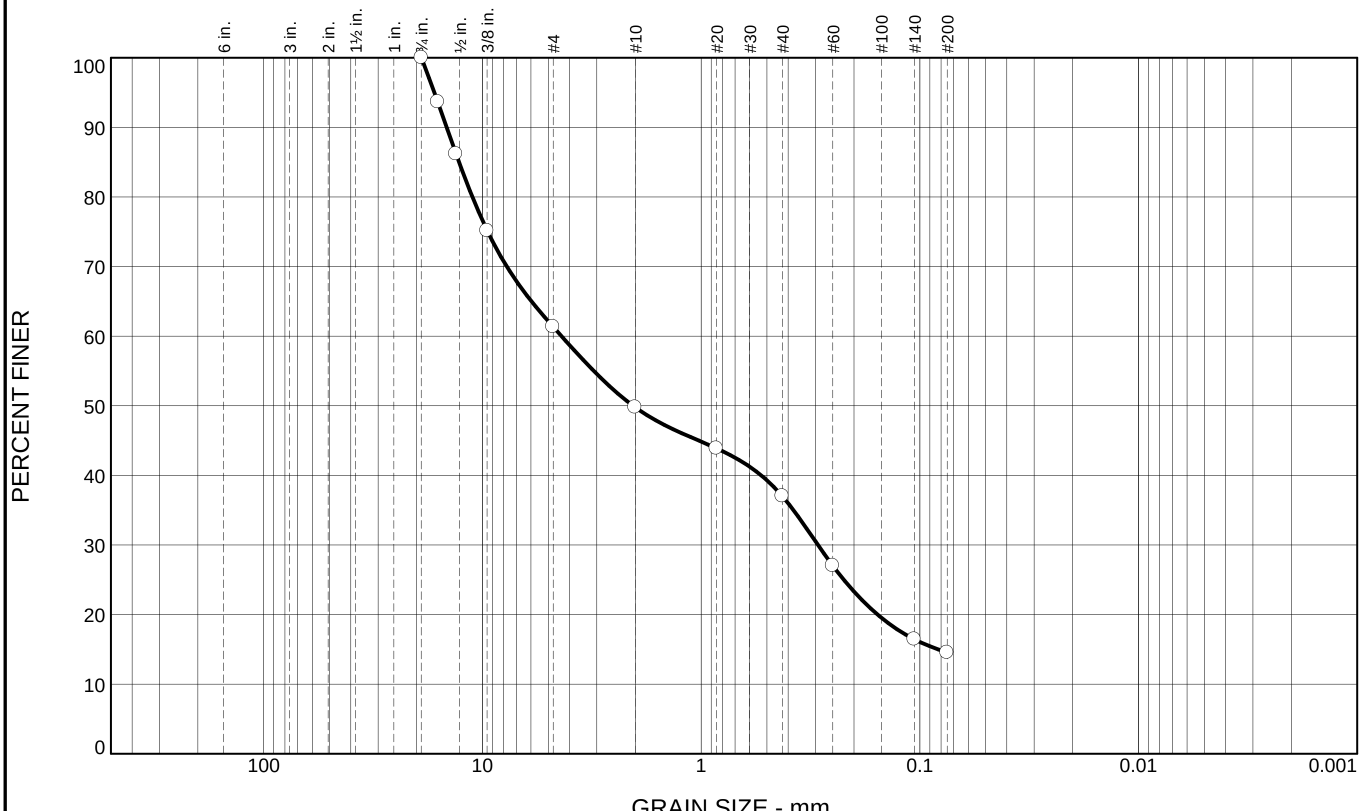
Sampled by D.Wall on July 27-30, 2015

Location: BH15-6 SS3
Sample Number: MM-1984 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.
Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	38.6	11.6	12.8	22.5	14.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
19mm	100.0		
16mm	93.6		
13.2mm	86.2		
9.5mm	75.1		
4.75mm	61.4		
2.00mm	49.8		
0.850mm	43.9		
0.425mm	37.0		
0.250mm	27.0		
0.106mm	16.5		
0.075mm	14.5		

* (no specification provided)

Soil Description

Sand and gravel, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 14.5666 D_{85}= 12.7911 D_{60}= 4.3492
D_{50}= 2.0478 D_{30}= 0.2921 D_{15}= 0.0824
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

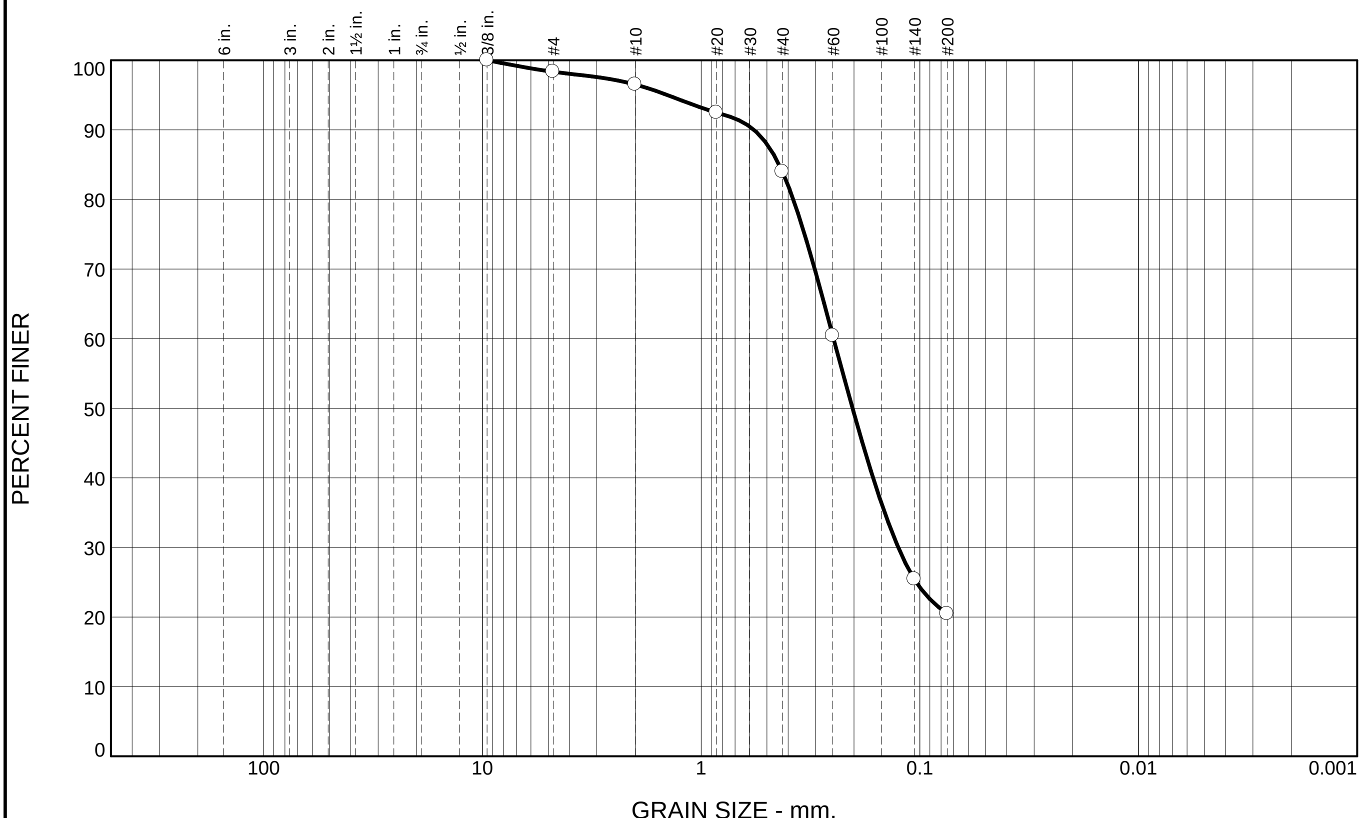
Sampled by D.Wall on July 27-30, 2015

Location: BH15-7 SS3
Sample Number: MM-1985
Date:

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.
Project No: 10001599
Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.6	1.9	12.5	63.5	20.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
9.5mm	100.0		
4.75mm	98.4		
2.00mm	96.5		
0.850mm	92.5		
0.425mm	84.0		
0.250mm	60.4		
0.106mm	25.5		
0.075mm	20.5		

* (no specification provided)

Soil Description

Silty sand, trace gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5730 D_{85}= 0.4400 D_{60}= 0.2480
D_{50}= 0.2031 D_{30}= 0.1255 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

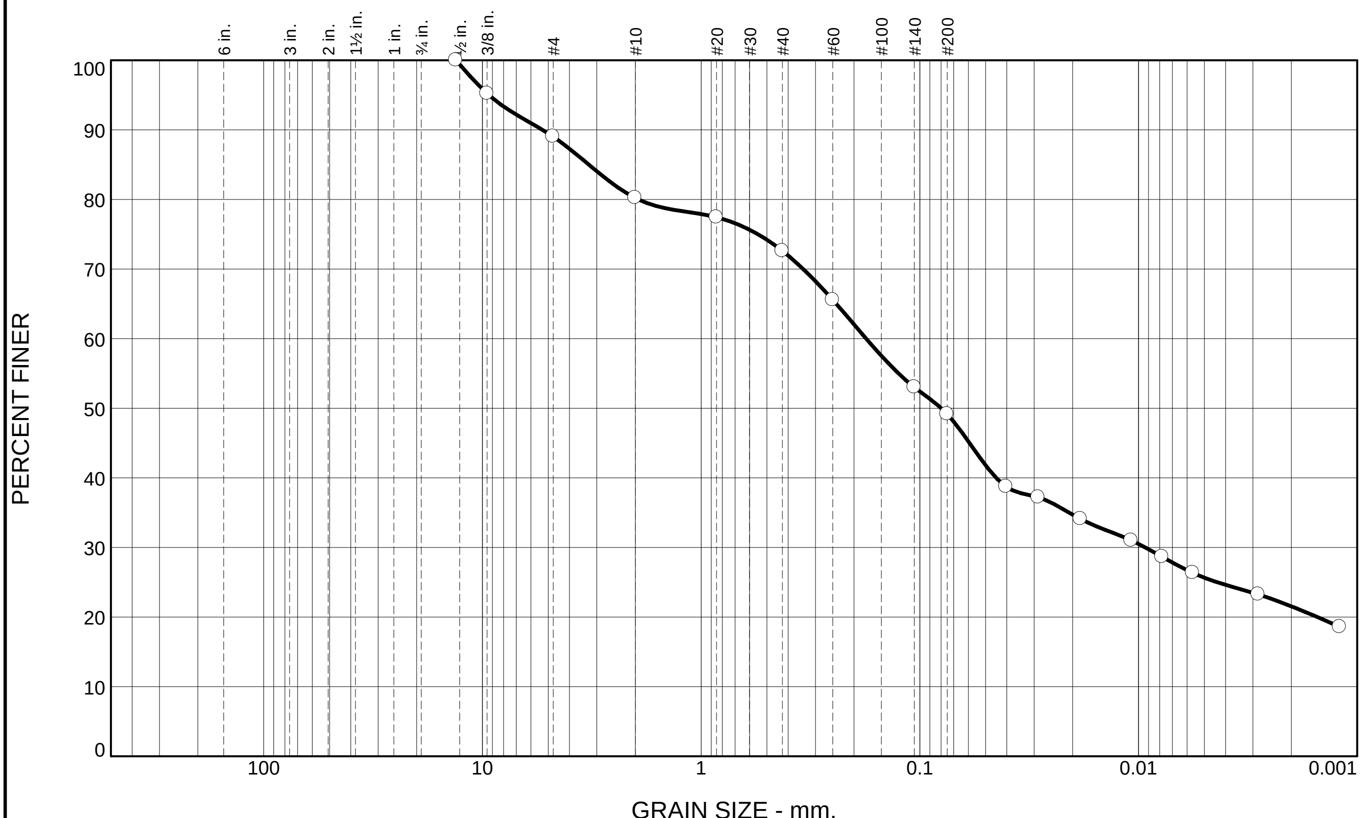
Location: BH15-8 GS1
Sample Number: MM-1986 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.0	8.8	7.6	23.4	27.7	21.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
13.2mm	100.0		
9.5mm	95.2		
4.75mm	89.0		
2.00mm	80.2		
0.850mm	77.4		
0.425mm	72.6		
0.250mm	65.6		
0.106mm	53.0		
0.075mm	49.2		
0.0403 mm.	38.8		
0.0287 mm.	37.2		
0.0184 mm.	34.1		
0.0108 mm.	31.0		
0.0078 mm.	28.7		
0.0056 mm.	26.4		
0.0028 mm.	23.3		
0.0012 mm.	18.6		

* (no specification provided)

Soil Description

Silty sand, some clay, some gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 5.3116 D_{85}= 3.2616 D_{60}= 0.1758
D_{50}= 0.0797 D_{30}= 0.0093 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

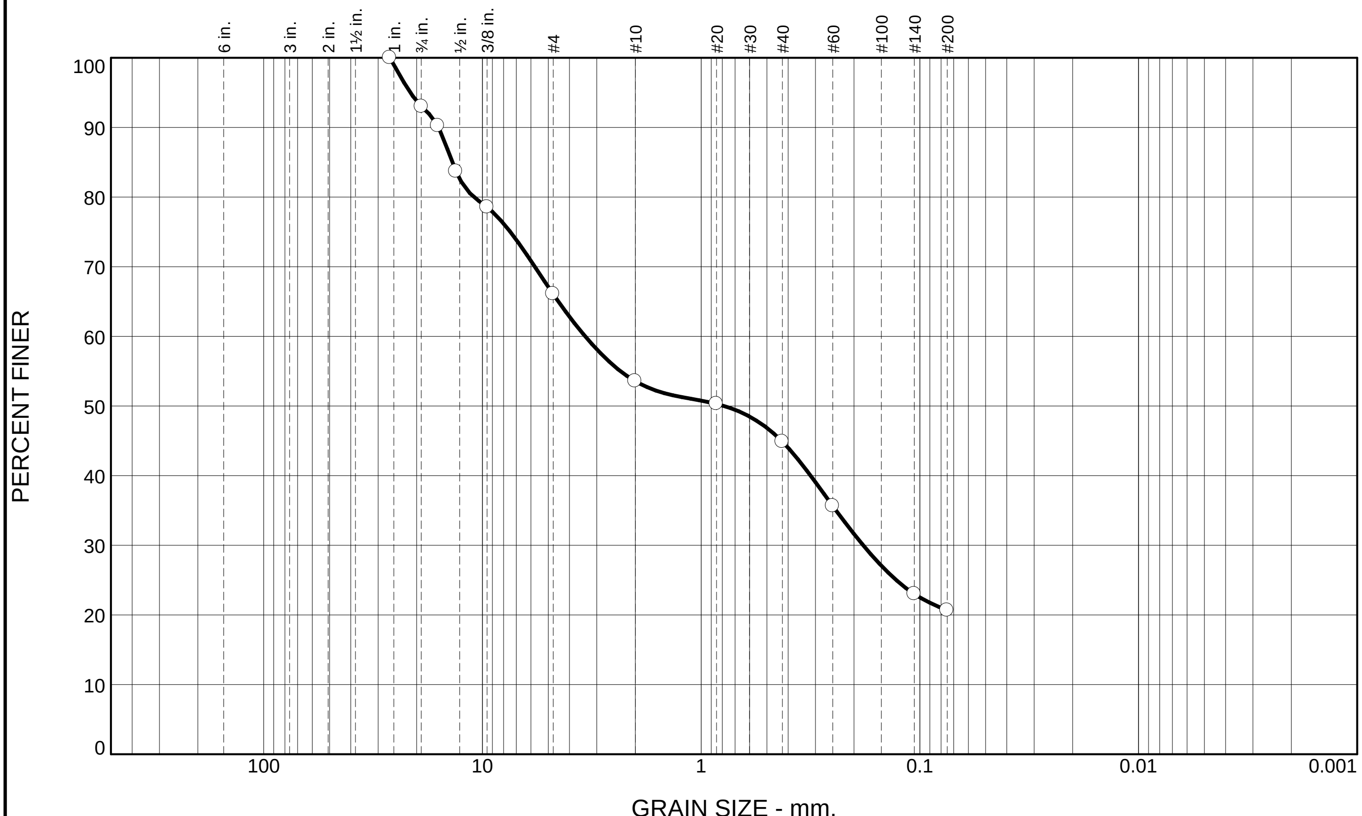
Sampled by D.Wall on July 27-30, 2015

Location: BH15-8 SS3
Sample Number: MM-1987
Date:

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.
Project No: 10001599 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.0	26.9	12.5	8.7	24.2	20.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
26.5mm	100.0		
19mm	93.0		
16mm	90.2		
13.2mm	83.7		
9.5mm	78.5		
4.75mm	66.1		
2.00mm	53.6		
0.850mm	50.3		
0.425mm	44.9		
0.250mm	35.6		
0.106mm	23.0		
0.075mm	20.7		

* (no specification provided)

Soil Description

Gravelly sand, some fines

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.8554 D_{85}= 13.7321 D_{60}= 3.3900
D_{50}= 0.7849 D_{30}= 0.1814 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by D.Wall on July 27-30, 2015

Location: BH15-11 GS1
Sample Number: MM-1988 **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

Client: National Capital Commission
Project: Richmond Landing, Ottawa.

Project No: 10001599 **Figure**

Appendix III

Chemical Test Results

EXOVA **ENVIRONMENTAL ONTARIO**

Certificate of Analysis

Client:	SPL Consultants Ltd. 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1	Report Number:	1515810
		Date Submitted:	2015-08-13
		Date Reported:	2015-08-21
Attention:	Ms. Wendy McLaughlin	Project:	10001599
PO#:		COC #:	505915
Invoice to:	SPL Consultants Ltd.		

Dear Wendy McLaughlin:

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 2015.08.21 11:40:16 -04'00'

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: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Ms. Wendy McLaughlin
PO#:
Invoice to: SPL Consultants Ltd.

Report Number: 1515810
Date Submitted: 2015-08-13
Date Reported: 2015-08-21
Project: 10001599
COC #: 505915

				Lab I.D.	1194919	1194920	1194921	1194922
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2015-07-27	2015-07-27	2015-07-27	2015-07-27
				Sample I.D.	BH 15-1 GS-1	BH 15-2 GS-1	BH 15-3 SS2	BH15-4 SS-3
Group	**Analyte**	**MRL**	**Units**	**Guideline**				
Agri. - Soil	pH	2.0			8.6	8.5	8.2	9.7
General Chemistry	Cl	0.002	%		<0.002	0.003	<0.002	0.003
	Electrical Conductivity	0.05	mS/cm		0.18	0.27	0.18	0.35
	Resistivity	1	ohm-cm		5560	3700	5560	2860
	SO4	0.01	%		0.03	0.02	0.02	0.07

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

Client:	SPL Consultants Ltd. 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1
Attention:	Ms. Wendy McLaughlin
PO#:	
Invoice to:	SPL Consultants Ltd.

Report Number:	1515810
Date Submitted:	2015-08-13
Date Reported:	2015-08-21
Project:	10001599
COC #:	505915

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 292290 **Analysis/Extraction Date** 2015-08-14 **Analyst** DML			
Method Ag Soil			
pH			90-110
Method Cond-Soil			
Electrical Conductivity			85-115
Method Resistivity - soil			
Resistivity			
Run No 292836 **Analysis/Extraction Date** 2015-08-20 **Analyst** NP			
Method C SM4500-SO4--D			
SO4	<0.01 %	98	70-130
Run No 292839 **Analysis/Extraction Date** 2015-08-20 **Analyst** NP			
Method C CSA A23.2-4B			
Chloride	<0.002 %	100	90-110

Exova

CHAIN OF CUSTODY

505915

- ☐ 146 Colonnade Rd., Unit 8, Ottawa, ON K2E 7Y1 Ph: (613) 727-5692 Fax: (613) 727-5222
- ☐ 608 Norris Court, Kingston, ON K7P 2R9 Ph: (613) 634-9307 Fax: (613) 634-9308
- ☐ 380 Vansickle Rd., Unit 630, St. Catharines, ON L2R 6P7 Ph: (905) 680-8887 Fax: (905) 680-4256
- ☐ 2395 Speakman Drive, Mississauga, ON, L5K 1B3 Phone: (905) 822-4111 Fax: (905) 823-1446

LABORATORY USE ONLY
Report #: 1518160

Report Information*:
Client: SPL Consultants
Contact: W.A. McLaughlin
Address:
Email: W.A. McLaughlin Phone:
Project: 10001599

Invoice Information*:
Invoice to the same as above? Yes / No, or:
Client:
Contact:
Address:
Email: Phone:
Purchase Order #:
Exova Quote # *: 140470

Criteria Required*:
- ☐ ODWSOG
- ☐ PWQO
- ☐ Ont. Reg. 558
- ☐ CCME
- ☐ Sanitary Sewer, City:
- ☐ Storm Sewer, City:
- ☐ Ont. Reg 153/04
  Table # ___, Coarse/Fine, Surface/Subsurface
  Type: Com-Ind / Res-Park / Agri / GW / Other
- ☐ Other, Specify:

The sample results from this submission will form part of a formal Record of Site Condition (RSC) under O.Reg. 153/04 *:
YES / NO

Is this a drinking water sample? YES / NO *
If yes, complete the drinking water COC

Additional Email/Fax:
1. Email:
2. Email:
3. Email:
Fax:

Report Format:
☐ PDF ☐ Excel ☐ Other, Specify:

Turnaround Time (*rush surcharges may apply*)*:
- ☒ 5 Business Days (Standard)
- ☐ 3 Business Days (Rush)
- ☐ 2 Business Days (Rush)
- ☐ 1 Business Day (Rush)
- ☐ Other (specifiy date):

Notes:

* Indicates a required field
Please note that incomplete information may result in turnaround time delays.
Samples should be kept cool (4-10°C) from sampling time through drop-off at the laboratory.

Sample ID*	Date/Time Sampled*	Sample Matrix*	# Bottles	Sample Location	BTEX	F1-F4	Full VOCs	PAHs	PCBs	R153 Metals Only	Metals+CrVI,Hg,B	Metals & Inorgs	TCLP Metals	TCLP Inorgs							Lab Use Only
BH 15-1 GS-1	27/07/15	Soil																			1194919
BH 15-2 GS-1	↓	↓				please test for corrosivity															20
BH 15-3 SS-2																					21
BH 15-4 SS-3																					22

Samples Relinquished By:	Date/Time:	Samples Received By:	Date/Time:	Temperature:	Condition:
				—	—
Samples Relinquished By: WAMcLaughlin	Date/Time: 13/08/15	Samples Received By: [signature]	Date/Time: 13/08 1045	Page # ___ of ___	

Appendix IV

Results of Geophysical Survey

GEOPHYSICS GPR INTERNATIONAL INC.
6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9
Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

February 5, 2015 Our File: T15840

Chris Hendry, M.Eng., P.Eng.
Sr. Geotechnical Engineer
WSP Canada Inc.
2611 Queensview Drive
Ottawa, ON
K2B 8K2

RE: Georadar and Seismic surveys for bedrock mapping on Victoria Island, Ottawa, Ontario.

Dear Mr. Hendry:

Geophysics GPR International Inc. was requested by WSP to perform a geophysical survey at the above site. The purpose of this investigation was to map the bedrock profile. The survey was performed from December 11 to 14, 2015. Figure 1 shows the site locations, in Ottawa.

Radar Survey Design

Georadar uses radar technology to obtain a near continuous profile of the subsurface. The basic principle is to send an electromagnetic impulse into the ground. This pulse will travel through the earth and reflect off boundaries of differing dielectric constants. A reflected pulse returns to the surface and is recorded by a receiver. Examples of boundaries included air/water (water table); water/earth (bathymetry); earth/metal, PVC, or concrete (pipe locating); and differing earth materials (stratigraphic profiles, including bedrock profiles). Only by moving the antennas along a profile directly over the targets can the locations and depths be determined. All data are generated in real time and recorded digitally. The 270 MHz antenna was used for this survey. This particular antenna is most appropriate for depth penetration in the upper 5–10 m, depending on material.

Radar reflections are generated at the boundaries of materials with contrasts in electromagnetic properties (dielectric value). Interpretation of radar data is based primarily on the qualitative analysis of two characteristics of radar reflections: continuity and amplitude. The true nature (cause) of a radar reflection can only be assumed with corroboration from intrusive methods or additional geophysical techniques.

The operator performed several passes along profiles from East to West. The profiles were taken along the embankments for the locations 1, 2, 3 and 4 from north to south. Data was restricted to the upper embankment and away from the brush areas. Good radar data needs to be moved in a smooth manner and stay close to or on the ground. Data collection would not have been possible in the brush areas or on the steep slopes with boulder cover material.

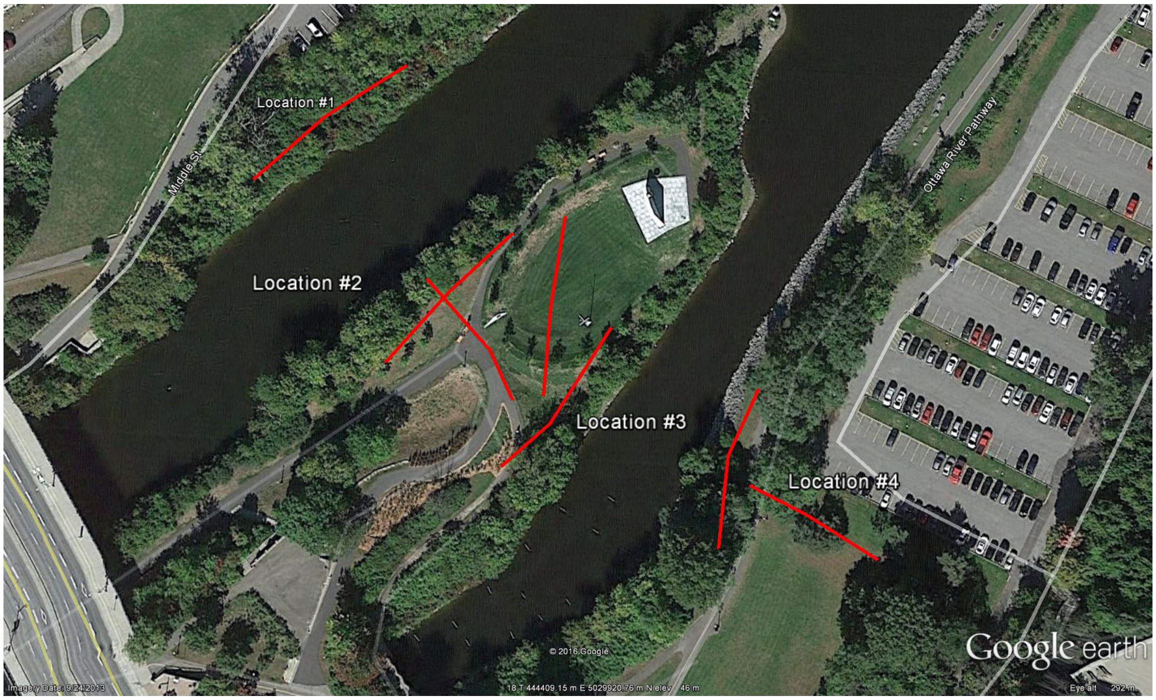


Figure. 1. Victoria Island georadar and seismic survey site.

Seismic Survey Design

The seismic refraction method relies on measuring the transit time of the wave that takes the shortest time to travel from the impact point (shot-point) to a series of receivers (geophones/hydrophones). The fastest seismic waves are the compressional (P) or acoustic waves, where displaced particles oscillate in the direction of wave propagation.

A seismic spread of 24 vibration monitoring devices (geophones) connected in line (spread) to a seismograph (ABEM Terraloc Mark 6) by connector cables. Seismic pulses (shots) are then generated at various locations with respect to the spread. The profiles for this seismic investigation used a spacing of two metres between geophones. Typically seven or more shots are executed per seismic spread: three to five shots within the profile to obtain the lateral velocity variation in the overburden and two shots on either side of the spread to provide the true velocity of the bedrock surface. A sledge hammer with a metal plate were used to generate the seismic signal.

Results

The background seismic noise levels at this site were high. The quality of the seismic records wasn't ideal. In fact, there seemed to be a pervasive noise source but it was not clear what it was. The results of seismic refraction investigation are summarized in the Figures 2 to 4.

The data quality was not sufficent to perform a full refraction calculation beneath every geophone but there could be a simple calculation refered to as 'critical distance'. This produces a rock depth beneath some of the shot locations.

Location #1

Figure 2 shows the location of the only seismic line parallel with the river. A second line was planned but there was a confrontation with a demonstrator so the second planned seismic line was cancelled. In addition, there was no ground radar profiles collected either. The seismic line crosses over the borehole with a rock depth of 1.6 meters. It was not feasible or safe to position the profile closer to the shoreline due to a combination of steep slope, boulders and brush growth.

Fig. 2. Surveyed locations with approximate bedrock depths.

Location #2

There was a seismic spread collected parallel with the river that crossed over a borehole that was 3.4 meters deep (Figure 3). There was also a spread perpendicular that also crossed the same borehole. There were no geophones down the slope to the river due to boulders but it was possible to obtain a hammer hit near the water. The drop in elevation is believed to be 2.5 meters and the approximate rock depth is 3.2 meters.

It was not feasible or safe to position the profile closer to the shoreline due to a combination of steep slope, boulders and brush growth.

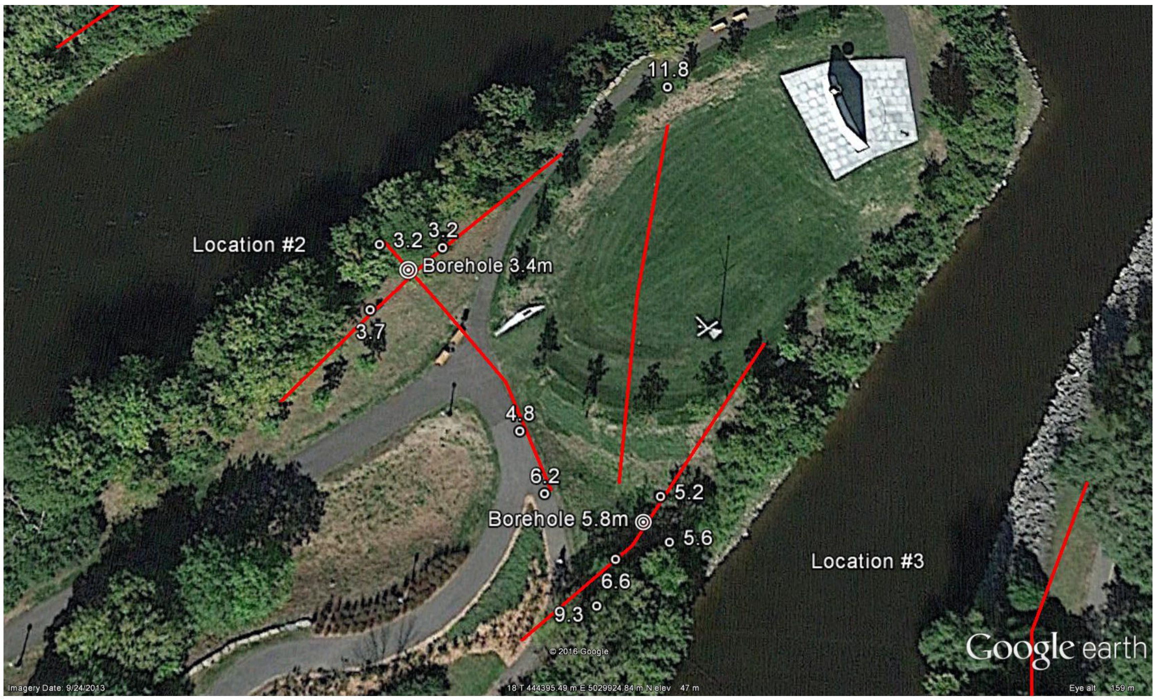


Figure 3: Locations 2 and 3 with bedrock depths.

Location #3

There was a seismic spread collected parallel with the river that crossed over a borehole that was 5.8 meters deep (Figure 3). The borehole was about 4 meters set back from the edge of the slope. There was also a spread at 45 degrees in the hope of obtaining a rock depth closer to the shoreline. There were no geophones down the slope to the river due to boulders and it was not possible to safely obtain a hammer hit near the water. The drop in elevation is believed to be 2.5 meters and the approximate rock depth is 4.0 meters.

It was not feasible or safe to position the profile closer to the shoreline due to a combination of steep slope, boulders and brush growth.

Location #4

There was a seismic spread collected parallel with the river that crossed over a borehole that was 6.3 meters deep (Figure 4). The borehole was about one meter set back from the edge of the slope. There was also a spread perpendicular to the river. There were no geophones down the slope to the river due to boulders but it was possible to obtain a hammer hit near the water.

It was not feasible or safe to position the profile closer to the shoreline due to a combination of steep slope, boulders and brush growth.

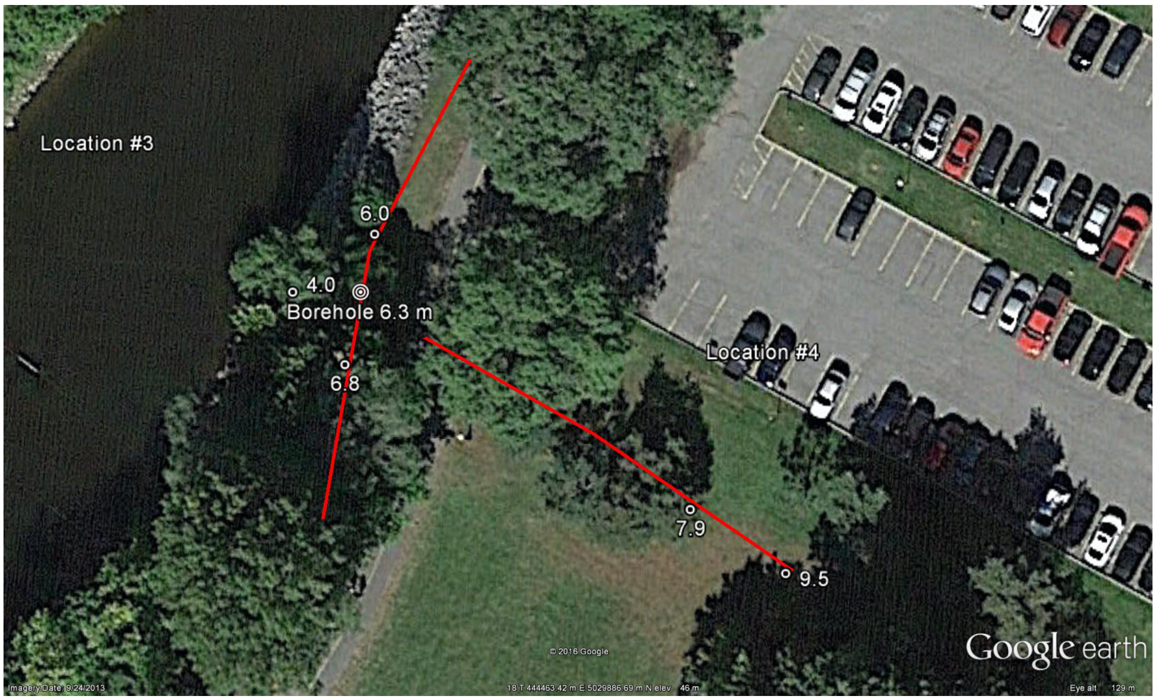


Figure 4: Location #4 with bedrock depth calculations

Ground Radar Results

There were 4 to 5 parallel profiles of ground penetrating radar collected parallel with the river with 2m spacing at each location with the exception of Location #1. There was also one perpendicular profile that crossed the borehole.

The penetration of the radar signal into the ground was quite good through the fill but there was a strong reflection from what appears to be native material. In most cases the fill material is 1.5 to 2 meters thick. Figures 3 and 4 are a typical example of an image of the overburden. There is homogeneous layers of predominantly fine sand and maybe some silt that makes up the fill. It is uncertain what the native material is composed of but borehole records show a material similar to the fill with sands, silts and gravels. The signal penetration depth in this zone is not more than two meters. All three locations appeared very similar. The stratigraphy layers are continuous for the most part and consistently alternating indicative of strong homogeneity. There were no boulders detected in the upper 4m. The only boulder may be those used for shoreline protection.

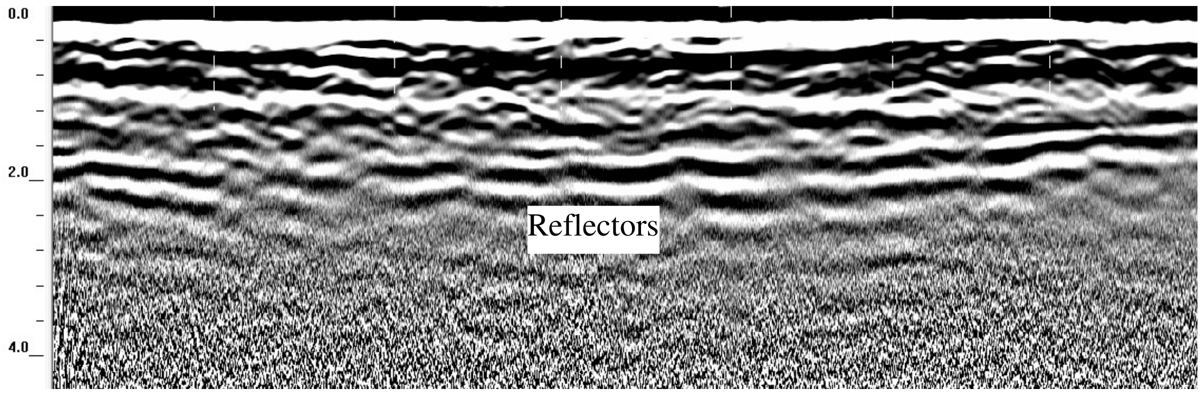


Fig. 3. Example radar image from location 2

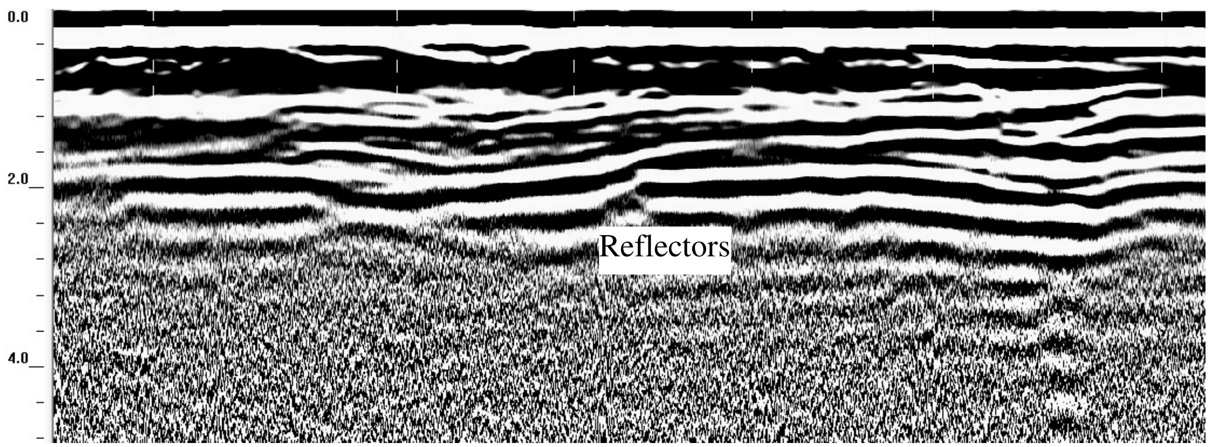


Fig. 4. Example radar image from location 4.

This letter has been written by Milan Situm, P.Geo.

If you have any questions please do not hesitate to call.

Sincerely,

Milan Situm

Milan Situm, P.Geo.
Manager

Appendix V

Explanation of Terms Used in This 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 VI

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 SPL Consultants Limited at the time of preparation. Unless otherwise agreed in writing by SPL Consultants Limited, 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.

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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.