GEOTECHNICAL INVESTIGATION REPORT

339 CUMBERLAND STREET, OTTAWA, ON

Prepared for:

EcoCorner Inc.
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Ottawa, ON
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FINAL REPORT

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1. INTRODUCTION

DST Consulting Engineers Inc. (DST) was retained by Mr. Bruce Yateman of EcoCorner Inc. to carry out a geotechnical investigation for the proposed nine (9) storey building with one (1) basement level to be located at 339 Cumberland Street in Ottawa, Ontario. The geotechnical investigation was completed in general accordance with the work plan described in DST's proposal submitted on August 31, 2014, and approved by Mr. Bruce Yateman.

A Phase I Environmental Site Assessment (ESA) of the site was undertaken by DST in conjunction with this geotechnical investigation. The results of the Phase I ESA are reported under separate cover.

This report is prepared for the sole use of EcoCorner Inc. and any use of the report or any reliance on it by any third party is the responsibility of such third party.

This geotechnical engineering report is subject to the limitations shown in Appendix A.

2. PROJECT DESCRIPTION

The site of the proposed development is located at 339 Cumberland Street, Ottawa, ON. The site is a paved area, square in shape, and bounded to the southwest by Cumberland Street, to the southeast by York Street, to the northwest by a two (2) storey commercial building, and to the northeast by a two (2) storey apartment building. The location of the site is shown in Figure 1, Appendix B.

Based on the proposed development plan provided by Mr. Bruce Yateman, the proposed building will have nine (9) storeys and one (1) basement level. Details regarding the building footprint size, location on site, basement floor slab elevation and final site grades were not available at the time of this investigation. For purposes of this investigation, it is assumed the basement floor will be set at 3.0 m below existing grade.

3. AVAILABLE INFORMATION

DST was provided with the following reference report:

- “Phase II Environmental Site Assessment, 333-335 Cumberland Street, Ottawa, Ontario”, dated December 2003 and prepared by Jacques Whitford Environment Ltd.

DST previously conducted Phase I Environmental Site Assessment and groundwater monitoring for the area. The results of the assessment and monitoring are summarized in the following reports:

- “Environmental Groundwater Monitoring Event – October 2010, 333-335 Cumberland Avenue, Ottawa, Ontario”, dated November 8, 2010;
- “Environmental Groundwater Monitoring Event – August 2009, 333-335 Cumberland Avenue, Ottawa, Ontario”, dated September 2, 2009;
- “Phase I Environmental Site Assessment, 333-335 Cumberland Avenue, Ottawa, Ontario”, dated August 20, 2007;
- “Environmental Groundwater Monitoring Event – April 2007, 333-335 Cumberland Avenue, Ottawa, Ontario”, dated May 7, 2007;
- “Environmental Groundwater Monitoring Event – May 2006, 333-335 Cumberland Avenue, Ottawa, Ontario”, dated May 30, 2006.

4. SCOPE OF WORK

The purpose of this geotechnical investigation is to provide geotechnical engineering comments and recommendations regarding the design and construction of the proposed building.

DST has completed the following scope of work to meet the project requirements:

Fieldwork:

- Placement of two (2) boreholes at the site and advanced to auger refusal depths of 6.3 and 7.6 m on inferred bedrock.
- Confirm the presence of bedrock by coring 5.0 m length of the bedrock in one (1) of the boreholes to termination depth of 11.3 m.
- Install a standpipe piezometer in each borehole for long term monitoring of groundwater conditions.

Laboratory Work:

- Moisture content determination on all recovered soil samples.
- Grain size analysis and Atterberg Limits determination of selected soil samples.
- Point load and uniaxial compressive strength tests of selected bedrock cores.
- Chemical analyses of selected soil samples for potential of sulphate attack on buried concrete structures and corrosion potential on buried steel structures.

Engineering Analyses and Reporting:

- Assessment of the subsurface soil, bedrock and groundwater conditions at the two (2) borehole locations.
- Site grade raise restrictions.
- Foundation recommendations.
- Slab-on-grade construction.
- Frost penetration and protection.
- Seismic site classification and comment on the liquefaction potential of soils.
- Lateral earth pressures on subsurface walls and backfill requirements.
- Excavation and de-watering requirements during construction.
- Corrosion potential of soils.

5. PHYSICAL GEOLOGY

The area in which the site is situated consists of Post-Champlain Sea deposits. This includes medium grained, stratified sand with some silt, in the form of fluvial terraces and channels cut into marine clay, and bars and spits within abandoned channels (NRCAN Surficial Geology MAP 2140A – Lower Ottawa Valley, 2009). The bedrock consists of limestone of the Eastview formation.

6. FIELD INVESTIGATION AND LABORATORY TESTING

6.1 Field Investigation

The field work was conducted on Nov. 17th, 2014 and consisted of two (2) sampled boreholes (BHs 1 and 2) advanced to auger refusal depths of 6.3 and 7.6 m on inferred bedrock. BH 1 extended into the bedrock to a termination depth of 11.3 m. The borehole locations are shown in Figure 2, Appendix B.

The borehole locations and elevations were determined on site by DST. The following temporary benchmark was used to determine the borehole elevations:

> Temporary Benchmark (TBM): Top of fire hydrant located at northwest corner of the intersection of Cumberland Street and York Street.
>
> Elevation: 100.00 m (assumed elevation) (refer to Figure 2, Appendix B).

The boreholes were advanced using a CME-75 drill rig equipped with soil sampling and rock coring capabilities. The borehole work was supervised on a full time basis by a geotechnical representative from DST.

Standard penetration tests (SPTs) were undertaken in each borehole at regular depth intervals with soil samples retrieved by the split spoon sampler. Auger samples of the soil were retrieved at selected depth intervals. Two (2) relatively undisturbed thin walled tube samples (Shelby tubes) were obtained at selected depths from the boreholes. In-situ field vane tests were performed in the clay soil. The bedrock was cored in one (1) borehole using conventional coring methods. One 19 mm diameter standpipe piezometer with 1.5m screened length was installed in the bedrock at BH 1 and in the overburden at BH 2 for groundwater level monitoring purposes. All boreholes were backfilled upon completion of drilling and sampling operations.

The subsurface stratigraphy encountered in each borehole was recorded by DST representative and the recovered soil samples and rock cores were labelled accordingly and submitted to the laboratories for detailed visual examination and laboratory testing.

6.2 Laboratory Testing

Laboratory testing of the soil samples consisted of moisture content determination on all samples. Grain size analysis and Atterberg Limit determination were conducted on selected soil samples. Chemical analyses limited to pH, sulphate, chloride and resistivity were also conducted on selected soil samples.

Laboratory testing of selected rock cores consisted of point load and uniaxial compressive strength tests.

7. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered in the boreholes are given in the borehole logs shown in Appendix C and are discussed in detail below. The moisture content, grain size analysis, and Atterberg Limit results are shown in the borehole logs in Appendix C and in Appendix D. The uniaxial compressive strengths of the rock cores are shown in Appendix D.

7.1 Asphalt

Asphalt was surficially encountered in both boreholes with a thickness of 50 mm.

7.2 Fill

Fill was contacted beneath the asphalt in both boreholes extending to depths of 1.4 and 1.6 m below existing grade. The fill consisted of a mixture of clay, silt and sand to gravelly sand with rootlets and brick pieces.

SPT 'N' values of the fill vary from 4 to 11 indicating a loose to compact condition. The moisture content of tested samples ranges from 5 to 22 %.

7.3 Clay-silty

The fill was underlain by very stiff clay extending to depths of 3.4 and 5.0 m below existing grade.

The field vane test results are greater than 105 kPa indicating a very stiff consistency for the silty clay. The results from grain size analysis and Atterberg Limits tests conducted on selected samples are summarized in Tables 7.1 and 7.2, respectively. The moisture content of the tested samples vary between 34 and 51%.

Table 7.1 Sumary of Grain Size Analysis Results

Sample ID	Grain Size Analysis	
BH2 SS3 (1.5 – 2.1 m)	Gravel %	0
	Sand %	19
	Silt %	42
	Clay %	39

Table 7.2 Summary of Atterberg Limit Test Results

Sample ID	Atterberg Limits	
BH2 SS4 (2.3 – 2.9 m)	Liquid Limit %	59
	Plastic Limit %	20
	Plasticity Index %	39

7.4 Till

Silty sand to sand and gravel till with possible cobbles and boulders was encountered at 3.5 and 5.0 m depths below existing grade in BHs 1 and 2, respectively. The till was 2.5 m thick in BH1 and extended to 6.0 m depth.

SPT ‘N’ values of the till range from 4 to greater than 50 indicating a loose to very dense condition. The results of the grain size analysis are summarized in Table 7.3. The moisture content of tested samples ranges from 7 to 10%.

Table 7.3 Summary of Grain Size Analysis Results

Sample ID	Grain Size Anlaysis	
BH1 SS5 (4.6 – 5.2 m)	Gravel %	47
	Sand %	53
	Fines %	0
BH2 SS6 (5.3 – 5.9 m)	Gravel %	13
	Sand %	47
	Silt %	28
	Clay %	12

7.5 Bedrock

Limestone bedrock was encountered in BH 1 at 6.3 m depth. The presence of bedrock was confirmed by coring 5.0 m into the bedrock. The rock quality designation (RQD) values of the bedrock ranges from 44 to 100% indicating poor to excellent quality. Auger refusal was encountered in BH 2 at 7.6 m depth on inferred bedrock. Detailed classification, test results and parameters of the bedrock are represented in Section 8 of this report.

7.6 Groundwater

Upon completion of augering and sampling, all boreholes were dry. Groundwater levels were measured in the standpipe piezometers installed in both boreholes on Nov. 19 and Nov. 28, 2014 (2 and 11 days following completion of drilling) and the measurements are summarized in Table 7.4.

Groundwater levels will fluctuate seasonally and in response to climatic conditions.

Table 7.4 Summary of Groundwater Level Measurements

Date Measured	Location	Borehole Surface Elevation* (m)	Groundwater Depth (m)	Groundwater Elevation (m)
Nov. 19, 2014	BH 1	99.09	6.58	92.51
	BH 2	99.22	6.59	92.63
Nov. 28, 2014	BH 1	99.09	6.56	92.53
	BH 2	99.22	6.59	92.63

***** The elevation of benchmark is assumed to be 100.00 m.

8. GEOTECHNICAL CHARACTERISTICS

8.1 Rock Characterization

During the coring of the bedrock and prior to laboratory testing of rock core samples, measurements including Total Core Recovery (TCR) and Rock Quality Designation (RQD) were carried out for rock quality classification as indicated in Table 8.1. TCR is defined as the sum of all recovered rock core pieces from a core run expressed as a percent of the total length of the core run. The RQD is defined as percentage of the sum of the core pieces over 100 mm divided by the total length of core run (Deere, 1989). Photographs of the rock cores are shown in Appendix E.

Table 8.1 Relationship between RQD and Rock Mass Measurement (Farmer, 1983)

Quality Classification	RQD (%)	Fracture frequency (per metre)	Mass factor, j
Very poor	0 – 25	>15	0.2
Poor	25 – 50	15 - 8	0.2
Fair	50 – 75	8 – 5	0.2 – 0.5
Good	75 – 90	5 - 1	0.5 – 0.8
Excellent	90 - 100	1	0.8 – 1.0

Selected rock core samples were tested for the point load index ($I_{s(50)}$), i.e. the corrected point load strength of a 50 mm diameter core. The uniaxial compressive strength (UCS or q_u) for the intact rock was tested on the selected rock cores. The strength designations of the rock cores are described based on point load index and uniaxial compressive strength as shown in Table 8.2.

Table 8.2 Classification of Rock with respect to Strength (CFEM, 2006)

Grade*	Term	UCS (MPa)	Point Load Index (MPa)
R6	Extremely strong	>250	>10
R5	Very strong	100 – 250	4 – 10
R4	Strong	50 – 100	2 – 4
R3	Medium strong	25 – 50	1 – 2
R2	Weak	5 – 25	**
R1	Very weak	1 – 5	**
R0	Extremely weak	0.25 – 1	**

* Grade according to ISRM (1981).
** Rocks with a uniaxial compressive strength (UCS) below 25 MPa are likely to yield highly ambiguous results under point load testing.
CFEM – Canadian Foundation Engineering Manual, Fourth Edition, 2006.

Based on the rock characterization for the collected rock core samples, the description of rock mass are described in accordance with the Rock Mass Rating (RMR) as shown in Table 8.3.

Table 8.3 Geomechanics Classification of Rock Mass (Goodman, 1989)

Class	Description of Rock Mass	RMR
I	Very good rock	81 – 100
II	Good rock	61 – 81
III	Fair rock	41 – 60
IV	Poor rock	21 – 40
V	Very poor rock	0 – 20

8.1.1 Total Core Recovery (TCR)

TCRs are 100% for all the rock core samples. The TCRs were measured for every core run and are recorded in the borehole log shown in Appendix C.

8.1.2 Rock Quality Designation (RQD)

The RQD ranges between 44% (poor quality) and 100% (excellent quality). The poor quality rock was located in the upper 0.2 m of the bedrock in BH 1. The RQD of rock samples are presented in the borehole log in Appendix C.

8.1.3 Point Load Tests

Table 8.4 summarizes the point load index ($I_{s(50)}$) of the samples collected from the site. The point load index of a rock core is the value of the point load strength (I_s) that would have been tested by a diameral test with diameter of 50 mm (Goodman, 1989 and ASTM D5731-08) calculated as:

$$I_{S(50)} = FI_S$$

Where: $I_s = \dfrac{P}{D_e^{\,2}}$ (Uncorrected Point Load Strength);

P is the load at rupture;

D_e is equivalent core diameter;

$F = \left(\dfrac{D_e}{50}\right)^{0.45}$ (Size Correction Factor to the Standard 50 mm Core Size);

Table 8.4 Summary of Point Load Test Results

Borehole Number	Sample Depth (m)	Diameter of Test Specimen (mm)	Point Load Index $I_{s(50)}$	Grade (Term)*
BH 1	7.45	47.53	3.36	R4 (Strong)
BH 1	9.03	47.53	3.42	R4 (Strong)
BH 1	10.7	47.53	4.30	R5 (Very strong)

* Extracted from Canadian Foundation Engineering Manual (CFEM), 2006.

8.1.4 Uniaxial Compressive Strength Tests

Uniaxial compressive strength (UCS) of selected rock cores were tested at Davroc Testing Laboratory Inc. Tests were carried out using ASTM D7012-14 (2014).Test results are shown in Appendix D. A summary of UCS test results is indicated in Table 8.5.

Table 8.5 Summary of Uniaxial Compressive Strength Test Results

Borehole Number	Sample Depth (m)	Diameter of Test Specimen (mm)	Uniaxial Compressive Strength (MPa)	Grade (Term)*
BH 1	7.60	47	71.2	R4 (Strong)
	9.24	47	106.1	R5 (Very strong)
	10.83	47	118.9	R5 (Very strong)

*Extracted from CFEM, 2006

8.1.5 Rock Mass Rating

Table 8.6 shows the Rock Mass Rating (RMR) determined using the methods in Goodman (1989).

Table 8.6 Rock Mass Rating

Borehole #	Depth (m)	RMR	Description*
BH 1	6.30-6.55	47	Fair Rock
	6.55-8.51	46	Fair Rock
	8.51-9.70	50	Fair Rock
	9.70-11.25	55	Fair Rock

* Extracted from Goodman (1989)

8.2 Geotechnical Soil Design Parameters

Based on the in-situ and laboratory tests carried out, the following parameters are suggested as design parameters for the soil types encountered in the boreholes (Table 8.7). The internal friction angles of granular materials were estimated from standard penetration tests (SPTs) applying Wolff (1989) which provides an empirical correlation between SPT and internal friction angle. The internal friction angles of cohesive materials were estimated from Atterberg Limits tests applying Kenney (1959) which provides an empirical correlation between plasticity index and internal friction angle. For overconsolidated clay this approach is conservative.

Table 8.7 Geotechnical Soil Design Parameters

Material	Total Unit Weight, γ (kN/m^3)	Undrained Shear Strength, c_u (kPa)	Internal Friction, Φ (°)
Sand Fill	19	-	30-36 (30)
Silty Clay	16-18 (18)	100	26
Till	20	-	28-39 (30)

The numbers in bracket should be used for preliminary design purposes.

9. DISCUSSION AND RECOMMENDATIONS

The general site stratigraphy consists of surficial pavement structure overlying fill, native clay and till mantling limestone bedrock.

Unless noted otherwise, foundation design parameters are given for static, vertically and concentrically loaded foundations in compression. Dynamic, lateral, eccentric and uplift design parameters can be provided upon request if applicable.

Note that the discussions presented herein are intended for the sole use of the designers/planners of the project and are subject to the limitations in Appendix A. Further geotechnical assessment and analyses will be required once details regarding the proposed building and site development are available.

All recommendations presented in this report are based on the assumption that an adequate level of construction monitoring of excavations and installation will be provided at the time of construction. An adequate level of construction monitoring is considered to be full time monitoring, inspection and compaction testing.

9.1 Site Grade Raise Restriction

The site is underlain by a compressible clay that is susceptible to consolidation settlement that may result in the settlement of building structures and infrastructure. Therefore, a grade raise restriction will be required for this site. It is recommended the grades at the site not be raised from the current level.

If detailed design plans include a site grade raise, DST must be contacted to conduct a settlement analysis to determine if settlements will be within tolerable limits due to the load increase from the proposed site grade raise.

9.2 Foundations

Based on a review of the borehole information and the understanding that the proposed building will be nine (9) storeys with one (1) basement level (assumed at 3.0 m depth), the native clay and upper portion of the till are not considered capable of supporting the anticipated loads from a building of this size on shallow foundations (footings and raft foundation types). The settlement of the footings will likely exceed the normally tolerable limits of 25 mm total and 19 mm differential. Therefore, the building will have to be supported by deep foundations consisting of either piles or caissons founded within the limestone bedrock. Details regarding pile and caisson foundations are discussed in the following sections of this report.

Driven Piles

The driven piles can comprise of steel H or pipe piles. In the Ottawa area, H piles are most common. The piles driven to refusal in the bedrock are suitable for high capacities; however unknowns exist at each pile tip including the exact contact area, the rock quality and the depth of penetration into the bedrock. Therefore, the capacity of such a pile using theoretical or semi-empirical methods cannot be made with certainty. Consequently, the capacity should be determined based on driving observations, local experience, dynamic and load testing.

Common sizes of steel piles driven to refusal into the bedrock can be designed to their full structural capacity (cross sectional area of the pile multiplied by the allowable stress of the pile material).

Refusal is tentatively defined as 20 blows for 25 mm for 75 mm consecutive millimetres, subject to the discretion of the geotechnical engineer. Approval of satisfactory refusal should also involve consideration of the driving performance and pile tip elevation with respect to the borehole data and other adjacent piles.

The depth to bedrock at the site is 6.0 m at BH1 and inferred at 7.6 m in BH2 and may vary unpredictably between and away from the two (2) boreholes. Penetration of piles into the bedrock to depths of 1.0 to 2.0 m, depending on rock quality, would not be unusual. Penetration will likely be deeper for H piles versus pipe piles.

The till soil may contain cobbles and boulders. It is therefore recommended the piles be equipped with a driving shoe.

The single pile capacities for various pile types were analysed using geotechnical parameters discussed in Section 8 of this report. Three (3) types of commonly used H pile sections were used in the analysis. The consolidation of the clay layer was not considered in the pile analysis. The allowable yield stress of 80 MPa for a driven steel pile was assumed for the analysis. Resistance factor of 0.4 was used in the ultimate limit state (ULS) design of piles as recommended by 2006 CFEM. A summary of the capacity of the three H pile sizes is shown in Table 9.1.

Table 9.1 Factored Axial Resistance and Axial Load Capacity of H-Piles

Pile Type	**HP310 @110 kg/m**	**HP310 @125 kg/m**	**HP310 @132 kg/m**
Steel Area, cm^2	141	159	167
Ultimate Limit State (ULS) Factored Axial Resistance, kN	450	500	530

Total settlement of piles designed for the above recommended factored geotechnical resistance at ULS is expected to be less than 10 mm.

ULS and SLS conditions for pile groups should be assessed, if piles are selected as the preferred foundation option.

The clay soil at pile locations has the potential to move downward relative to the pile resulting in downdrag load on the pile. The downdrag load will have to be considered as part of the capacity of the piles in final design. The downdrag force for the pile foundation option as a result of small future settlement of the surrounding ground (clay profile) is estimated using the negative skin friction. The downdrag increases the loads in the pile and thus has to be accounted for when evaluating the structural ultimate limit state of the pile.

Bored Cast-In-Place Concrete Piles (Caissons)

Bored cast-in-place concrete piles (caissons) founded on the bedrock may be considered for the proposed building. Caissons socketed into the bedrock are also feasible, although the variable depths to bedrock and quality of the fractured upper bedrock zone (0.5 to 1.5 m) provide for a high degree of uncertainty (and therefore cost) for caisson contractors. Table 9.2 shows the factored geotechnical resistance values at ULS for caissons founded below fractured zones of the bedrock and designed in end bearing.

Table 9.2 Factored Geotechnical Resistance at ULS for 1.0 m Diameter Caisson

Pile Type	**1.0 m Diameter**
End Bearing Area, cm^2	7854
Ultimate Limit State (ULS) Axial Load Pile Capacity, kN*	1000

*This assumes reinforced concrete pile with $F_{c'}$ = 30 MPa

The settlement of the end bearing caissons founded on competent bedrock is considered to be negligible.

The installation of the caissons will require the use of at least one liner driven to the limestone bedrock. The caissons will require de-watering since the measured groundwater level is anticipated to be above and close to the bedrock surface.

Additional Comments

Additional comments regarding piles and caissons as follows:

- The contractor should be made aware that the till may contain cobbles and boulders which may cause difficulties in advancing the piles and caissons.

- The installation of piles or caissons should be undertaken by contractors specializing in this field with local experience.

9.3 Slab-On-Grade

Slab-on-grade construction for the basement floor at an assumed depth of 3.0 m below existing grade is considered feasible provided certain precautions are undertaken. The subgrade for the basement floor slab is anticipated to be native clay. It is recommended the floor slab be set on a granular pad consisting of 200 mm thick bed of well packed clear stone (19 mm) or on an Ontario Provincial Standard Specification (OPSS) Granular A layer compacted to 98 % standard Proctor maximum dry density (SPMDD) underlain by a minimum 300 mm thick OPSS Granular B Type II granular pad placed on the approved silty clay and compacted to 98 % SPMDD. A geotextile should be placed between the clay and Granular B Type II.

The basement slab should be structurally independent from walls and columns, which are supported by the foundations. This is to reduce any structural distress that may occur as a result of differential movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement of the walls can occur freely.

The subgrade beneath the slab-on-grade should be protected at all times from rain, snow, freezing temperatures, excessive drying and the ingress of water. This applies during and after the construction period.
Some relative movement between the slab-on-grade floor and adjacent walls or foundations and differential movement within the slab should be anticipated. Generally, if the recommendations outlined in this report are followed, these movements are estimated to be less than 10 mm.

9.4 Lateral Earth Pressure against Subsurface Walls

Subsurface walls (basement walls) should be backfilled with free draining material such as OPSS Granular B Type II material compacted to 95 % SPMDD. Caution should be exercised when compacting near foundation walls. The walls should have a perimeter drainage system that is directed to a positive outlet.

For the above construction, the lateral static earth pressure against the subsurface walls may be calculated using Equation 9.4.1.

$P = K_0\ (\gamma H + q)$ Equation 9.4.1

P	=	lateral earth pressure acting on subsurface wall (kPa)
K_0	=	lateral earth pressure coefficient for at rest condition for Granular B Type II (OPSS) backfill material = 0.5
γ	=	bulk unit weight of Granular B Type II = 22 kN/m^3
H	=	depth below final grade (m)
q	=	surcharge pressure at ground level (kPa)

The lateral force due to seismic loading may be computed from Equation 9.4.2 (Wood, 1973).

$\Delta P_E = (a_h/g)F_p\ \gamma H^2 = 0.23\ \gamma\ H^2$ Equation 9.4.2

ΔP_E = resultant force due to seismic loading (kN/m)
a_h = pseudo static horizontal accelerlation; $a_h = 0.21g$ (Ottawa area)
F_p = dynamic thrust factor = 1.1
γ = bulk unit weight of Granular B Type II = 22 kN/m^3
H = height of backfill behind wall (m)

The resultant force should be assumed to act 0.6H from bottom of the wall.

Exterior backfill against foundation walls should be capped with an impervious layer.

Final site grades should be sloped to direct water away from the proposed building. Minimum landscape gradients of 2% are recommended to reduce the risk of runoff ponding in localized areas.

9.5 Seismic Site Classification

Based on the calculated average N value of the top 30 m of the soil and bedrock, the site has been classified as Class D for seismic site response in accordance with Table 4.1.8.4.A of the Ontario Building Code, 2012.

Consideration may be given to conducting shear wave velocity measurement survey of the site to determine if the site class can be improved from Class D.

The till may potentially liquefy during a seismic event. This potential will have to be evaluated further by detailed analysis.

9.6 Frost Protection

Based on the Ministry of Environment published data, for an 85% probability level, the design freezing index for Ottawa has been estimated as 1,050° C-Days (1,922°F-Days). The design depth of frost penetration for an area that has been kept clear of snow cover should be taken as 1.8 m for well graded sand with more than 8 % fine content soil cover. Frost penetration depth will vary with type of soil cover.

For pile caps and unheated structures, the frost cover should be 1.8 m. Where earth cover is less than the required, thermal equivalent rigid insulation should be provided. The rigid insulation should be installed in accordance with the manufacturer's directions.

9.7 Excavations and Dewatering

Excavations

Excavations for the proposed building are expected to extend into the clay and will be above the measured groundwater level.

Excavation of the soils may be undertaken with large mechanical equipment capable of removing possible debris within the fill.

Excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA). The soils are considered to be Type 3 soil and as per OHSA, the excavation side slopes must be cut back at 1H: 1V. Local flattening of the sideslopes may be required in zones of persistent seepage. If it is not possible to cut back the excavation sideslopes due to space restrictions on site, it is recommended the excavations be undertaken within the confines of an engineered support system designed and installed in accordance with OHSA. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to the depth of excavation, unless an excavation support system is designed and incorporated to accommodate such surcharge.
The soil parameters provided in Table 8.7 may be used for the design of the shoring system.

Information regarding the adjacent buildings (such as the lowest floor slab and foundation type), infrastructure (invert levels) and any other nearby settlement sensitive structures will be required to assist in establishing the most appropriate type of shoring system.

A condition survey of nearby buildings, infrastructure (such as roadways, underground services) and any other settlement sensitive structures should be undertaken prior to excavation and construction of the building.

De-Watering

The excavations are anticipated to be above the measured groundwater level. The de-watering of excavations may be undertaken using conventional sump pump techniques. High capacity pumps may be required in areas of persistent seepage.

It is noted that dewatering effort will depend on a number of factors, including excavation depth, season and weather conditions and the length of time the excavation is left open.

It should be realized that dewatering within compressible deposits, such as the silty clay can cause ground settlement that extends laterally beyond the immediate area of dewatering. It is recommended the likely impact of dewatering be assessed in detail and use methods which will control de-watering impact on nearby existing structures and infrastructure to tolerable limits. A preconstruction survey documenting the conditions of nearby settlement-sensitive facilities/infrastructure be completed prior to start of construction.

9.8 Corrosiveness of Sub-Surface Soils

Selected soil samples were submitted to Paracel Laboratories Ltd. in Ottawa for chemical analyses (pH, sulphate, resistivity and chloride) to assess the potential for corrosion and sulphate attack on buried structures.

The results are presented below in Table 9.3 and a copy of the Laboratory Certificate of Analysis is provided in Appendix F.

Table 9.3 Chemical Test Results

Sample ID	Sulphate (mg/kg)	Chloride (mg/kg)	pH	Resistivity (ohm - cm)
BH1 SS3	81	51	7.39	8300
BH2SS7	979	70	7.64	37200

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) standards and are given in Table 9.4 below.

Table 9.4 Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposer	Degree of Exposer	Water soluble Sulphate in soil sample (%)	Cementing Material to be used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 – 2.0	HS or HSb
S-3	Moderate	0.10 – 0.20	MS, MSb, LH, HS, or HSb

*Information from Table 3 of CSA Standards A23.1-04

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of 81 mg/kg (BH1 SS3) and 979 mg/kg (BH2 SS7) in soil, as shown in Table 9.3. The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possess a “negligible” risk for sulphate attack on concrete material. Accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH values for the soil samples were reported to be 7.39 and 7.64, indicating a durable condition against corrosion. These results were evaluated using Table 2 of Building Research Establishment

(BRE) Digest 363 (July 1991). The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids. Soil resistivity was found to be 8300 ohm.cm (BH1 SS3) and 37200 ohm.cm (BH2 SS7). The test results shown in Table 18.1 may assist in determining corrosion protection and coatings for buried steel structures by a corrosion specialist.

9.9 Planting of Trees

The clay in the Ottawa area is susceptible to shrinking on drying as water is removed by the root system from trees. This process cannot be reversed. The shrinking of the clay can result in settlement and cracking of nearby structures, infrastructure and other settlement sensitive features.

Therefore, if plans call for the planting of trees at this site, an arborist should be contacted to provide comments and recommendations regarding the most appropriate types of trees to plant and location (distance) from settlement sensitive features.

10. CLOSURE

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix A and forms an integral part of this report.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Sincerely,

For DST CONSULTING ENGINEERS INC.

Prepared By:

LICENSED PROFESSIONAL ENGINEER
JAN. 12, 2015
S.M. POTYONDY
37232568
PROVINCE OF ONTARIO

for
Cheng (Gary) Zhao, E.I.T.
Junior Project Manager

Susan Potyondy, P.Eng.
Senior Geotechnical Engineer

REFERENCES

ASTM D5731-08, 2008. ASTM: Standard Test Methods for Determination of the Point Load Strength of Rock and Application to Rock Strength Classifications, ASTM International.

ASTM D7012-14, 2014. ASTM: Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures, ASTM International.

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Deere, D.U., 1989. Rock Quality Destignation (RQD) after 20 years. U.S. Army Corps Engrs Contract Report GL-89-1. Viskurg, MS: Waterways Experimental Station.

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Kenney, T. C., (1959), Discussion, Proc. ASCE, Vol. 85 (3), pp. 67-69.

APPENDIX A

LIMITATIONS OF REPORT

LIMITATIONS OF REPORT
GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testhole may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, 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 details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid. Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

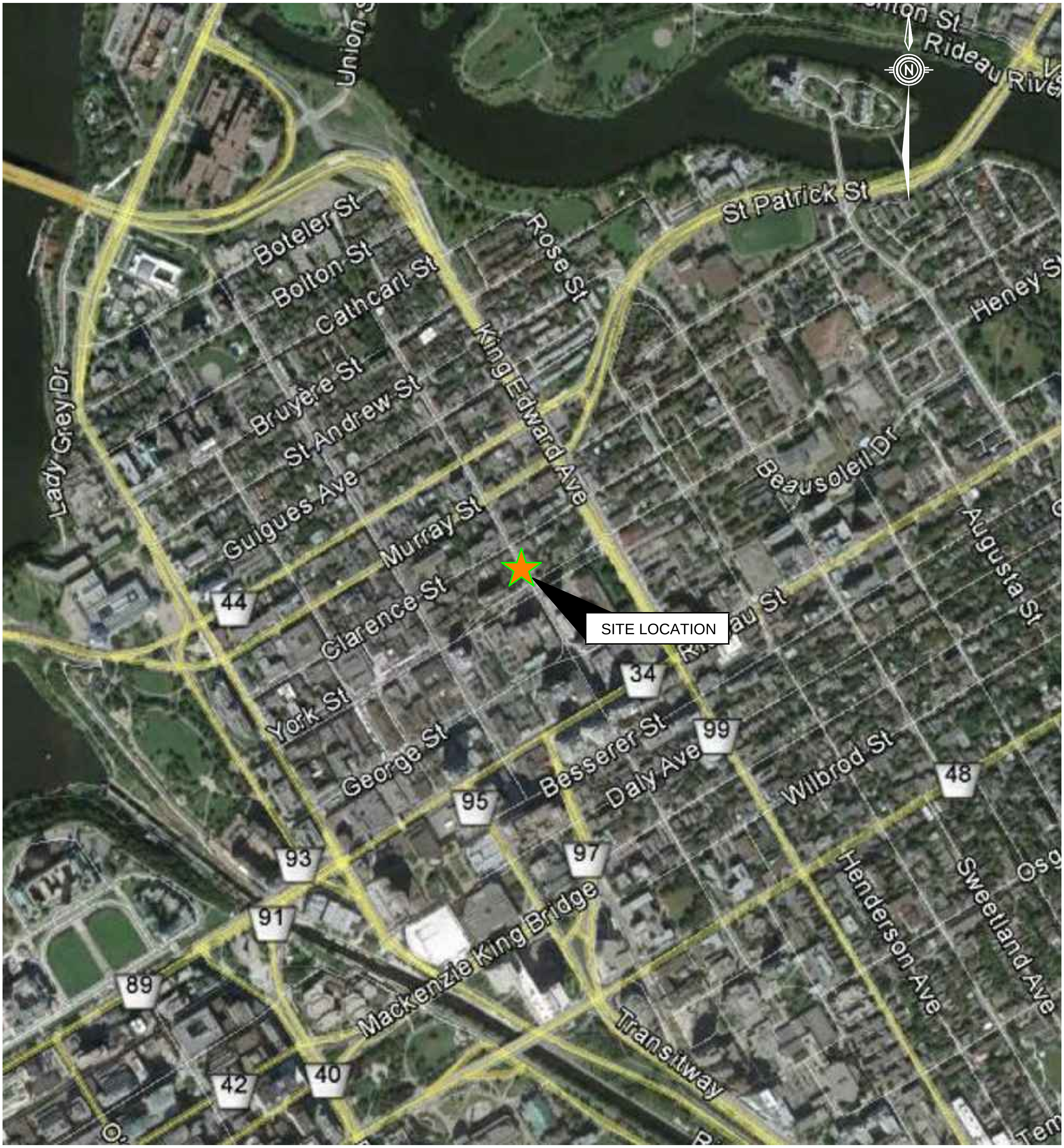
The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil 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 conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the Client.

APPENDIX B

SITE LOCATION MAP

BOREHOLE LOCATION PLAN



SOURCES:
1. GOOGLE EARTH © 2014 GOOGLE

NOTE:
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH THE ASSOCIATED TECHNICAL REPORT.

APPROXIMATE SCALE
0 100 200 400m

REV	DATE	ISSUE	APPROVAL
0	11/12/14	ORIGINAL	S.P.

DRAWN BY: R.P.	DATE: December 2014
PROJECT MANAGER: S.P.	PROJECT NO.: OE-OT-019637
FIGURE No.: FIGURE 1	

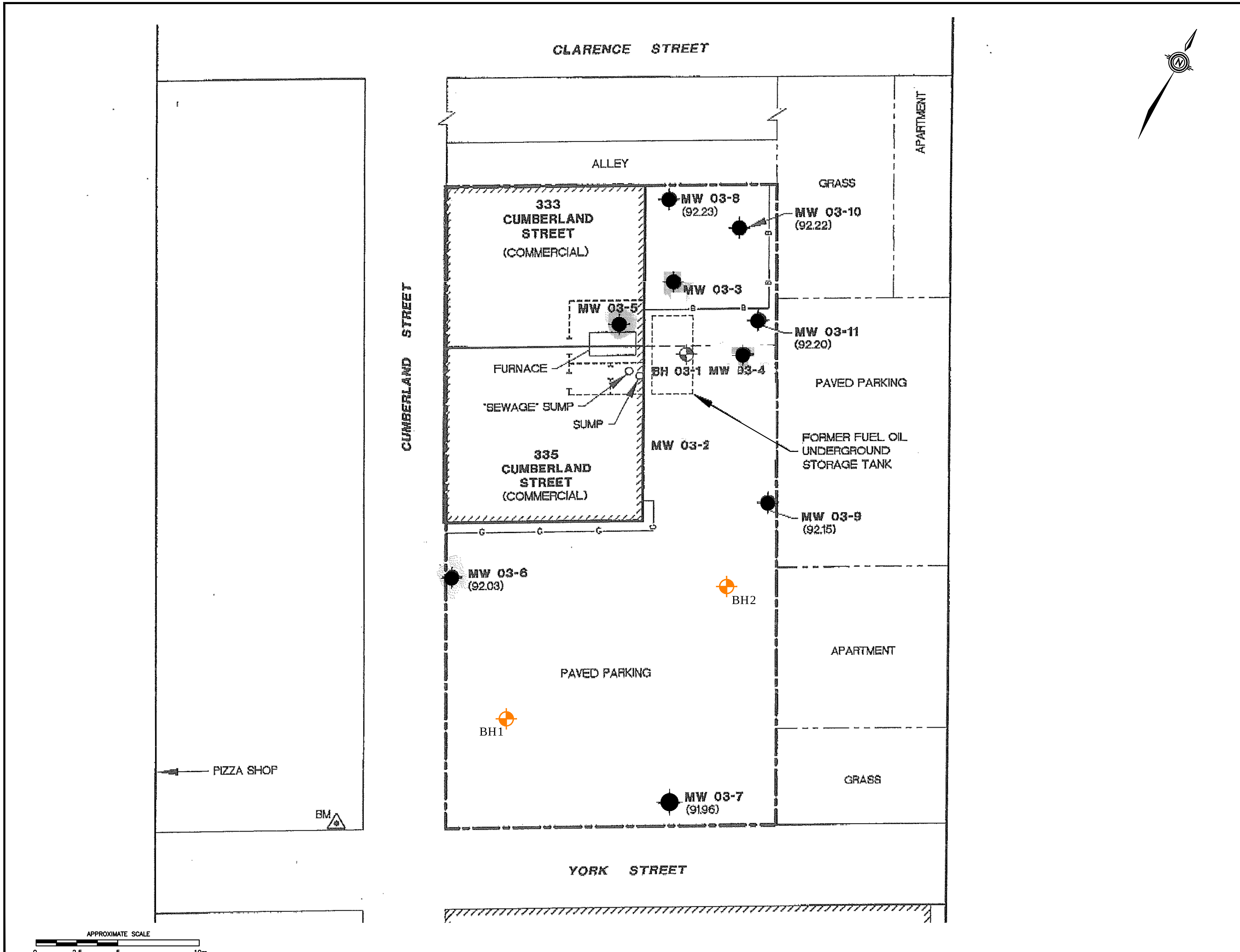
PROJECT TITLE

GEOTECHNICAL INVESTIGATION,
339 CUMBERLAND STREET, OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION MAP

DST consulting engineers
2150 THURSTON DRIVE, SUITE 203
OTTAWA, ONTARIO, K1G 5T9
TEL (613) 748-1415 FAX (613) 748-1356
www.dstgroup.com



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2150 THURSTON DRIVE, SUITE 203
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TEL (613) 748-1415 FAX (613) 748-1356
www.dstgroup.com

LEGEND:

- BH2 GEOTECHNICAL BOREHOLE (DST)
- BOREHOLE BY OTHERS (JACQUES WHITFORD ENVIRONMENT LIMITED, PHASE II ESA, DECEMBER, 2003)
- BOREHOLE BY OTHERS (JACQUES WHITFORD ENVIRONMENT LIMITED, PHASE II ESA, DECEMBER, 2003)
- BM BENCHMARK (TOP OF FIRE HYDRANT, LOCATION IS APPROXIMATE) ASSUMED EL=100.00 m

NOTES:
1. THESE DRAWINGS SHALL BE READ IN CONJUNCTION WITH THE ASSOCIATED TECHNICAL REPORT.

REV	DATE	ISSUE	APPROVAL
0	16/12/14	ORIGINAL	S.P.

PROJECT TITLE

GEOTECHNICAL INVESTIGATION,
339 CUMBERLAND STREET, OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION PLAN

DESIGNED BY	C.Z.	SCALE	As Shown
DRAWN BY	R.P.	DATE	December 2014
PROJECT MANAGER	S.P.	PROJECT NO.:	OE-OT-019637
FIGURE No.:	FIGURE 2		

APPENDIX C

BOREHOLE LOGS

LOG OF BOREHOLE BH 1

DST REF. No.: **OE-OT-019637**
CLIENT: **Eco Corner Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **339 Cumberland, Ottawa, Ontario**
SURFACE ELEVATION: 99.09 metres

Drilling Data
METHOD: **Hollow Stem Auger/Rock Coring**
DIAMETER: **80 mm**
DATE: **17 November 2014**
COORDINATES: m N, m E

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE (W_p W W_l; 20 40 60 80)	Symbol	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	VANE (kPa) ✕ 20 40 60 80 / SPT (N) □ DCPT ◆ Blows/0.3m 20 40 60 80	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	99				ASPHALT - 50 mm.	AS1				
					FILL - Clay, silt and sand mixture, some gravel and grey silt, dark brown, to black, loose.	SS1		4		
1.0	98				- Rootlets below 0.8 m.	SS2		5		
					CLAY - Silty, trace rootles, some oxidized stains, brown, very stiff.	SS3		6		
2.0	97									>105 kPa
3.0	96					ST1				>105 kPa
					TILL - Sand and gravel, silty some gravel, light grey, loose to compact.					
4.0	95					SS4		4		
										47 53 0 0
5.0	94					SS5		5		
						SS6		14		
6.0	93				BEDROCK - Limestone.	SS7		50+		
					RC1 - 0.25 m, TCR - 100 %, SCR - 60 %, RQD - 44%	RC1				
7.0	92									
					RC2 - 1.96 m, TCR - 100 %, SCR - 52 %, RQD - 52%	RC2				

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Email: ottawa@dstgroup.com
Web: www.dstgroup.com

SAMPLE TYPE LEGEND

Auger Sample, Split Spoon Sample, Bulk Sample, Rock Core, Hiller Peat Sampler, 70mm Thin Wall Tube, Bentonite, Sand

ENCLOSURE 1

LOG OF BOREHOLE BH 1

DST REF. No.: **OE-OT-019637**
CLIENT: **Eco Corner Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **339 Cumberland, Ottawa, Ontario**
SURFACE ELEVATION: 99.09 metres

Drilling Data
METHOD: **Hollow Stem Auger/Rock Coring**
DIAMETER: **80 mm**
DATE: **17 November 2014**
COORDINATES: m N, m E

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE (W_p, W, W_l; 20 40 60 80)	Symbol	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	VANE (kPa) × 20 40 60 80; SPT (N) □ Blows/0.3m; DCPT ◆ 20 40 60 80	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	91				BEDROCK - Limestone.					
9.0	90				RC3 - 1.19 m, TCR - 100 %, SCR - 92%, RQD - 69%	RC3				
10.0	89				RC4 - 1.55 m, TCR - 100 %, SCR - 100%, RQD - 100%	RC4				
11.0	88									
					End of Borehole at 11.25 m.					
12.0	87									
13.0	86									
14.0	85									
15.0	84									

BOREHOLE (STANDARD) - OTTAWA OE-OT-019637 - GS.GPJ DST_MIN.GDT 16/12/14

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Email: ottawa@dstgroup.com
Web: www.dstgroup.com

SAMPLE TYPE LEGEND

Auger Sample | Rock Core | Bentonite
Split Spoon Sample | Hiller Peat Sampler | Sand
Bulk Sample | 70mm Thin Wall Tube

ENCLOSURE 2

LOG OF BOREHOLE BH 2

DST REF. No.: **OE-OT-019637**
CLIENT: **Eco Corner Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **339 Cumberland, Ottawa, Ontario**
SURFACE ELEVATION: 99.22 metres

Drilling Data
METHOD: **Hollow Stem Auger/Rock Coring**
DIAMETER: **80 mm**
DATE: **17 November 2014**
COORDINATES: m N, m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION
	99.22	ASPHALT - 50 mm.
		FILL - Sand, gravelly, some silt and clay, rootlets, brick pieces, dark grey, loose to compact.
		CLAY - Silty, trace rootlets, some sand, oxidized stains, brown, very stiff.
		TILL - Silty sand, some clay and gravel, light grey cobbles, loose to compact.
7.6		End of Borehole at 7.6 m. Auger Refusal.

SAMPLE #	SAMPLE TYPE	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	Split Spoon Sample	11	
SS2	Split Spoon Sample	6	
SS3	Split Spoon Sample	5	0 19 42 39
			VANE: >105 kPa
SS4	Split Spoon Sample	0	
			VANE: >105 kPa
ST1	70mm Thin Wall Tube		
			VANE: >105 kPa
SS5	Split Spoon Sample	6	
SS6	Split Spoon Sample	15	13 47 28 12
SS7	Split Spoon Sample	28	
SS8	Split Spoon Sample	17	

% MOISTURE: Wp, W, Wl (20, 40, 60, 80)
VANE (kPa) × (20, 40, 60, 80)
SPT (N) □ Blows/0.3m; DCPT ◆ (20, 40, 60, 80)

BOREHOLE (STANDARD) - OTTAWA OE-OT-019637 - GS.GPJ DST_MIN.GDT 16/12/14

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FX: (613)748-1356
Email: ottawa@dstgroup.com
Web: www.dstgroup.com

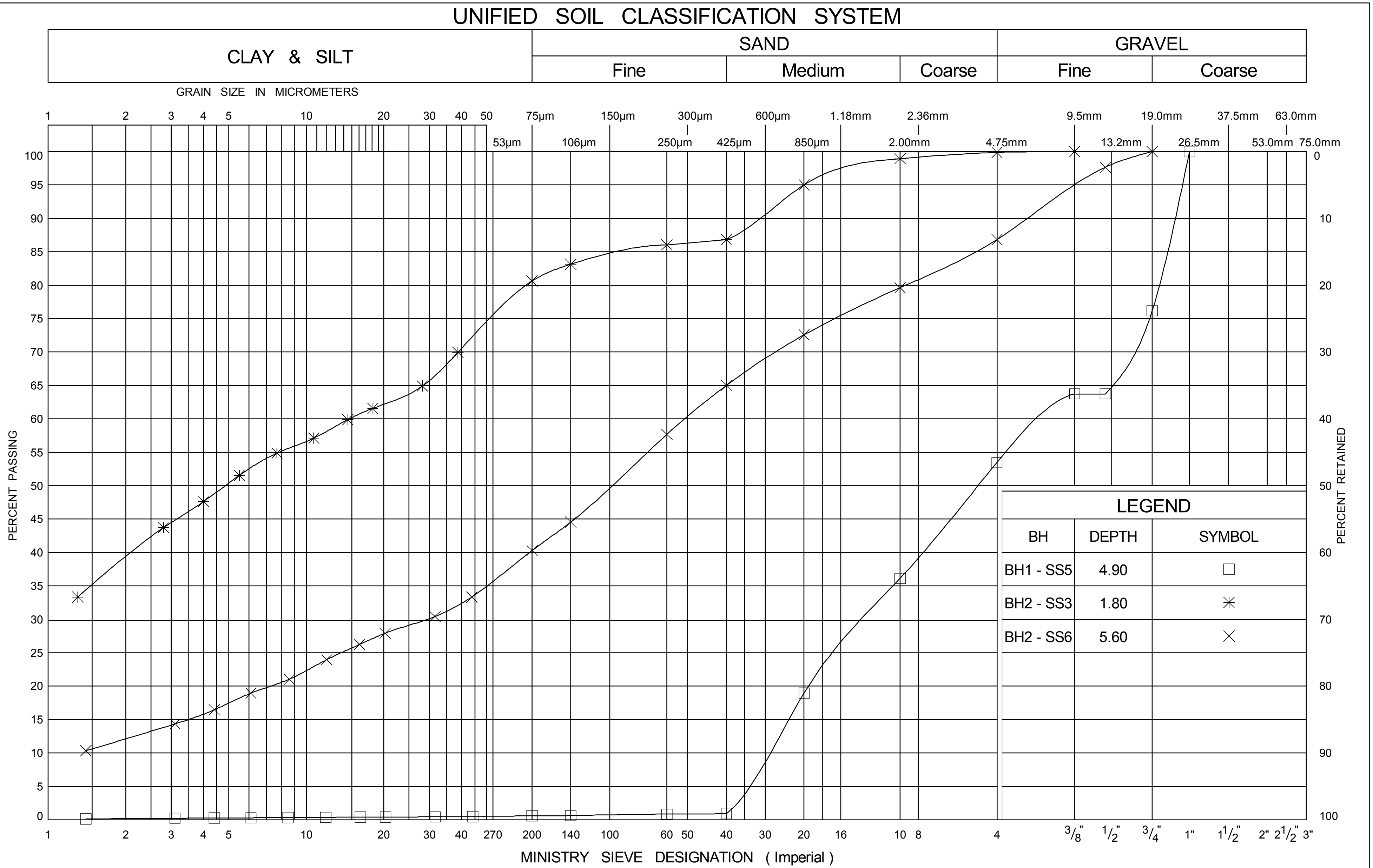
SAMPLE TYPE LEGEND

Auger Sample; Split Spoon Sample; Bulk Sample; Rock Core; Hiller Peat Sampler; 70mm Thin Wall Tube; Bentonite; Sand

ENCLOSURE 3

APPENDIX D

LABORATORY TEST RESULTS



LEGEND

BH	DEPTH	SYMBOL
BH1 - SS5	4.90	□
BH2 - SS3	1.80	✳
BH2 - SS6	5.60	×

consulting engineers

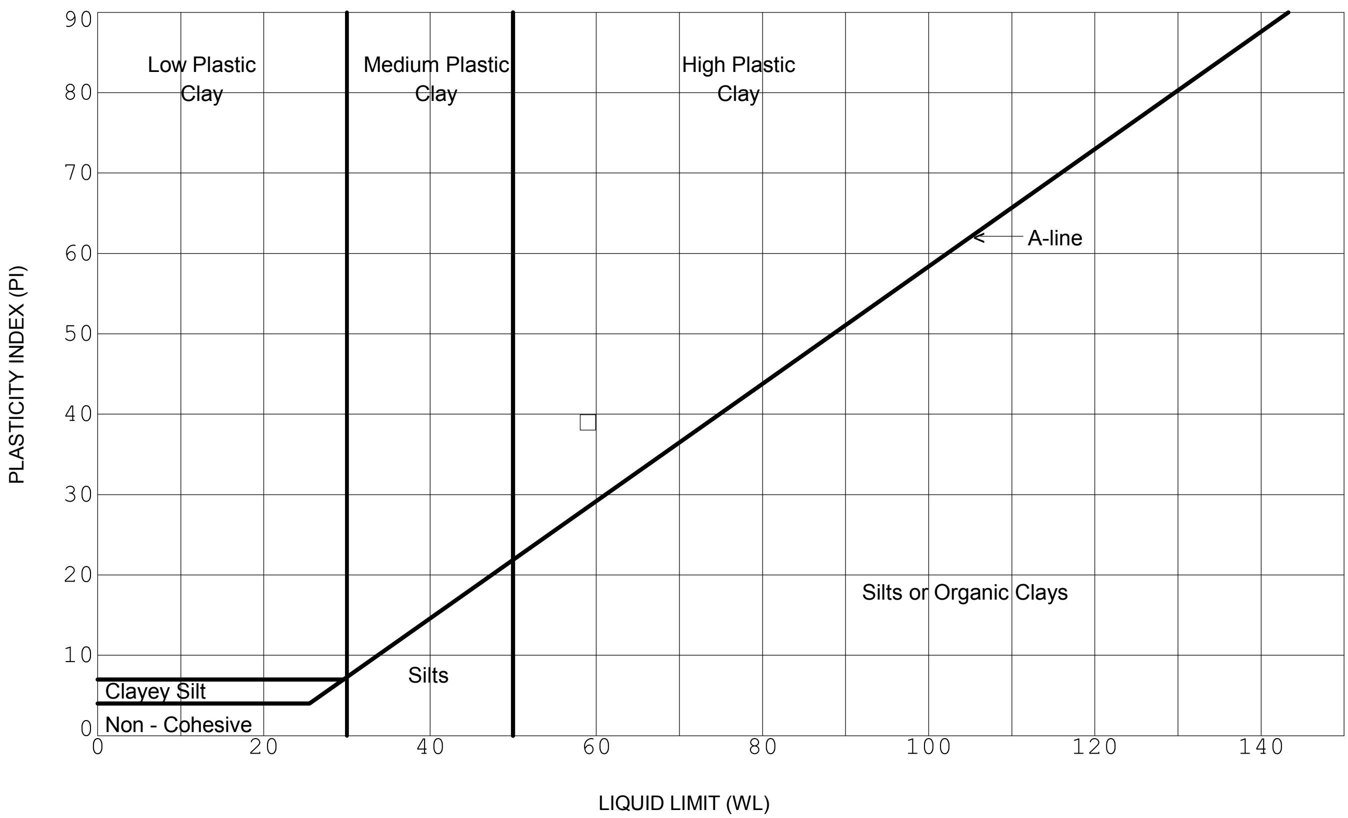
GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION

ENCLOSURE 1

OE-OT-019637

ONTARIO MOT GRAIN SIZE OE-OT-019637 - SIEVES AND GRAINSIZE.GPJ DST_MIN.GDT 12/11/14

ATTERBERG LIMIT TEST RESULTS



LEGEND:

	W_L	W_P	PI	W
☐ BOREHOLE BH 2 DEPTH 2.70	59	20	39	51

December 2014

Reference No.: OE-OT-019637

Geotechnical Investigation -

DST CONSULTING ENGINEERS INC.

Enclosure 1

ATT-BH-FIX OE-OT-019637 - GS.GPJ DST_MIN.GDT 15/12/14

DAVROC
TESTING LABORATORIES INC.

CONSULTING ENGINEERS
Materials Testing
and Inspection

File: L14-0189RC

November 28, 2014

DST Consulting Engineers Ltd.
203-2150 Thurston Drive
Ottawa, Ontario
K1G 5T9

Attn.: Mr. Cheng (Gary) Zhao
Junior Project Manager

Dear Sir;

Unconfined Compressive Strength Testing
Rock Core Samples
DST Project No.: OEOT019637

Further to receipt of three (3) 47 mm nominal diameter size rock core samples in our laboratory on November 26, 2014, Davroc Testing Laboratories Inc. is pleased to report the results of our tests.

As instructed, the core sample ends were sawn and then were ground down to the required tolerance, and the prepared core samples were tested for compressive strength in accordance with ASTM D 7012 Standard Test Method for "Compressive Strength and Elastic Modula of Intact Rock Core Specimens under Varying States of Stress and Temperatures".

Test Results

The results of our tests are summarized on the following Table No. 1, and detailed test data is shown on the attached Rock Core Test Certificates.

Table No. 1
Rock Core Unconfined Compressive Strength Test Result Summary

Davroc Sample No.	DST Sample No.	Borehole No.	Depth (m)	Unconfined Compressive Strength (MPa)
3256-1	UCS-1	BH 1	7.60	71.2
3256-2	UCS-2	BH 1	9.24	106.1
3256-3	UCS-3	BH 1	10.83	118.9

We trust the above is satisfactory. Should you require any further information, please do not hesitate to contact the undersigned.

Yours very truly,
Davroc Testing Laboratories Inc.

Kateryna Fiyalko, C.E.T.
Concrete Laboratory Supervisor

Sal Fasullo, C.E.T.
Vice President

SF/kf
14-0189-2

DAVROC

ROCK CORE TEST REPORT

File No.: L14-0189RC
Davroc Sample No.: 3256

Project No.: OEOT019637
Project Name: Not Known

DST Core No.	UCS-1	UCS-2	UCS-3
Location	BH 1		
Depth (m)	7.60	9.24	10.83
Date Cored	Not Known	Not Known	Not Known
Date Tested	Nov. 27, 2014	Nov. 27, 2014	Nov. 27, 2014
Height (mm)	96.0	110.0	112.5
Average Diameter (mm)	47.2	47.5	47.5
L/D Ratio	2.03	2.32	2.37
Density (kg/m^3)	-	-	-
Compressive Strength (MPa)	71.2	106.1	118.9
Mode of Failure	*	*	*
Direction of Loading	Not Known	Not Known	Not Known
Moisture Condition at Time of Test	As Received	As Received	As Received

Remarks: Diagonal and vertical shear breaks alone the vane in the rock bedding.

Date: November 28, 2014

Signed:____________________
Sal Fasullo, C.E.T.

APPENDIX E

ROCK CORE PHOTOGRAPH

Photo of Rock Cores in Borehole 1

APPENDIX F

LABORATORY CERTIFICATES OF ANALYSIS

TRUSTED.
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www.paracellabs.com

Certificate of Analysis

DST Consulting Engineers Inc. (Ottawa)
203-2150 Thurston Dr.
Ottawa, ON K1G 5T9
Attn: Cheng Zhao

Phone: (613) 748-1415
Fax: (613) 748-1356

Client PO:
Project: OE OT 019637
Custody: 20667

Report Date: 28-Nov-2014
Order Date: 25-Nov-2014
Order #: 1448074

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1448074-01	BH1 SS3
1448074-02	BH2 SS7

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 28-Nov-2014
Order Date:25-Nov-2014

Client: **DST Consulting Engineers Inc. (Ottawa)**
Client PO:
Project Description: OE OT 019637

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	26-Nov-14	26-Nov-14
pH	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	27-Nov-14	27-Nov-14
Resistivity	EPA 120.1 - probe, water extraction	26-Nov-14	26-Nov-14
Solids, %	Gravimetric, calculation	26-Nov-14	26-Nov-14

Certificate of Analysis

Report Date: 28-Nov-2014
Order Date:25-Nov-2014

Client: **DST Consulting Engineers Inc. (Ottawa)**
Client PO:
Project Description: OE OT 019637

	Client ID:	BH1 SS3	BH2 SS7	-	-
	Sample Date:	17-Nov-14	17-Nov-14	-	-
	Sample ID:	1448074-01	1448074-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.4	91.9	-	-
General Inorganics					
pH	0.05 pH Units	7.39	7.64	-	-
Resistivity	0.10 Ohm.m	8.30	37.2	-	-
Anions					
Chloride	5 ug/g dry	51	70	-	-
Sulphate	5 ug/g dry	81	979	-	-

Certificate of Analysis

Report Date: 28-Nov-2014
Order Date:25-Nov-2014

Client: **DST Consulting Engineers Inc. (Ottawa)**
Client PO:
Project Description: OE OT 019637

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis

Report Date: 28-Nov-2014
Order Date:25-Nov-2014

Client: **DST Consulting Engineers Inc. (Ottawa)**
Client PO:
Project Description: OE OT 019637

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	21.6	5	ug/g dry	16.4			27.4	20	QR-01
Sulphate	1580	5	ug/g dry	1530			3.3	20	
General Inorganics									
pH	7.26	0.05	pH Units	7.39			1.8	10	
Resistivity	8.82	0.10	Ohm.m	8.30			6.0	20	
Physical Characteristics									
% Solids	99.6	0.1	% by Wt.	99.7			0.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	10.3		mg/L	1.6	86.6	78-113			
Sulphate	9.95		mg/L	ND	99.5	78-111			

Qualifier Notes:

QC Qualifiers :

QR-01 : Duplicate RPD is high, however, the sample result is less than 10x the MDL.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

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(Lab Use Only)
Nº 20667

Page 1 of 1

Client Name: DST Consulting Engineers Inc.	Project Reference: OEOT 019657
Contact Name: Cheng Zhao	Quote #
Address: 203, 2150 Thurston Dr. Ottawa, ON, K1G 5T9	PO #
Telephone: 613-748-1415 ext. 238	Email Address: czhao@dstgroup.com

TAT: [✓] Regular [ ] 3 Day [ ] 2 Day [ ] 1 Day

Date Required:

Criteria: [ ] O. Reg. 153/04 (As Amended) Table __ [ ] RSC Filing [ ] O. Reg. 558/00 [ ] PWQO [ ] CCME [ ] SUB (Storm) [ ] SUB (Sanitary) Municipality: ______ [ ] Other: ______

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm Sanitary Sewer) P (Paint) A (Air) O (Other)

Paracel Order Number: 1448074

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	PH	Sulphate	Chloride	Soil Resistivity							
1	BH1 SS3	S			11/17/2014		✓	✓	✓	✓				1X250M			
2	BH2 SS7	S			11/17/2014		✓	✓	✓	✓				"			
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

Comments:

Method of Delivery: Walkin

Relinquished By (Sign): [signature]	Received by Driver/Depot:	Received at Lab: [signature]	Verified By: [signature]
Relinquished By (Print): Cheng Zhao	Date/Time:	Date/Time: Nov 25, 14 11	Date/Time: Nov 25/14 12:11
Date/Time: 11/25/2014 11:47 am	Temperature: ___ °C	Temperature: 11.7 °C	pH Verified [ ] By: N/A