GEOTECHNICAL INVESTIGATION
120 HEARST WAY
OTTAWA, ONTARIO

Prepared for:

BROCCOLINI CONSTRUCTION

By:

SPL CONSULTANTS LIMITED

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Geotechnical Environmental Materials Hydrogeology

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1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by Broccolini Construction (Broccolini) to conduct a geotechnical investigation at 120 Hearst Way in Ottawa, ON. The terms of reference for the project are as outlined in our proposal number P-12.12.027 dated December 20th, 2012 as well as subsequent project correspondence.

This report presents the results of the investigation and provides geotechnical recommendations related to the design of the proposed development. This report does not contain recommendations related to environmental issues which are outside the scope of this study.

2. PROJECT AND SITE DESCRIPTION

The site is located at 120 Hearst Way, in Ottawa, ON (see Figure 1). The site is bounded on the north by the Highway 417 and to the south by Hearst Way, beyond which is a residential development. To the east and west of the site are similar undeveloped lots.

The topography of the site is generally flat to gently sloping with the exception of a depression on the south side of the lot which is approximately 0.5 m to 1 m lower than the rest of the site. A rock outcrop is visible in the central portion of the site. The site is currently undeveloped and vegetated with grass and small brush. There is a small stand of trees in the southern portion of the site, around the area where the depression is present.

It is understood the proposed development will include a single storey commercial building with a footprint of approximately 1,000 m^2. The building will not include a basement.

3. INVESTIGATION PROCEDURES

The geotechnical investigation was carried out on January 8th to 11th, 2013. The scope of work included a field investigation, laboratory testing, analysis and preparation of this report.

A total of five boreholes (BH12-1 through BH12-5) and eleven test-pits (TP12-1 through TP12-11) were excavated at the site.

The boreholes were advanced using a track-mounted drill rig supplied and operated by George Downing Estates Drilling of Hawkesbury, ON. The boreholes were drilled using hollow-stem auger drilling. During drilling, sampling and in-situ testing [including Standard Penetration (SPT) Testing, field shear vane testing and Dynamic Cone Penetration (DCPT) testing], was carried out at regular intervals and disturbed soil samples were recovered.

All boreholes were advanced to refusal which ranged from 5.4 m to 8.4 m below the existing ground surface. At Borehole BH12-2 auger refusal was encountered at 7.3 m depth and rock was cored to 10 m depth using “N” size core barrels.

Standpipe piezometers were installed in Boreholes BH12-2 and BH12-3 to allow for subsequent measurement of stabilized groundwater levels at the site. All boreholes were backfilled with bentonite and soil cuttings and were sealed at the ground surface.

Test-pits were excavated using a rubber-tired hydraulic excavator, supplied and operated by Landraulics Equipment Rentals of Ottawa, ON. The depth of test pitting ranged from 0.7 m to 5.2 m below the existing ground surface. Disturbed soil samples were recovered during excavation of the test-pits.

Upon completion of the field work soil samples were returned to SPL's laboratory for further examination, classification and testing. A laboratory testing program, including determination of natural water content, grain size distribution and Atterberg limits (plasticity) was carried out. Chemical analyses for soil corrosivity were carried out on selected representative soil samples.

An oedometer (consolidation) test was carried out on a relatively undisturbed sample of silty clay obtained during drilling to determine the load-settlement properties of the silty clay layer. The results of this test are included in Appendix C.

Borehole and test pit locations are shown in Drawing No. 2. Locations and elevations were provided to SPL by Broccolini based on a site survey completed subsequent to the field investigation.

4. SUBSURFACE CONDITIONS

The subsurface conditions at the site are discussed in the following sections. Detailed descriptions of the stratigraphy encountered at each of the borehole locations are included in the individual borehole records included in Appendix A. Detailed descriptions of the stratigraphy encountered at each of the test pit locations are included in Appendix B.

4.1 Soil Conditions

The following provides a general description of the major soil types and the general stratigraphy encountered across the site.

4.1.1 Topsoil

Topsoil was encountered at all of the borehole and test-pit locations. The thickness of the topsoil is typically 100 mm to 300 mm in most areas, with the TP12-3 encountering 600 mm in the depression on the south side of the lot.

4.1.2 Brown Sandy Silty Clay

Underlying the topsoil is a layer of brown sandy silty clay. This soil typically extends to a depth of between 1.2 m and 2.7 m below the ground surface. In one test-pit (TP12-11) the brown sandy silty clay material was found to extend to a depth of 3.7 m below ground surface. The consistency of these soils, as interpreted by SPT testing, would be described as firm. Test-pits TP12-4 and 4A and Borehole BH12-4 terminate within this layer (the test pits met refusal on rock at shallow depth; at the borehole drilling was terminated at 1.5 m depth).

Atterberg limit testing carried out on selected samples of the silty clay yielded plastic limit values of 15 to 28, with an average value of around 20, and liquid limit values of 28 to 66 with an average value of 43. These values indicate a silty clay of low to high plasticity. Individual test results are included on the borehole and test pit records.

Natural water contents within the silty clay were found to be between 24 and 35. Individual test results are presented on the borehole and test pit records.

The grain size distributions for selected samples of the sandy silty clay are presented on the borehole records and are summarized in Table 1 below.

Table 1 – Results of Grain Size Analyses for Sandy Silty Clay

Borehole/ Testpit No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH 12-4	2	0	27	46	28
TP 10-1	1	0	23	49	28
TP12-8	1	0	22	46	32

4.1.3 Grey Silty Clay

Underlying the brown silty clay is a layer of grey silty clay. This layer extends underlies most of the site (the only exception being the central area where rock is close to the ground surface). In seven of the test pits excavated the clay was present to the depth of excavation (approximately 5 m) and in most of these locations likely extends beyond the base of the test pit (i.e. is present to more than 5 m below the ground surface). In the boreholes drilled at the site the grey silty clay layer extended to depths of 4.9 m to 8.0 m.

Atterberg limit testing carried out on selected samples of the grey silty clay yielded plastic limit values of 17 to 23, with an average value of around 21, and liquid limit values of 38 to 59 with an average value of 49. These values indicate a clay of medium to high plasticity. The results of Atterberg limit testing in the grey silty clay are included on the relevant borehole and test pit records.

Natural water contents within the silty clay were found to be between 40 and 64 and are generally close to, or above, the materials liquid limit. Individual test results are presented on the borehole and test pit records.

The grain size distributions for two selected samples of the silty clay are presented in Table 2 below.

Table 2 – Results of Grain Size Analyses for SIlty Clay

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
12-1	5	0	2	41	57
12-3	5	0	3	38	60

Oedometer testing was carried out on a single sample of grey silty clay to determine the consolidation characteristics of the soil. The results of this test are included in Appendix C, and are summarized in Table 3 below.

Table 3 – Summary of Consolidation Properties for Silty Clay

Borehole/ Sample No.	Depth	σ_p' (kPa)	σ_{v0}' (kPa)	C_r	C_c	e_0	OCR
BH12-3 Sample 5	4.6 m	150	65	0.15	1.62	1.95	2.3

4.1.4 Sandy Silty Clay Till

In Boreholes BH12-1 and BH12-3 and Test-pits TP12-3 and TP12-9 the silty clay layer was underlain by a layer of sandy silty clay till with trace gravel. This layer extends to refusal in BH12-1 and BH12-3 and test-pits TP12-3 and TP12-9 terminate in this layer. The consistency of the till (interpreted based on SPT "N" values) is compact. In Borehole BH12-5, it is inferred, based on the results of the DCPT that this till layer was encountered at 4.9 m below the ground surface. The till layer was found to be approximately 400 mm to 600 mm thick in the locations where it was encountered, suggesting it is present as a thin veneer over the rock surface.

Atterberg limit testing carried out on a selected sample of the grey silty clay till and yielded a plastic limit value of 16 and a liquid limit value of 30. These values indicate a clay of low to medium plasticity. The natural water content of this sample was determined to be 33, and the till was visibly observed to be in a saturated, relatively soft state. The till was also observed to be intermixed with the grey silty clay where it was encountered in the test pits.

The grain size distributions of two selected samples of the sandy silty clay till are presented in Table 4 below. It should be noted that the grain size distribution tests were carried out on samples obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles. Because of this the grain size distribution shown on Drawing No. 3 and Table 4 may be finer overall than some portions of the materials in the field. Cobbles and boulders were noted within the till and should be anticipated during construction.

Table 4 – Results of Grain Size Analyses for Sandy Silty Clay Till

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
12-1	7	10	23	32	36
12-2	5	6	22	32	40

4.1.5 Auger/Excavator Refusal

All boreholes, with the exception of BH-4 were drilled to auger refusal. At these locations the depth to refusal was found to be 5.4 m to 8.4 m below the existing ground surface. In Borehole BH12-2 after auger refusal was encountered at 7.3 m and rock was cored to a depth of 10.0 m. The bedrock recovered is described as fresh to slightly weathered grey granite. Based on the Rock Quality Designation (RQD) of the cores obtained the rock quality would be described as ranging from good to excellent.

Test pits TP12-2, TP12-4, TP12-4A, TP12-6, and TP12-7 encountered refusal at depths ranging from 0.3 m to 4.0 m on what was inferred to be either bedrock or the overlying till layer.

4.1.6 Simplified Soil Profiles

The following table provides an overview of the soil strata encountered at each of the borehole and test pit locations. Detailed descriptions are included on the relevant borehole records included in Appendix A and test pit records included in Appendix B.

Table 5 – Simplified Soil Profiles

Borehole/ Testpit	Simplified Stratigraphy (m)				Refusal Depth (m)	Ground water Depth (m)	Termination
	Topsoil	Brown Silty Clay	Grey Silty Clay	Silty Clay Till			
BH12-1	0 - 0.2	0.2 - 2.3	2.3 - 6.1	6.1 - 6.7	6.7	--	Auger and SPT Refusal at 6.7 m
BH12-2	0 - 0.1	0.1 - 2.3	2.3 - 7.3	--	7.3	2.4	Auger Refusal at 7.3 m; rock cored to 10.0 m
BH12-3	0 - 0.3	0.3 - 2.3	2.3 - 8.0	8.0 - 8.4	8.4	2.6	Auger Refusal at 8.4 m
BH12-4	0 - 0.2	0.2 - 1.5	--	--	--	--	No Refusal Encountered
BH12-5	0 - 0.2	0.2 - 2.3	2.3 - 4.9	4.9 - 5.4	5.4	--	DCPT refusal at 5.4 m
TP12-1	0 - 0.2	0.2 - 2.6	2.6 - 5.0	--	--	--	No Refusal Encountered
TP12-2	0 - 0.3	0.3 - 2.6	2.6 - 4.0	--	4	--	Excavator Refusal at 4.0 m
TP12-3	0 - 0.6	0.6 - 2.4	2.4 – 4.5	--	--	--	No Refusal Encountered
TP12-4	0 - 0.1	0.1 - 0.7	--	--	0.7	--	Excavator Refusal at 0.7 m
TP12-4A	0 - 0.1	0.1 - 0.3	--	--	0.3	--	Excavator Refusal at 0.3 m
TP12-5	0 - 0.3	0.3 - 1.8	1.8 - 4.9	--	--	--	No Refusal Encountered
TP12-6	0 - 0.4	0.4 - 1.2	1.2 - 2.8	--	2.8	--	Excavator Refusal at 2.8 m
TP12-7	0 - 0.2	0.2 - 1.8	1.8 - 3.5	--	3.5	--	Excavator Refusal at 3.5 m

Borehole/ Testpit	Simplified Stratigraphy (m)				Refusal Depth (m)	Ground water Depth (m)	Termination
	Topsoil	Brown Silty Clay	Grey Silty Clay	Silty Clay Till			
TP12-8	0 - 0.2	0.2 - 2.1	2.1 - 5.2	--	--	--	No Refusal Encountered
TP12-9	0 - 0.2	0.2 – 1.2	1.2 - 4.6	4.6 - 5.0	--	--	No Refusal Encountered
TP12-10	0 - 0.1	0.1 - 2.7	2.7 - 5.2	--	--	--	No Refusal Encountered
TP12-11	0 - 0.2	0.2 - 3.7	3.7 - 5.0	--	--	--	No Refusal Encountered

4.2 Groundwater Conditions

Standpipe piezometers were installed in Boreholes BH12-2 and BH12-3 to allow for subsequent measurement of stabilized groundwater levels at the site. The measured groundwater levels were 2.6 and 2.5 m, respectively. This corresponds to elevations of 91.1 m and 91.2 m, respectively.

During the excavation of test pits TP12-3 and TP12-9, seepage of groundwater was noted and the test-pits were left open. In TP12-3, excavated to a depth of 4.5 m and in TP12-9, excavated to a depth of 5.0 m, less than 0.1 m of water accumulated in the base of the excavation over a period of approximately four hours.

5. DISCUSSION AND RECOMMENDATIONS

5.1 General

This section of the report presents geotechnical recommendations for the proposed development. The recommendations included in this section are intended to provide the designer with the information required to select the most suitable foundation type(s) and to complete the detailed design of the various components of the project. Where comments are made concerning construction considerations they are intended to provide the designer with an understanding of the geotechnical issues associated with the various aspects of the project. Those requiring detailed information regarding construction aspects of the project should review the factual information and draw their own conclusions as to how the subsurface conditions may affect their work.

5.2 Site Classification for Seismic Site Response

The site can be classified as a Site Class “D” for the purposes of site-specific seismic response to earthquakes.

5.3 Frost Protection

Exterior foundations of heated structures should be provided with a minimum of 1.5 m of cover (or the thermal equivalent if insulation is used) for the purposes of protection from frost. Foundations of unheated structures should be provided with a minimum of 1.8 m of earth cover (or equivalent insulation).

5.4 Grade Raise

Detailed building plans are not available at this time, however it is understood the grade on the site will be raised to slightly higher than Hearst Way. Based on survey data provided to us the elevation of Hearst Way is approximately 94.5 m to 94.9 m. The south half of the site is at an elevation as low as 92.8 m and the site rises to the north to as high as 94.2 m. This implies the grade at the site will be raised up to two meters in the area where the existing ground is depressed, and up to 1 m in other areas (based on a preliminary survey and our understanding of the building plans).

This grade raise will cause settlement in the grey silty clay layer which underlies the site. For discussion purposes, a preliminary settlement analysis has been carried out to determine the approximate amount of settlement which should be expected. The analysis is based on the consolidation properties measured during oedometer testing of the silty clay and the magnitude of the grade raise discussed above.

The results of this preliminary analysis indicate the proposed grade raise would be expected to cause settlement on the order of:

Table 6 – Calculated Consolidation Settlement Due to Grade Raise

Grade Raise	Calculated Settlement
1 m	30 mm
2 m	55 mm

These values are the settlements due to the raising of the grade only, and do not include settlement due to the actual building foundations.

It is our understanding that pre-loading the site is not an option due to schedule concerns, and so settlement of these magnitudes should be anticipated post-construction due to the grade raise.

5.5 Foundations

The subsurface conditions at the site are highly variable. In the central portion of the site bedrock is exposed at the ground surface or is at very shallow depth. In these locations, conventional spread footings can be constructed on rock.

In other areas the boreholes encountered rock at depths ranging from 5.4 m to 8.4 m below the existing ground surface. In addition, several test pits were ended at depths of approximately 5 m in the grey silty clay, suggesting rock is more than 5 m below the ground surface in these areas.

In areas where rock is 5 m or more below the existing ground surface, the estimated settlement due to the raising of the site grade is on the order of 30 mm to 60 mm, depending upon the actual rock depth and the amount by which the site is raised. Settlements of this magnitude are unlikely to be tolerable and therefore a system of deep foundations with a structural slab will likely be required where footings cannot be founded on rock.

5.5.1 Shallow Foundations on Rock

In areas where the bedrock is shallow, conventional spread footings may be founded directly on rock. For foundations placed on rock the unfactored ultimate geotechnical bearing resistance can be taken as 3,000 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 1,500 kPa at ULS (Ultimate Limit States).

Shallow foundations on rock typically experience relatively small settlement, and would be expected to be less than the 25 mm which is commonly allowed for.

5.5.2 Deep Foundations

The combination of the raising of the site grade and foundation loads is expected to cause more settlement than is typically considered to be acceptable for normal building construction. As a consequence deep foundations will likely be required for any foundation elements where the silty clay layer is thickest.

The most cost-effective type of pile is likely to be driven steel piles. Drilled cast-in-place concrete piles are also technically feasible, but less commonly used. Two types of driven steel piles are commonly used in the area:

- H-piles; and
- Concrete filled, closed ended, steel pipe piles

Either pile type would be expected to be adequate.

<u>Compressive Resistance</u>

Steel piles should be driven to rock which was encountered at depths up to 8 m below the existing ground surface.

Piles driven to 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 this value to obtain the factored geotechnical resistance of a pile driven to 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 rock can therefore be assumed to be 1,395 kN (0.4 x 3,490).

Settlements for piles driven to 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 rock.

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:

$$q_s = \alpha S_u$$

where: q_s = the unfactored shaft resistance (in kPa)
α = a shaft resistance factor based on soil type (use 0.7)
S_u = the undrained shear strength of the soil (use 35 kPa)

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 a soil weight of 18 kN/m^3).

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

It should be noted that the uplift resistance is highly dependant 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.

5.5.1.3 Lateral Resistance

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 piles is the coefficient of horizontal subgrade reaction (k_h). For this site k_h may be assumed to be:

$$k_h = 67\ S_u$$

Where: k_h = the modulus of subgrade reaction (kN/m^3);

S_u = undrained shear strength (use 50 kPa for upper 2 m and 35 kPa below);

This parameter is associated with acceptable deflections, and therefore represents an unfactored SLS value.

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_h) 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 spacing's 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).

It should be noted that many of the piles will likely be relatively short and will function essentially as short columns rather than piles in the conventional sense. These very short piles will be incapable of developing any significant resistance to lateral loads.

Negative Skin Friction

The raising of the grade will cause settlement of the existing soils which will in turn cause negative friction or down drag on the piles.

The magnitude of negative skin friction depends on the pile loading, dimensions and the final grading of the site 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 be 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.

Construction Considerations

The piles will be driven to bedrock (which is expected to be up to 8 m below the existing ground surface, and in many places will be much shallower). Based on the results of the test pitting and borehole drilling the rock surface is expected to be sloping and piles should be driven with rock points to avoid sliding along the rock surface. Pipe piles should be driven closed-ended. Even with these measures, 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.

5.6 Lateral Earth Pressures

The lateral earth pressure acting on below-grade walls, retaining walls, etc. may be calculated using the following expression:

$$P = K(\gamma h+q)$$

Where P = lateral earth pressure (kPa) acting at depth h

K = earth pressure coefficient; for unrestrained walls and structures where some movement is acceptable use a coefficient of active earth pressure (K_a) equal to 0.3, for restrained walls which cannot move use the coefficient of earth pressure at rest (K_0) equal to 0.5

γ = the density of the backfill; use 21 kN/m^3 for compacted granular backfill

h = the depth to the point of interest (m)

q = the magnitude of any design surcharge at the ground surface; a minimum nominal surcharge of 10 kPa is recommended, a higher value should be used if appropriate for the building/site design

A minimum lateral earth pressure of 12 kPa should be used to account for the effects of compaction-induced earth pressure (i.e. if the calculated earth pressure at a given point is less than 12 kPa, use 12 kPa).

The above values assume free-draining granular backfill will be used. If this is not the case then the above values may need to be adjusted based on the soil type used, and water pressures should be considered in the calculation of lateral pressures. SPL can provide additional guidance based on actual building plans if required.

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the total earth pressure during a seismic event (including both the seismic and static components) may be assumed to be:

$$\sigma_h(z) = K_a \gamma z + (K_{AE} - K_a) \gamma (H\text{-}z)$$

Where $\sigma_h(z)$ = the total earth pressure at depth z (kPa);
K_a = the active earth pressure coefficient (0.3);
γ = the unit weight of soil (21 kN/m^3 for granular fill or 19 kN/m^3 for native soils);
K_{AE} = the combined active earth pressure and seismic earth pressure coefficient (use 0.8);
H = the total height of the wall (m)
z = the depth below the top of the wall (m)

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

5.7 Temporary Excavations and Groundwater Control

All temporary excavations should be carried out in accordance with the most recent Occupation Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations. The soils encountered at the site include brown sandy silty clay, grey silty clay and silty clay till. For the purposes of preliminary excavation planning the brown sandy clay material may be considered as a Type 3 soil. The grey silty clay and the silty clay till (which is below the water table) should be considered as Type 4 soil. This classification should be confirmed by qualified individuals as the site is excavated and if necessary adjusted.

Shallow excavations (less than 2.4 m deep) will likely be at or above the groundwater table and, given that, it is likely that seepage into the excavations can be managed using properly filtered sumps, ditches, etc. In the event that larger or deeper excavations are required then additional dewatering or more complex excavation support may be required. SPL can provide additional guidance if required during detailed design.

5.8 Backfilling and Compaction

Backfill for below-grade walls, retaining walls, foundation excavations, etc. should comprise fee draining Granular “A” or “B” materials. Backfill should be placed in shallow lifts, not exceeding 200 mm loose thickness, and compacted to 98% SPMDD where it is supporting any structures or services, or 95% in other areas.

The existing site materials do not meet the requirements for Granular “A” or “B” materials. The suitability of imported materials should be confirmed prior to placement from both a geotechnical and environmental perspective.

To avoid damaging or laterally displacing the structures, care should be excercised when compacting fill adjacent to new structures. Where possible, backfilling should be carried out on all sides of the structure simultaneously. Heavy equipment should be kept a minimum of 1 m away from the structure during backfilling. The 1 m width adjacent to the wall should be backfilled using hand-operated equipment unless otherwise authorized.

5.9 Site Services

Water-bearing services should be placed a minimum of 1.8 m below grade to provide protection from frost. Alternatively, equivalent insulation cover may be provided in lieu of burial.

Details of the proposed site services are not available at this time; however it is assumed that they will include localized trenches throughout the site. Trenches can be temporarily supported using sloped excavations (see Section 5.7) or trench boxes.

The designer of the site services should be aware of the potential settlement which could arise as a result of the raising of the grade. This would apply to services below the structural floor slab as well, which would need to either be attached to the underside of the floor slab to prevent settlement or designed to tolerate the expected settlement.

Bedding for site services should be in accordance with the relevant OPSD standard drawing and would typically consist of Granular "A" compacted to 95% SPMDD. Where wet or disturbed conditions are encountered in the base of the trench it may be necessary to over-excavate and replace unsuitable soils with compacted granular fill to provide a stable sub-grade for the bedding. The use of clear stone as a bedding and cover material is not recommended as the finer particles of the native soils and backfill may migrate into the voids of the clear stone, resulting in loss of pipe support.

Cover material above the spring line should consist of Granular "A" or Granular "B" material with a maximum particle size of 25 mm. Cover material should be compacted to a minimum of 95% SPMDD.

Backfill may consist of additional granular fill and should be compacted to 95% SPMDD (98% if below structures). Where backfill is below paved areas (such as parking lots and access roads) and is within the frost depth, the backfill profile (above the minimum cover required) in the trench should be made to match the native soils on either side as much as is practical in order to minimize the potential for differential frost heave. As a result, portions of the brown silty clay which is typically encountered in the upper 2 m may be retained, moisture conditioned and re-used.

Any service trenches which extend below the water table should have clay cut-offs installed across the trench to prevent the trench acting as a drain and lowering the groundwater table in the general area.

The above are general guidelines for typical site services. All services installations should be completed in accordance with the relevant OPSS's and OPSD's for the particular application and size. SPL can provide additional review during detailed design based on the actual services proposed if required.

5.10 Pavement Structures

Typical pavement structures are provided in Table 8. No detailed traffic information has been provided as part of this study and these structures are based on experience with similar projects using an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples.

A functional design life of eight to ten years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming regular maintenance is carried out. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements but would involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from the client.

It should also be noted that due to the raising of the grade the site is expected to settle some 30 mm to 60 mm following construction. Most of this settlement will occur before the normal eight to ten year lifespan of the asphalt. As a result, some re-grading of the parking areas may be required prior to the normal pavement lifespan at this particular site (particularly in areas with a large grade raise).

Table 8: Recommended Pavement Structure Thickness

Pavement Layer	**Light Duty Parking (Cars)**	**Heavy Duty Parking (Delivery Trucks, Fire Route, etc.)**
Asphaltic Concrete	40 mm HL 3 or SP 12.5 40 mm HL 8 or SP 19.0	40 mm HL 3 or SP 12.5 80 mm HL 8 or SP 19.0
OPSS Granular A Base	150 mm	150 mm
OPSS Granular B Sub-Base	300 mm	450 mm

The long term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over-emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond within or adjacent to paved areas. Subdrains should be installed (as per typical civil design) to intercept excess subsurface moisture and prevent subgrade softening. This is particularly important in heavy-duty pavement areas.

Additional comments on the construction of parking areas and access roadways are as follows:

- As part of the subgrade preparation, proposed parking areas and access roadways should be stripped of topsoil and other obvious objectionable material. Fill required to raise the grades to design elevations should conform to backfill requirements outlined in previous sections of this report. The subgrade should be properly shaped, crowned then proof-rolled in the full time presence of a qualified individual. Soft or spongy subgrade areas should be sub-excavated and properly replaced with suitable approved backfill compacted to 98% SPMDD.

- Proper sub-drainage should be included in the overall pavement design to ensure that water which infiltrates through the pavement surface is drained away and does not become trapped at the base of the granular layers. Assuming that satisfactory crossfalls in the order of two percent have been provided, subdrains extending from and between catch basins may be satisfactory. In the event that shallower crossfalls are considered, a more extensive system of sub-drainage may be necessary.

- The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted access lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavourable weather.
- It is recommended that SPL Consultants Limited be retained to review the final pavement structure designs and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

5.11 Corrosion and Cement Type

Samples of the existing soils were submitted to Exova Accutest for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix D and summarized in Table 8 below.

Table 9 – Results of Soil Corrosivity Testing

Borehole/Testpit Sample No.	Soil Type	Chloride (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)	Sulphate (%)
BH12-1/S2	Silty Clay	0.003	0.11	7.5	9090	<0.01
BH12-5/S3	Silty Clay	0.003	0.20	8.1	5000	<0.01
TP12-8/S3	Silty Clay	0.006	0.20	8.7	5000	<0.01

The soil resistivity values measured in the existing fill suggest a moderately corrosive environment for buried steel elements.

The test results indicate negligible soluble sulphate content and as such sulphate resistant Portland cement is not required.

6. GENERAL COMMENTS

It is understood that SPL Consultants Limited will provide a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the opportunity to undertake this review, SPL Consultants Limited will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and laboratory test results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

7. LIMITATIONS OF REPORT

The limitations of this report are included in Appendix E.

8. CLOSURE

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

SPL CONSULTANTS LIMITED

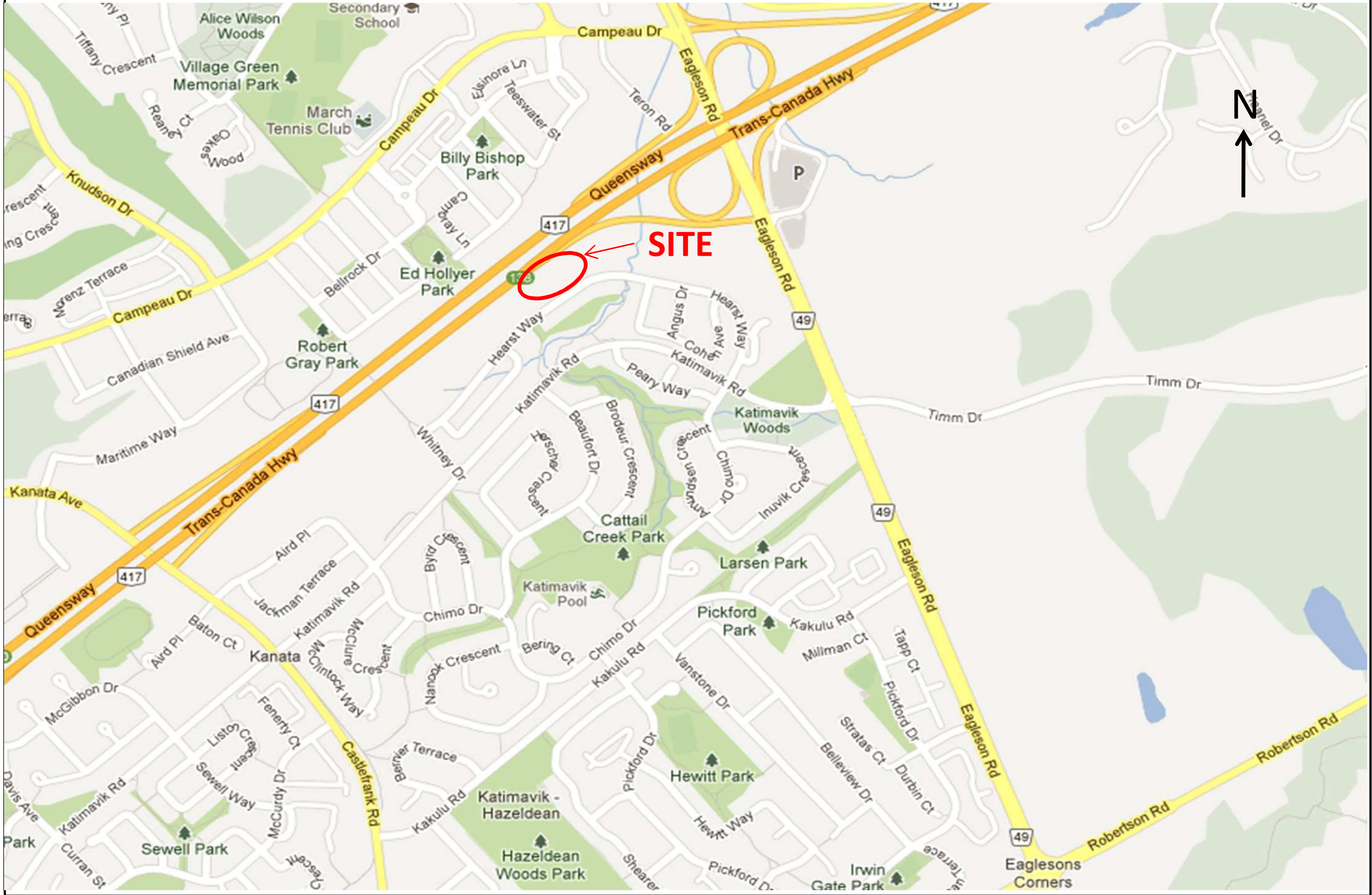
LICENSED PROFESSIONAL ENGINEER
C. G. HENDRY
100011328
PROVINCE OF ONTARIO

Chris Hendry, M.Eng., P.Eng.

SBandukwala

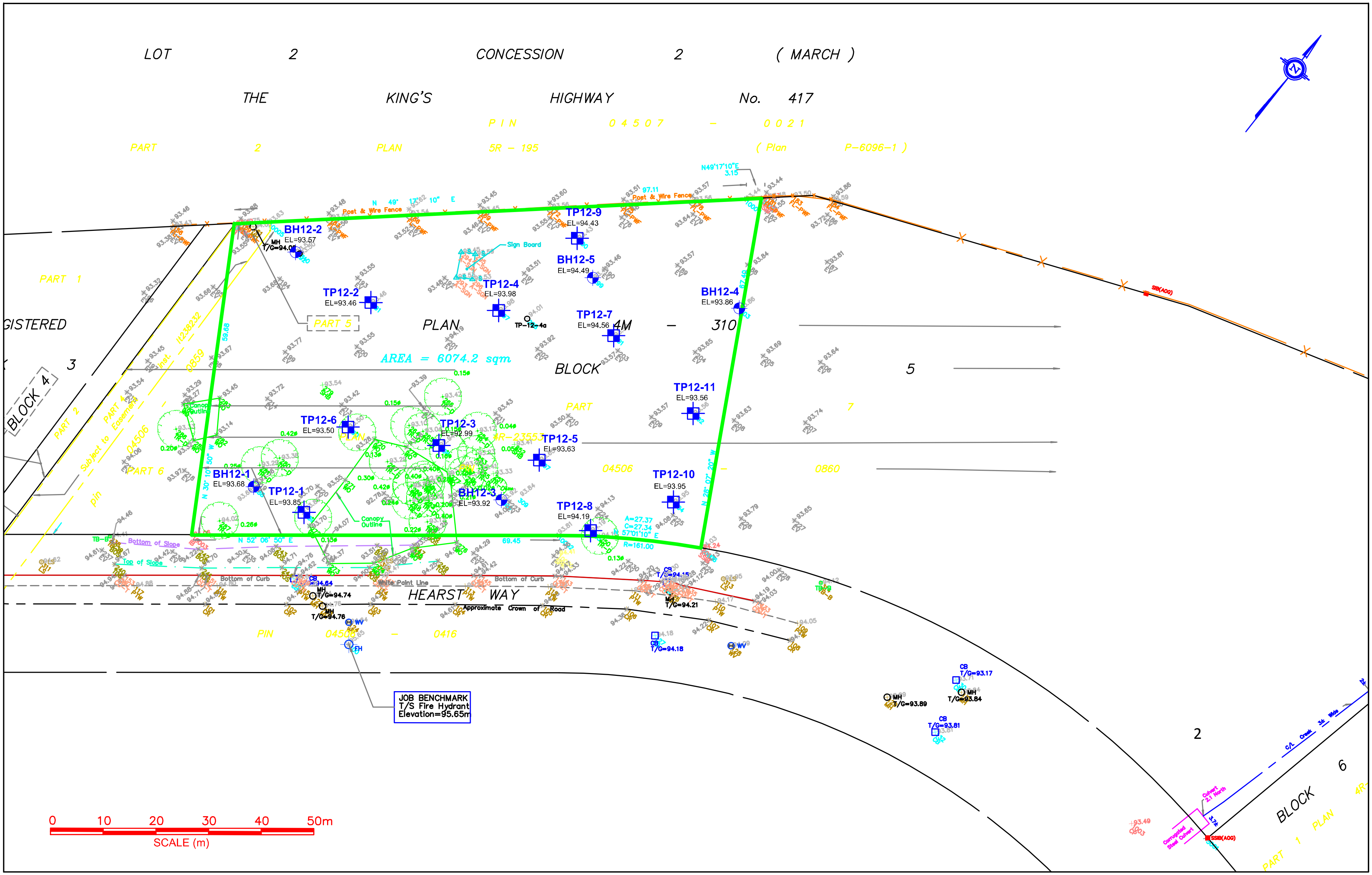
Shabbir Bandukwala, M.Eng., P.Eng.

Drawings



Client:	Broccolini Construction			Title:	SITE PLAN
Project#:	1465-710	DWG #:	1	Project:	Geotechnical Investigation 120 Hearst Way, Ottawa, ON
Drawn:	DW	Approved:	CH		
Date:	January, 2013	Scale:	N. T. S.		SPL Consultants Limited Geotechnical Environmental Materials Hydrogeology
Size:	Letter	Rev:	0		

server\public\Geotechnical\1400 GeoProjects\1465-710 – 120 Hearst Way, Ottawa\1465-710 Borehole and Testpit Location Pla



LEGEND

BH12-1 ◆ Borehole Location

TP12-1 ✚ Testpit Location

Client: BROCCOLINI CONSTRUCTION		Project No.: 1465-710	Drawing No.: 2
Drawn: ZO	Approved: CH	Title: BOREHOLE AND TESTPIT LOCATION PLAN	
Date: January, 2013	Scale: As shown	Project: GEOTECHNICAL INVESTIGATION - 120 HEARST WAY, OTTAWA, ONTARIO	
Original Size: Letter	Rev: N/A	SPL Consultants Limited Geotechnical * Environmental * Materials * Hydrogeology	

Appendix A

Borehole Records

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-1

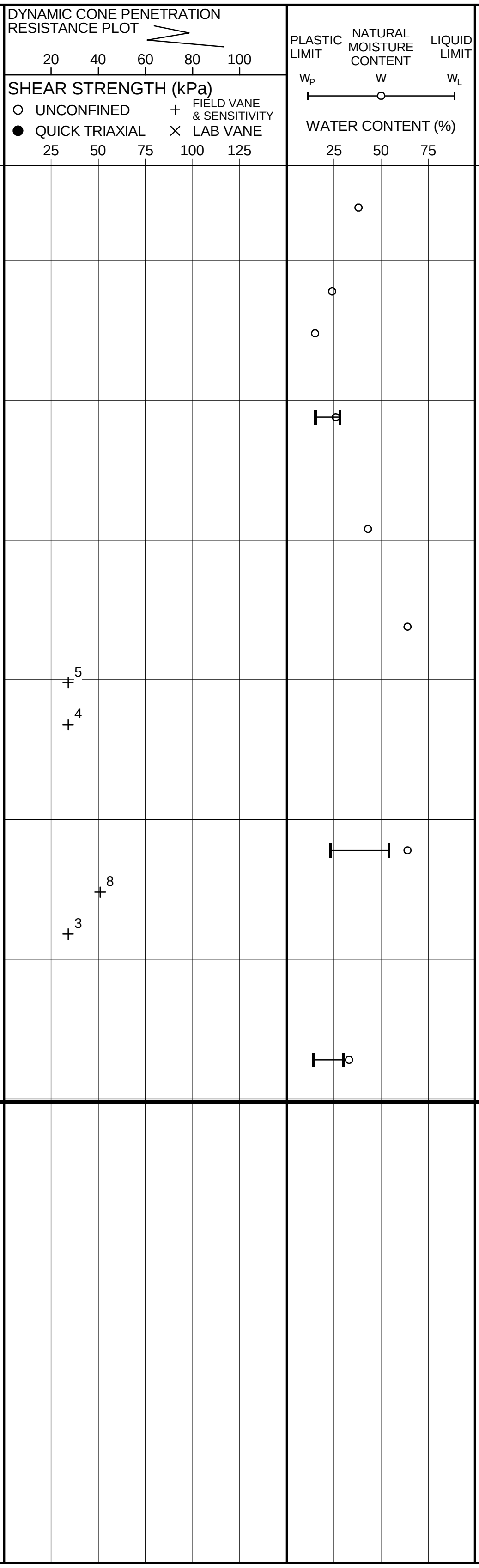
PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger
Diameter: 203mm
Date: Jan/11/2013

REF. NO.: 1465-710
ENCL NO.:

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
93.7 0.0	**TOPSOIL AND ORGANICS**						
93.5 0.2	**SANDY SILTY CLAY** brown, moist, firm	1	SS	7			
						93	
		2	SS	5			
	-Sandy seam from 1.2 m to 1.35 m	2B	SS	5			
						92	
		3	SS	7			
91.4 2.3	**SILTY CLAY** grey, moist, firm	4	SS	WH			
						91	
		5	SS	WH			
						90	
			VANE				
			VANE				
						89	
		6	SS	WH			
	- firm to stiff below 5.2 m		VANE				
			VANE				- SPT Refusal at 6.7 m (50/0 mm)
						88	
87.6 6.1	**SANDY SILTY CLAY** trace gravel, grey, moist (Till)	7	TW				10 23 32 36
87.0 6.7	**END OF BOREHOLE**					87	

Notes:
1. SPT refusal at 6.7m
2. Borehole was dry upon close



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

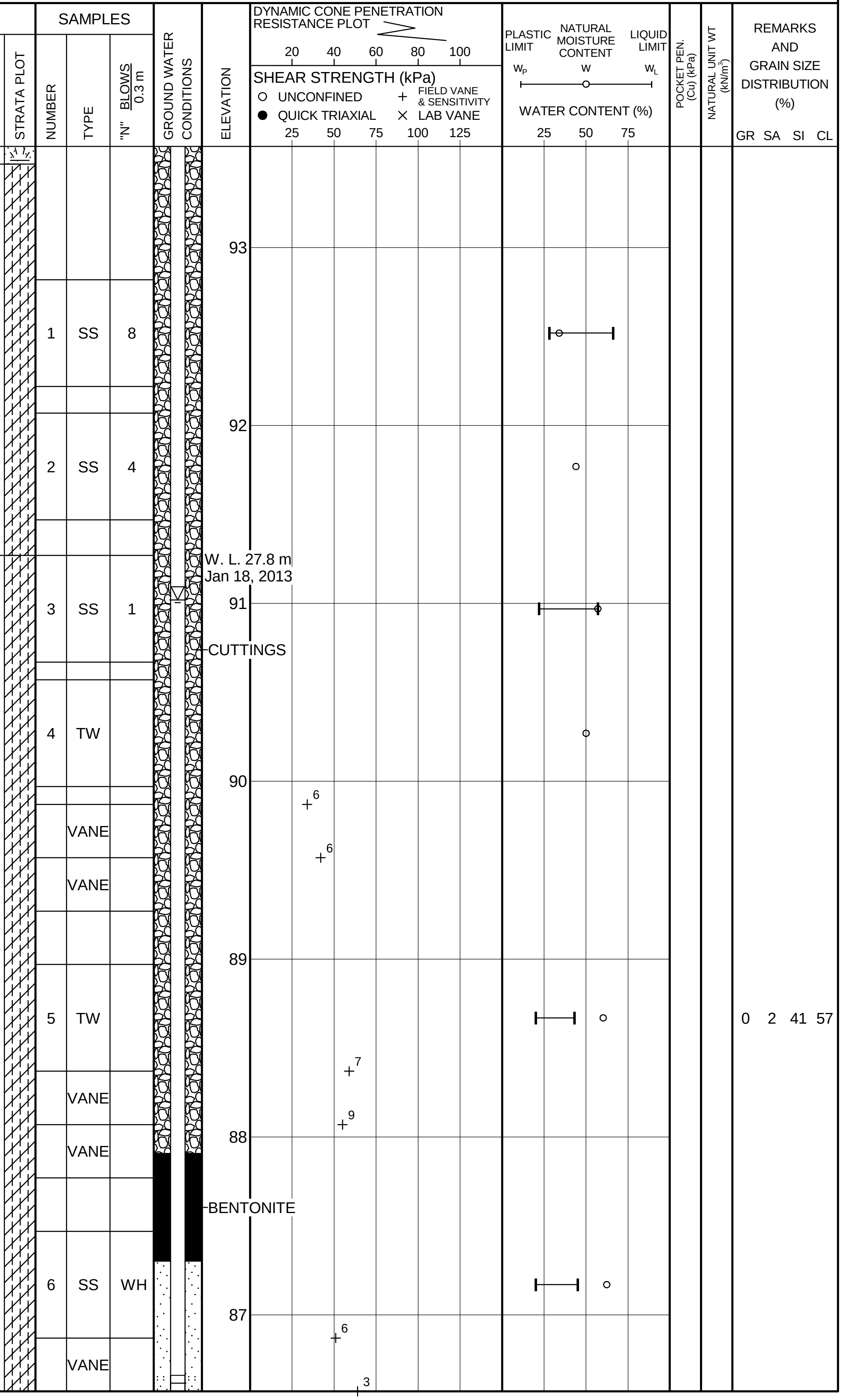
LOG OF BOREHOLE BH12-2

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger/N Size Core
Diameter: 203mm/N size core
Date: Jan/10/2013

REF. NO.: 1465-710
ENCL NO.:

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	GR	SA	SI	CL
93.6								
93.5 0.0	**TOPSOIL AND ORGANICS**							
0.1	**SANDY SILTY CLAY** brown, moist, firm							
		1	SS	8				
		2	SS	4				
	- Sandy seam 1.9 - 2.1 m							
91.3 2.3	**SILTY CLAY** trace sand, grey, moist, firm							
		3	SS	1				
		4	TW					
			VANE					
			VANE					
		5	TW		0	2	41	57
	- stiff below 5.2 m		VANE					
			VANE					
		6	SS	WH				
	- SPT refusal at 7.3 metres (50/25 mm). Bedrock fragment in spoontip.		VANE					



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GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

SPL SOIL LOG 120 HEARST WAY.GPJ SPL.GDT 25/1/13

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-2

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger/N Size Core
Diameter: 203mm/N size core
Date: Jan/10/2013

REF. NO.: 1465-710
ENCL NO.:

SOIL PROFILE			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)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION					
86.3	Switched to coring **SILTY CLAY** trace sand, grey, moist, firm			VANE								
7.3 85.0	**Granite**. Fresh to slightly weathered, very widely spaced horizontal joints. Grey. - Vertical joint from 7.62 m to 8.19 m. - Horizontal joint between 7.86 m to 7.89 m with calcite TCR = 100% SCR = 95% RQD = 95%		RC 1	CORE		SAND SCREEN	86 85					
8.6 83.5	**Granite**. Fresh to slightly weathered, very widely spaced horizontal joints. Grey. TCR = 94% SCR = 83% RQD = 79% - Vertical joint between 9.53 m to 9.92 m		RC 2	CORE		SLOUGH	84					
10.1	**END OF BOREHOLE**											

Notes:
1. 3/4" piezometer installed at 10 m
2. Water levels

Date	Depth
Jan 18th, 2013	2.55 m BGL

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

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-3

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger
Diameter: 203mm
Date: Jan/10/2013

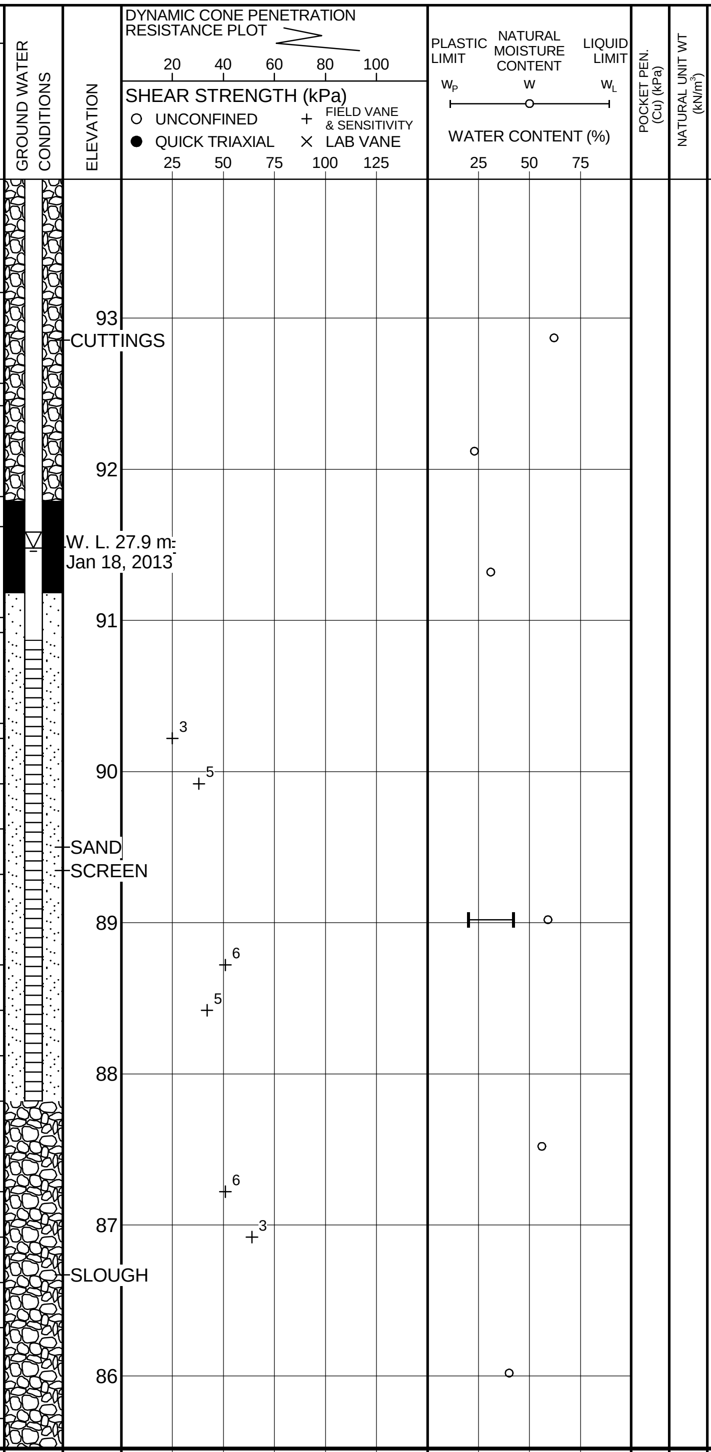
REF. NO.: 1465-710
ENCL NO.:

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
93.9	
0.0	**TOPSOIL AND ORGANICS**
93.7 / 0.3	**SANDY SILTY CLAY** brown, moist, firm
91.6 / 2.3	**SILTY CLAY** trace sand, grey, moist, firm - firm to stiff below 5.2 m - stiff below 6.1 m
85.9 / 8.0	**SANDY SILTY CLAY** some sand, trace gravel, moist, firm-stiff (Till)
85.5 / 8.4	**END OF BOREHOLE**

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	
2	SS	4	
3	SS	1	
4	SS	WH	
	VANE		
	VANE		
5	TW		0 3 38 60
	VANE		
	VANE		
6	SS	WH	
	VANE		
	VANE		
7	SS	1	6 22 32 40



Notes:
1. Water level was at apx. 0.1 m above base of borehole upon close
2. 3/4" piezometer installed at 6.1 m in depth
3. Water levels

<u>Date</u>	<u>Depth</u>
Jan 18th, 2013	2.44 m

Continued Next Page

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-3

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger
Diameter: 203mm
Date: Jan/10/2013

REF. NO.: 1465-710
ENCL NO.:

SOIL PROFILE			SAMPLES			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
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m							
	BGL											

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

SPL SOIL LOG 120 HEARST WAY.GPJ SPL.GDT 25/1/13

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

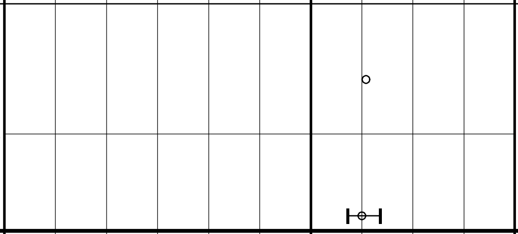
LOG OF BOREHOLE BH12-4

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger
Diameter: 203mm
Date: Jan/11/2013

REF. NO.: 1465-710
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
93.9												
93.9 0.0	**TOPSOIL AND ORGANICS**											
93.7 0.2	**SANDY SILTY CLAY** brown, moist		1	GRAB								
							93					
92.4			2	GRAB								0 27 46 28
1.5	**END OF BOREHOLE** Notes: 1. Borehole was dry upon close											



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

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

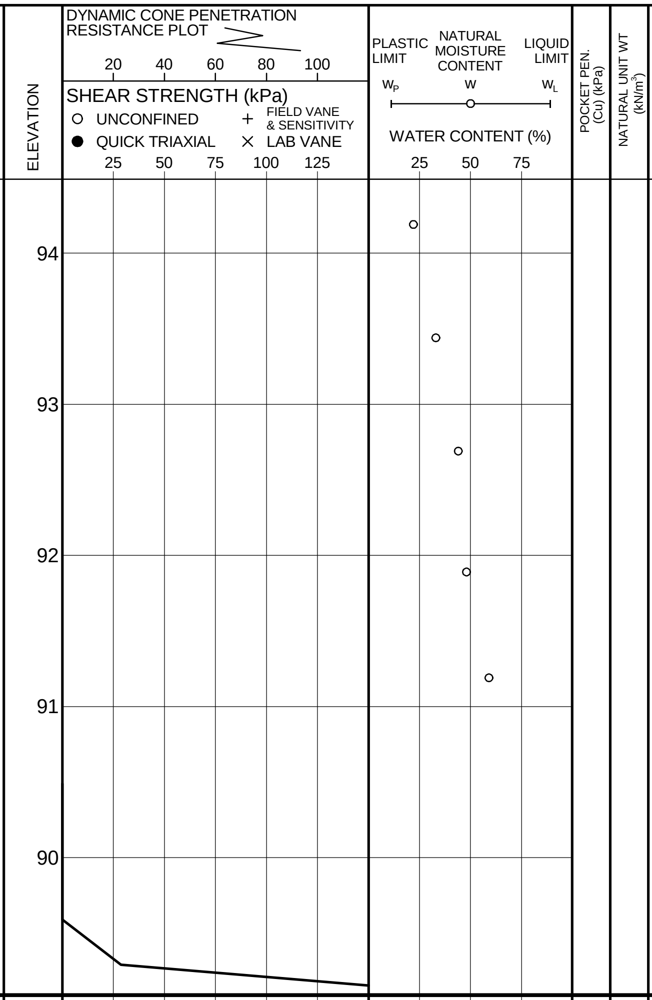
LOG OF BOREHOLE BH12-5

PROJECT: 120 Hearst Way
CLIENT: Broccolini
PROJECT LOCATION: 120 Hearst Way, Ottawa, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Track Mounted Hollow Stem Auger
Diameter: 203mm
Date: Jan/11/2013

REF. NO.: 1465-710
ENCL NO.:

SOIL PROFILE		SAMPLES				REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	GR SA SI CL
94.5 0.0	**TOPSOIL AND ORGANICS**					
94.3 0.2	**SANDY SILTY CLAY** brown, moist, firm	1 2 3	SS SS SS	6 6 4		
92.2 2.3	**SILTY CLAY** grey, moist - DCPT blow count interperated as 0 (rods sinking under the weight of the hammer)	4 5	SS SS	WH WH		
89.6 4.9	- Based on DCPT results, infered to be glacial till					- DCPT refusal at 5.4 m
89.1 5.4	**END OF BOREHOLE** Notes: 1. Borehole drilled to 3.7 m and continued using DCPT until refusal 2. Borehole dry upon close					



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

SPL SOIL LOG 120 HEARST WAY.GPJ SPL.GDT 25/1/13

Appendix B

Test Pit Records

Record of Test Pitting

Notes:

w = natural water content

LL = Liquid Limit

PL = Plastic Limit

Gravel: % = % gravel in sample

Sand: % = sand in sample

Silt: % = % silt in sample

Clay: % = % clay in sample

Test Pit TP12-1 January 8, 2013 Elevation 93.85

Depth (m)	Material Description	Samples
0 - 0.15	Topsoil and organics	
0.15 - 0.6	Brown loose silty clay with roots	S1 - 0.6 m: w = 23%
0.6 - 2.6	Silty clay, brown (weathered), moist	S2 - 2.3 m: w = 24% LL = 44 PL = 20
2.6 - 5.0	Silty clay, grey (unweatered), moist to very moist	S3 - 3.0 m: w = 60%

Notes: End of Test Pit @ 5 m. No seepage noted in test pit during excavation.

Test Pit TP12-2 January 8, 2013 Elevation 93.46

Depth	Material Description	Samples
0 – 0.3	Topsoil	S1 - 0.1 m: w = 26%
0.3 – 2.6	Silty Clay, brown (w/oxidization), roots cobbles/boulders noted	S2 - 0.3 m: w = 25% S3 - 1.9 m: w = 39%
2.6 – 4.0	Silty Clay, grey, moist	S4 - 3.1 m w = 52% S5 - 3.4 m w = 50%

Notes: End of Test Pit @ 4 m due to excavator refusal. No seepage noted in test pit during excavation.

Test Pit TP12-3 January 8, 2013 Elevation 92.89

Depth (m)	Material Description	Samples
0 – 0.6	Topsoil	S1 - 0.2 m W = 32%
0 – 2.4	Silty clay, brown, trace organics (roots)	S2 - 0.8 m W = 28% LL = 52 PL = 24 S3 - 2.2 m W = 31% LL = 54 PL = 21
2.4 – 4.5	Silty clay, grey, moist	S4 - 3.2 m W = 60% LL = 53 PL = 23

Notes: End of Test Pit @ 4.5 m. Seepage noted at base of test pit. Cobble noted at base of test pit. Test pit left open for approximately 4 hours. 25 mm of water accumulated in the base of pit.

Test Pit TP12-4 **January 8, 2013** **Elevation 93.98**

Depth (m)	Material Description	Samples
0 – 0.1	Topsoil	
0.1 – 0.7	Silty Clay, brown, some roots	S1 - 0.67 m W = 38%

Notes: End of Test Pit @ 0.7 m due to excavator refusal (rock). No seepage noted in test pit during excavation.

Test Pit TP12-4A **January 8, 2013** **Elevation 93.98**

Depth (m)	Material Description	Samples
0 – 0.1	Topsoil	
0.1 – 0.3	Silty Clay, brown, some roots	

Notes: End of Test Pit @ 0.3 m due to excavator refusal (rock). No seepage noted in test pit during excavation.

Test Pit TP12-5 **January 8, 2013** **Elevation 93.63**

Depth (m)	Material Description	Samples
0 – 0.3	Topsoil and organics	
0.3 – 1.8	Stiff brown silty clay, moist	S1 - 0.7 m w = 18% S2 - 2.2 m w = 48%
1.8 – 4.9	Soft-Firm silty clay, grey, moist	S3 - 3.2 m W = 57% LL = 46 PL = 21

Notes: End of Test Pit @ 4.9 m. No seepage noted in test pit during excavation.

Test Pit TP12-6 **January 8, 2013** **Elevation 93.50**

Depth (m)	Material Description	Samples
0 – 0.4	Topsoil and organics	
0.4 – 1.2	Stiff brown silty clay, moist	S1 - 0.9 m w = 16%
1.2 – 2.8	Soft-Firm silty clay, grey, moist	S2 - 2.3 m w = 50% S3 - 2.6 m w = 16%

Notes: End of Test Pit @ 2.8 m due to excavator refusal. No seepage noted in test pit during excavation.

Test Pit TP12-7 **January 8, 2013** **Elevation 94.56**

Depth (m)	**Material Description**	**Samples**
0 – 0.15	Topsoil and organics	
0.15 – 0.6	Stiff brown silty clay, moist, some roots	
0.6 – 1.8	Stiff brown silty clay, moist	S1 - 1.5 m W = 20%
1.8 – 3.5	Soft-Firm silty clay, grey, moist	S1 - 2.7 m W = 59%

Notes: End of Test Pit @ 3.5 m due to excavator refusal. No seepage noted in test pit during excavation.

Test Pit TP12-8 **January 8, 2013** **Elevation 94.19**

Depth (m)	**Material Description**	**Samples**
0 – 0.15	Topsoil and organics	
0.15 – 2.1	Stiff brown silty clay, moist	S1 - 1.2 m Gravel: 0% Sand: 22% Silt: 46% Clay: 31% w = 23%
2.1 – 3.1	Becoming more grey and firm	S2 - 2.7 m w = 31% LL = 29 PL = 15
3.1 – 5.2	Soft-Firm grey silty clay moist to very moist	S3 - 5.2 m w = 62% LL = 50 PL = 23

Notes: End of Test Pit 5.2 m. No seepage noted in test pit during excavation.

Test Pit TP12-9 **January 8, 2013** **Elevation 94.43**

Depth (m)	**Material Description**	**Samples**
0 – 0.2	Topsoil and organics	
0.2 – 1.2	Stiff brown silty clay, moist	S1 - 1.1 m w = 34% LL = 50 PL = 24
1.2 – 2.1	Stiff grey silty clay (less weathered), moist	
2.1 – 4.6	Soft-Firm silty clay, grey, moist	S2 - 2.6 m w = 52% LL = 59 PL = 21
4.6 – 5.0	(Difficult excavating) Silty clay mixed with sand and gravel (Till)	S3 - 5.0 m

Notes: End of Test Pit @5.0 m. Seepage noted in base of test pit. Test pit left open for approx. 4 hours; 25 mm of water accumulation in test pit.

Test Pit TP12-10 **January 8, 2013** **Elevation 93.95**

Depth (m)	**Material Description**	**Samples**
0 – 0.1	Topsoil and organics	
0.1 – 1.4	Stiff brown silty clay, moist	S1 - 0.9 m Gravel: 0% Sand: 23% Silt: 49% Clay: 28%

Depth (m)	Material Description	Samples
		w = 24% LL = 32 PL = 17
1.4 – 2.7	Stiff brownish/grey clayey silt	S2 - 2.3 m w = 28%
2.7 – 5.2	Soft-Firm grey silty clay moist to very moist	S3 - 5.2 m w = 61%

Notes: End of Test Pit 5.2 m. No seepage noted in test pit during excavation.

Test Pit TP12-11 January 8, 2013 Elevation 93.56

Depth (m)	Material Description	Samples
0 – 0.15	Topsoil and organics	
0.15 – 2.1	Stiff brown silty clay, moist	S1 - 0.9 m w = 29%
2.1 – 3.7	Stiff brownish/grey clayey silt, moist	S2 - 2.4 m w = 27%
3.7 – 5.0	Soft-Firm grey silty clay	S3 - 4.9 m w = 58%

Notes: End of Test Pit @ 5.0 m. No seepage noted in test pit during excavation.

Appendix C

Oedometer Test Results

Golder
Associates

February 15, 2013

Project No. 13-1183-0016

1465-710

Chris Hendry
SPL Consultants Ltd.
146 Colonnade Road
Unit 17
Ottawa, Ontario
K2E 7Y1

GEOTECHNICAL LABORATORY TESTING

Dear Sir

This letter reports the results of laboratory testing carried out on the samples received at our office in Mississauga. The results of the tests are summarized in the attached table and figures.

The testing services reported herein have been performed in accordance with the indicated recognized standard, unless noted otherwise. This report is for the sole use of the designated client. This report constitutes a testing service only and does not represent any results interpretation or opinion regarding specification compliance or material suitability.

We trust that the results are sufficient for your current requirements. If you have any questions, please do not hesitate to call us.

GOLDER ASSOCIATES LTD.

Marijana Manojlovic

Marijana Manojlovic
Laboratory Manager

MM/lg

Golder Associates Ltd.
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Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America

CONSOLIDATION TEST SUMMARY

FIGURE

SAMPLE IDENTIFICATION

Project Number	13-1183-0016	Sample Number	5
Borehole Number	3	Sample Depth, m	4.57

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	1/20/2013		
Date Completed	2/06/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	15.74
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	9.23
Area, cm^2	31.68	Specific Gravity, measured	2.78
Volume, cm^3	59.84	Solids Height, cm	0.640
Water Content, %	70.45	Volume of Solids, cm^3	20.27
Wet Mass, g	96.05	Volume of Voids, cm^3	39.57
Dry Mass, g	56.35	Degree of Saturation, %	100.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.889	1.952	1.889				
5.97	1.889	1.952	1.889	1	7.56E-01	1.77E-05	1.31E-06
10.60	1.884	1.944	1.886	86	8.77E-03	5.72E-04	4.91E-07
20.62	1.878	1.935	1.881	470	1.60E-03	3.06E-04	4.79E-08
39.76	1.866	1.916	1.872	240	3.10E-03	3.40E-04	1.03E-07
78.62	1.838	1.873	1.852	194	3.75E-03	3.75E-04	1.38E-07
156.10	1.751	1.736	1.795	1215	5.62E-04	5.96E-04	3.28E-08
310.79	1.441	1.251	1.596	1009	5.35E-04	1.06E-03	5.57E-08
619.70	1.293	1.020	1.367	392	1.01E-03	2.53E-04	2.51E-08
1236.84	1.185	0.853	1.239	194	1.68E-03	9.20E-05	1.51E-08
2478.08	1.098	0.717	1.142	205	1.35E-03	3.71E-05	4.91E-09
1236.84	1.104	0.725	1.101				
310.79	1.136	0.775					
78.62	1.164	0.819					
20.62	1.190	0.860					
5.97	1.211	0.893					

Note:
k calculated using cv based on t_{90} values.
Specimen swelled under 6 kPa

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.21	Unit Weight, kN/m^3	19.14
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.40
Area, cm^2	31.68	Specific Gravity, measured	2.78
Volume, cm^3	38.37	Solids Height, cm	0.640
Water Content, %	32.90	Volume of Solids, cm ^{3}	20.27
Wet Mass, g	74.89	Volume of Voids, cm ^{3}	18.10
Dry Mass, g	56.35		

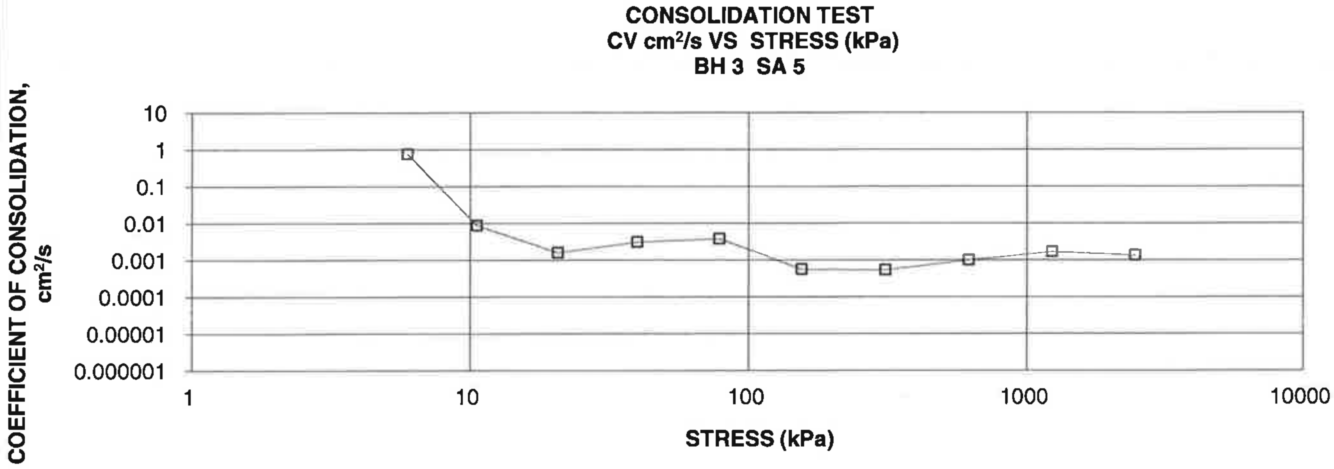
Prepared By: LH

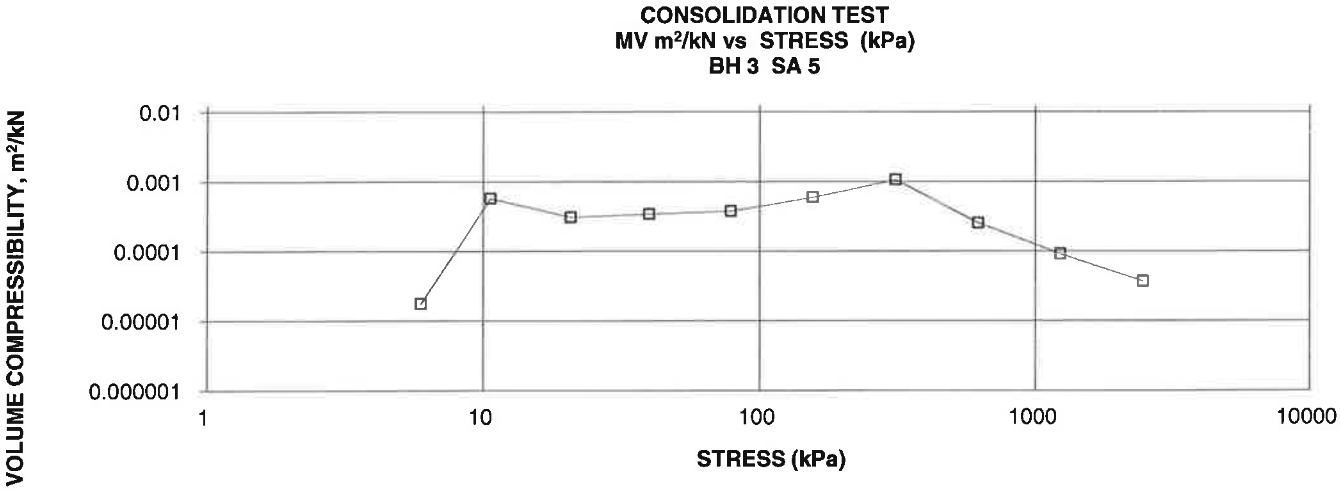
Golder Associates

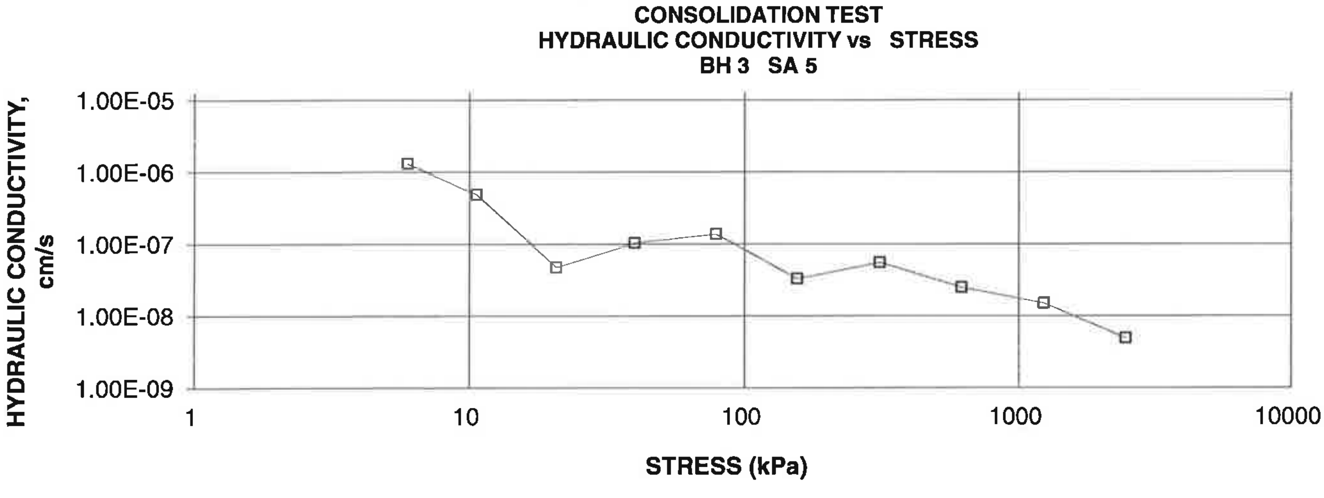
Checked By:

CONSOLIDATION TEST SUMMARY

FIGURE







Project No. 13-1183-0016

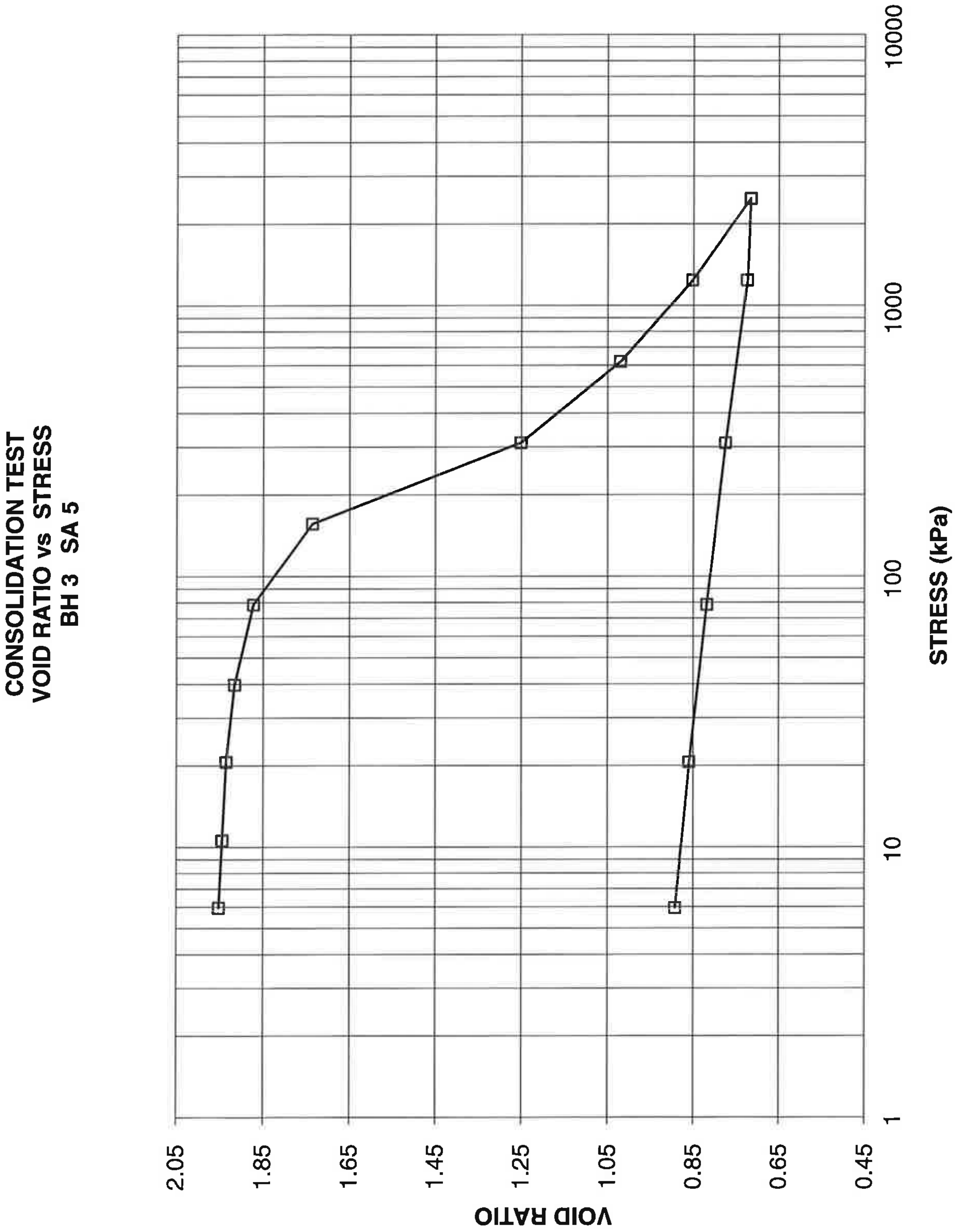
Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE



Project No. 13-1183-0016

Prepared By: LH

Golder Associates

Checked By:

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

PROJECT NUMBER	13-1183-0016
PROJECT NAME	SPL / Testing / 1465-710
DATE TESTED	January, 2013

Borehole No.	Sample No.	Specific Gravity
3	5	2.78

Note: Test carried out on soil particles <2.00mm using distilled water.

Checked By:

Golder Associates

Appendix D

Soil Corrosivity Test Results

Certificate of Analysis

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Daniel Wall
PO#: VISA
Invoice to: SPL Consultants Ltd.

Report Number: 1300776
Date Submitted: 2013-01-14
Date Reported: 2013-01-16
Project: 1465-710
COC #: 162091

Dear Daniel Wall:

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:

Digitally signed by Lorna Wilson
Date: 2013.01.16 14:48:04 -05'00'

APPROVAL: ____________________

Lorna Wilson
Inorganic Laboratory Supervisor

Exova (Ottawa) is certified and accredited for specific parameters by:
CALA, Canadian Association for Laboratory Accreditation (to ISO 17025), OMAF, 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.

Certificate of Analysis

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Daniel Wall
PO#: VISA
Invoice to: SPL Consultants Ltd.

Report Number: 1300776
Date Submitted: 2013-01-14
Date Reported: 2013-01-16
Project: 1465-710
COC #: 162091

Group	Analyte	MRL	Units	Guideline	1007118	1007119
				Lab I.D.	1007118	1007119
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2013-01-11	2013-01-11
				Sample I.D.	1465 - 710 BH1 S*2	1465 - 710 BH5 S*3
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.11	0.20
	pH	2.0			7.5	8.1
General Chemistry	Cl	0.002	%		0.003	0.003
	Resistivity	1	ohm-cm		9090	5000
	SO4	0.01	%		<0.01	<0.01

Group	Analyte	MRL	Units	Guideline	1007120
				Lab I.D.	1007120
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2013-01-08
				Sample I.D.	1465 - 710 TP8 S*3
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.20
	pH	2.0			8.7
General Chemistry	Cl	0.002	%		0.006
	Resistivity	1	ohm-cm		5000
	SO4	0.01	%		<0.01

Guideline = *** = Guideline Exceedence**
** = Analysis completed at 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.

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Daniel Wall
PO#: VISA
Invoice to: SPL Consultants Ltd.

Report Number: 1300776
Date Submitted: 2013-01-14
Date Reported: 2013-01-16
Project: 1465-710
COC #: 162091

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 244844 **Analysis Date** 2013-01-15 **Method** Ag Soil			
Electrical Conductivity	<0.05 mS/cm	92	80-120
pH		101	90-110
Resistivity			
SO4	<0.01 %	108	70-130
Run No 244847 **Analysis Date** 2013-01-15 **Method** C CSA A23.2-4B			
Cl	<0.002 %	103	90-110

Guideline = *** = Guideline Exceedence**
** = Analysis completed at 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.

Appendix E

Limitations of This Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to 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 testhole 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 testholes may differ from those encountered at the testhole 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 testhole 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 testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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

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