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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Mixed-Use Development
Mer Bleue Road
Ottawa (Cumberland), Ontario

Prepared For

Black Sheep Developments

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Report: PG4286-1

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Appendices:

1.0 Introduction

Paterson Group (Paterson) was commissioned by Black Sheep Developments to conduct a geotechnical investigation for a proposed mixed-use development, located on Mer Bleue Road, south of Vanguard Road, in Ottawa (Cumberland), Ontario (refer to Figure 1 - Key Plan presented in Appendix 3).

The terms of reference for the geotechnical investigation were outlined in File P9714-PRO.01R, dated September 13, 2017.

The objectives of the investigation were to determine the subsoil and groundwater conditions at the subject development site by means of boreholes and, based on the results of the test holes, to provide geotechnical recommendations pertaining to the proposed development.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

The subject development parcel is located on the east side of Mer Bleue Road, north of a hydro easement and south of Vanguard Road. The site is relatively flat, although somewhat hummocky. Much of the site was observed to have ponded water, perched on the ground surface, after significant precipitation after the drilling program and during the recording of groundwater levels.

Mixed-use development is proposed for this development parcel, consisting of basementless slab-on-grade commercial and/or office buildings, of undetermined height, an athletic building, and a potential seniors building. One objective of the current study is to determine the capability of the site with respect to foundation loads, so that the potential sizes of the structures can be determined. Parking areas, roadways and sewer and water supply services will also be required to service the subject development.

3.0 Method of Investigation

3.1 Field Investigation

The fieldwork program for the investigation was conducted on October 19, 2017, and consisted of the putting down of a total of six (6) boreholes to depths of between 2.8 and 8.9 metres. The boreholes are numbered BH 1 to BH 6, inclusive.

The locations of the boreholes are shown on Drawing No. PG4286-1, Test Hole Location Plan, included in Appendix 3.

The boreholes were put down using a track-mounted auger drill rig operated by a two-men crew. The fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. Sampling and testing the overburden was completed in general accordance with ASTM D5434-12 - Guide for Field Logging of Subsurface Explorations of Soil and Rock.

The drilling procedure consisted of augering to the required depths at the selected locations and sampling and testing the overburden soils.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler. Auger cuttings samples were recovered of surficial soils. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further review and testing. Transportation of the samples was completed in general accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soil Samples.

The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the applicable Soil Profile and Test Data (SPTD) sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the upper split-spoon sample in each borehole. Note that the deeper split-spoon samples were taken to recover samples of the soil for classification purposes, after conducting shear strength testing, so the SPT results are not representative for those samples. For these samples, “P”, for “push”, is recorded as the “N” value on the borehole logs.

The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

The current investigation included determining the thickness of the silty clay layer and the inferred bedrock depth in all the test holes. The inferred bedrock depth was evaluated, during the course the investigation, by extending the holes to auger refusal, or conducting dynamic cone penetration testing (DCPT) at BHs 1 and 2. Note that two (2) boreholes, labelled BH 19 and BH 20, were previously put down on the parcel, for another client, under Paterson File No. PG0713.

The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. The cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered in BH 1 and BH 2. The hammer was then used to confirm practical refusal. At present there is no ASTM standard for the DCPT.

Undrained shear strength testing was carried out in cohesive soils using both a small field vane, for high shear strength soils, and an MTO field vane apparatus. This testing was done in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report. The Soil Profile and Test Data sheets from the previous test holes are included after those for the current boreholes.

Groundwater

Flexible stand pipes were installed in all six (6) boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples from the current investigation will be stored in our laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The locations of the boreholes in the field were spotted by Paterson and were subsequently marked out in the field, and the ground surface elevation at the borehole locations were determined, by Atrel Engineering Limited (Atrel). It is understood that the elevations are referenced to Geodetic datum.

The locations of the boreholes and the ground surface elevation at each borehole location are presented on Drawing No. PG4286-1, Test Hole Location Plan, included in Appendix 3. The locations of the two (2) previous boreholes are also shown, with the ground surface measured at the time they were drilled.

3.3 Laboratory Testing

The soil samples recovered from the boreholes were examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject development property is a vacant undeveloped parcel. Review of historical air photos indicate there was a barn in the southwest corner of the parcel. The area to the south consists of a hydro easement with hydro towers. The area to the north consists of an industrial/commercial development fronting on Vantage Road.

The present ground surface at the six (6) boreholes, and including the previous two (2) boreholes ranges from 88.00 to 88.35 m, with a mean elevation of 88.20 m. The existing ground elevation is inferred to also be the original ground surface.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the test holes locations consists of topsoil deposits, overlying a deposit of sensitive silty clay. The silty clay is underlain by glacial till in several boreholes. The inferred bedrock surface is deeper within the south part of the site and becomes shallower toward the north.

Reference should be made to the Soil Profile and Test Data (SPTD) sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

Topsoil

At all the boreholes, an organic-rich topsoil layer was encountered of between 0.25 m (BH6 and BH20) and 0.36 m (BH2) in thickness.

Silty Clay

Silty clay was encountered directly beneath the topsoil at all borehole locations. The upper portion of the silty clay has been weathered to a brown to grey-brown desiccated “crust” at all test hole locations. The crust extends to between 2.5 and 3.5 m in the deeper silty clay area, and includes the entire silty clay thickness of 2.4 m in BH 4. In situ shear vane field tests carried out within the silty clay crust yielded peak undrained shear strength values in excess of 100 kPa, and all shear strengths in excess of 50 kPa, indicating a stiff to very stiff consistency in the silty clay crust.

Grey silty clay was encountered below the grey-brown silty clay crust in all boreholes, except for BH 4. In situ shear vane field testing conducted within the grey silty clay layer yielded undrained shear strength values generally ranging from 40 to 50 kPa. These values are indicative of a firm to stiff consistency. Perusal of the Soil Profile and Test Data sheets confirms that the thickness of the silty clay decreases significantly within the central and northern parts of the site.

Based on the results of extensive Atterberg Limits testing on the silty clay from investigations on adjacent parcels, it can be classified as a silty clay of high plasticity (CH).

Glacial Till

Within the central and northern parts of the site the silty clay is underlain by glacial till. The glacial till is a heterogeneous mixture of sand, silt, gravel and clay particles that can be described as a silty clay with sand, gravel, cobbles and boulders to a silty sand and gravel with cobbles and boulders.

The results of the Standard Penetration Testing indicates that the glacial till has a compaction condition within the range of loose to very dense.

Practical Refusal to DCPT and Auger Refusal

Practical refusal to DCPT penetration was observed in boreholes BH 1 and BH 2 at depths of 8.9 and 8.7 m, respectively. Practical refusal to augering was achieved in all the other boreholes. This information is shown on the applicable SPTDs, in Appendix 1, and is summarized in Table 1, on the following page.

Table 1: Summary of Inferred Bedrock Surface Levels

Borehole Number	Ground Elevation (m)	Inferred Bedrock Surface (m)		Nature of the Refusal
		Depth	Elevation	
BH 1	88.25	8.92	79.33	Practical Cone Refusal
BH 2	88.00	8.66	79.34	Practical Cone Refusal
BH 3	88.25	4.01	84.24	Practical Auger Refusal
BH 4	88.20	2.77	85.43	Practical Auger Refusal
BH 5	88.35	3.89	84.46	Practical Auger Refusal
BH 6	88.15	5.54	82.61	Practical Auger Refusal
BH 19 (2005)	88.26	4.65	83.61	Practical Auger Refusal
BH 20 (2005)	88.15	5.84	82.31	Practical Auger Refusal

Based on digital geological mapping produced by Natural Resources Canada, sourced from the Geological Survey of Canada, the bedrock in this area mostly consists of interbedded limestone and shale of the Lindsay formation.

4.3 Groundwater

The measured groundwater levels in the current boreholes are presented in Table 2, on the following page. The groundwater observations are also recorded on the applicable SPTDs, in Appendix 1.

The measured groundwater levels in the current boreholes are located at essentially the ground surface, but extensive ponded water was observed at the ground surface, at the time of the groundwater readings, that has influenced the standpipe installations. Relatively shallow groundwater levels were also encountered in the previous boreholes. Based on the depth of development of the stiff clay “crust”, the long-term low pre-development groundwater levels are expected to be about 0.5 m above the base of the crust, or approximately 2.0 to 3.0 m below the ground surface.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could be different at the time of construction.

Table 2: Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Remarks
		Depth	Elevation	
BH 1	88.25	0.00	88.25	Recorded November 8, 2017 GWL inferred to be influenced by extensive perched surface water
BH 2	88.00	0.00	88.00	
BH 3	88.25	0.37	87.88	
BH 4	88.20	0.15	88.05	
BH 5	88.35	0.53	87.82	
BH 6	88.15	0.54	87.61	
BH 19 (2005)	88.26	0.86	87.40	October 28, 2005
BH 20 (2005)	88.15	0.86	87.29	October 28, 2005

5.0 Discussion and Recommendations

5.1 Geotechnical Assessment

General Comments

The details of the development mix for this development have not been established at the time of this investigation, although several types of development are being considered. It can be assumed that the development may include the following:

- ☐ Several one to two storey steel-frame basementless slab-on-grade retail and/or office buildings are proposed, to be founded on routine footing foundations.
- ☐ A slab-on-grade athletic building, that may incorporate a high ceiling/roof and greater than normal roof span distances, is proposed.
- ☐ A multi-storey seniors or retirement residence is proposed that will likely include a full basement and will be a concrete frame structure.
- ☐ Parking areas, roadways and municipal services are required to service the subject development.

Generally, the subsoil conditions at the borehole locations vary across the site. It is recommended that the subsoil conditions at the site be considered in two halves, divided roughly in an east to west direction across the middle of the property.

The subsoil conditions at the two (2) more southerly boreholes, BH1 and BH2, consist of a topsoil layer overlying a thick sensitive silty clay deposit. The subsurface conditions are favourable for shallow foundation design and conventional light to medium basementless slab-on-grade commercial structures. Due to the presence of the sensitive silty clay layer, that part of the subject site will be subjected to grade raise restrictions. The subsoil conditions at the boreholes within the middle of the site will generally not govern the foundation design within the south half, where BH1 and BH2 can be used to establish the foundation design.

The subsoil conditions at the borehole locations are relatively similar from the middle of the site to the north. The silty clay layer is thinner, and is underlain by glacial till and the depth to inferred bedrock, at the depth of auger refusal, is shallower. The subsoil profiles are slightly weaker in the central part of the site and, therefore the boreholes within the middle part of the site will generally govern the foundation design for the north half of the site.

South Part of Property

Typical commercial structures, that are well-suited to the central to south part of the property are steel-frame, for light weight and basementless. Full basement structures can be built, if desired, but the location of the footings, lower in the soil profile, would result in low bearing resistance values for foundation design. Grade raise restrictions will be applicable and the lower the grade raise, the more strength will be available for the footing loads. Greater grade raises would reduce the capability of the site with respect to the foundation loads.

Two cases should be considered when determining the allowable bearing pressures for the design of shallow footings placed within the silty clay. Namely, the shear failure case and the settlement (serviceability) case. Paterson has estimated typical foundation loads for several configurations of steel-frame commercial building and, in conjunction with specified finished grading, has provided foundation design recommendations and bearing resistance values in Subsection 5.3 of this report.

North Part of Property

The subsoil conditions within the central to north part of the property consist of a thinner silty clay layer, that is of higher shear strength than the south part, and then a glacial till layer and relatively shallow inferred bedrock. As such, there is less impact from grade raises for structures founded on the silty clay and there is the alternative to found full basement structures on the glacial till or bedrock.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and existing non-specified fill materials should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the buildings, beside footings and foundation walls, and under the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II. This material should be tested and approved prior to delivery to the site. The granular fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and below the subgrade level of paved areas should be compacted to at least 95% of its standard Proctor maximum dry density (SPMDD), although stricter compaction requirements are applicable for fill directly below footings and within the base and subbase layers of pavements.

The zone of influence of a footing is considered to be the area beneath the footing limited sideways by planes extending out from the bottom edges of the footing, at a slope of 1.5H:1V, and down to the undisturbed in situ soil (below any fill or organic matter). Throughout the zones of influence of the footings, the engineered fill should consist of OPSS Granular A or Granular B Type II crushed stone materials.

The above-noted fill materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed within the zones of influence of the footings should be compacted to at least 98% of the material's SPMDD.

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern, such as soft landscaped areas. There is no specific compaction requirement in soft landscaped areas, but these materials should be spread in thin lifts and compacted/consolidated by the tracks of the spreading equipment to minimize voids.

If these non-specified fill materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided, or, as a minimum, an adfreezing bond break is provided.

5.3 Foundation Design

Limit States Design

The Ontario Building Code (OBC 2012) Part 4 commercial and/or residential structures that are proposed for the development will be founded on sensitive silty clay, glacial till or bedrock.

Limit States Design is the only design method permitted under Part 4 of OBC 2012. As such, the footing foundations for the structures are required to be designed for both the Bearing Resistance at Serviceability Limit States (SLS) and the Factored Bearing Resistance at Ultimate Limit States (ULS). The Bearing Resistance at SLS values pertain to the permissible (geotechnical) serviceability or deformation-related bearing resistance.

Unfactored foundation loads (i.e. service loads) are generally used in conjunction with the bearing resistance at SLS values. The exception is when the live and companion loads are reduced to represent “sustained loading” conditions when undertaking consolidation settlement analyses. Within the south part of this development, for the types of structures presently proposed, the bearing resistance at SLS will generally govern the foundation design for structures to be supported on footings.

The Factored Bearing Resistance at ULS pertains to the ultimate (geotechnical) capacity of the bearing medium, reduced by a geotechnical resistance factor. The geotechnical resistance factor is 0.5 for footing foundations. Factored loads are used in conjunction with the factored bearing resistance at ULS values.

Building Loads

Paterson has estimated the column loads of typical one (1) and two (2) storey basementless steel-frame structures, of various column spacings in order to provide the necessary information to evaluate the site capability. Typical column loadings are provided in Table 3, in Appendix 2.

Bearing Resistance Values for Basementless Buildings

Bearing Resistance Values for Silty Clay

Founding conditions at this site are favourable for the construction of the light to medium steel-frame basementless commercial structures that are expected to be constructed, provided that the grade raise is within an acceptable range. Based on the subsurface profile encountered, it is expected that stiff to very stiff silty clay will generally be encountered at the interior and exterior founding levels of conventional basementless slab-on structures, as described below.

The recommended grading, considering geotechnical constraints, is a maximum grade raise at the structures of 1.5± m, for a FFL (finished floor level) of 89.7 m or lower. The exterior/perimeter footing level for basementless slab-on-grade structures with routine soil cover would be set at 1.5± m below exterior finished grade, or approximately 1.7 m below the FFL, while the interior footing level would be set at least 1.0 m below the FFL. Depending on the amount of grade raise, footings may have to be founded on engineered granular fill over the silty clay.

Considering that the base of the stiff to very stiff “crust” is within an elevation range of 84.5 to 86.0 m, it is expected that stiff to very stiff silty clay will generally be encountered at the interior and exterior founding levels.

For basementless slab-on structures, the bearing resistance at SLS will generally govern the foundation design for structures to be supported on footings. Below the “crust” the compressibility and strength of the firm to soft grey silty clay will govern the bearing resistance values. Settlement considerations are discussed later in this section, although they have been used to establish the bearing resistance at SLS values.

Bearing resistance values are based on several variables, including location on the site (south or north half) the footing shape, the size of the footing (i.e. the wall or column load), and the USF (underside of footing) elevation, as each of these will affect the amount of footing stress that will be exerted on the firm grey silty clay. Representative bearing resistance at SLS values are tabulated in Tables 3A and 3B, for strip and square footings, respectively, founded on undisturbed silty clay bearing surfaces.

The factored bearing resistance at ULS values have been determined based on the location on the site and the relationship between the footing sizes and levels and the shear strength of the soil profile. Representative factored bearing resistance at ULS values are tabulated in Tables 4A and 4B, for strip and square footings, respectively.

Table 4A: Bearing Resistance at SLS Values
For Strip Footings on Silty Clay or Engineered Fill Over Silty Clay

Location on Site and Elevation of Underside of Footing (m)	Bearing Resistance (kPa) at SLS for Footing Width (m)			
	0.8	1.0	1.2	1.5
South Half of Development				
USF Elevation 87.5±m	150	150	135	115
USF Elevation 87.0±m	135	115	100	90
North Half of Development				
USF Elevation 87.5±m	150	150	150	140
USF Elevation 87.0±m	150	140	120	115

Table 4B: Bearing Resistance at SLS Values
For Square Footings on Silty Clay or Engineered Fill Over Silty Clay

Location on Site and Elevation of Underside of Footing (m)	Bearing Resistance (kPa) at SLS for Footing Size (m by m)				
	1.0 by 1.0	1.5 by 1.5	2.0 by 2.0	2.5 by 2.5	3.0 by 3.0
South Half of Development					
USF Elevation 87.5±m	180	160	120	100	90
USF Elevation 87.0±m	170	120	90	80	75
North Half of Development					
USF Elevation 87.5±m	180	180	150	130	110
USF Elevation 87.0±m	180	145	110	100	90

The above-tabulated bearing resistance at SLS values are for use with unfactored loads (D+L+S), and assume the sustained loads will be approximately represented by loads factored as D+0.5L+0.5S. The loads can be compared with applicable column loads provided in Table 3 in Appendix 2, for steel frame buildings, or for loads applicable to other basementless structures.

Table 5A: Factored Bearing Resistance at ULS
For Strip Footings on Silty Clay or Engineered Fill Over Silty Clay

Location on Site and Elevation of Underside of Footing (m)	**Factored Bearing Resistance (kPa) at ULS for Footing Width (m)**			
	0.8	**1.0**	**1.2**	**1.5**
South Half of Development				
USF Elevation 87.5±m	225	225	200	175
USF Elevation 87.0±m	210	180	150	140
North Half of Development				
USF Elevation 87.5±m	225	225	225	215
USF Elevation 87.0±m	225	210	185	175

Table 5B: Factored Bearing Resistance at ULS
For Square Footings on Silty Clay or Engineered Fill Over Silty Clay

Location on Site and Elevation of Underside of Footing (m)	**Factored Bearing Resistance (kPa) at ULS for Footing Size (m by m)**				
	1.0 by 1.0	**1.5 by 1.5**	**2.0 by 2.0**	**2.5 by 2.5**	**3.0 by 3.0**
South Half of Development					
USF Elevation 87.5±m	270	250	185	155	140
USF Elevation 87.0±m	260	180	140	120	115
North Half of Development					
USF Elevation 87.5±m	270	270	230	200	170
USF Elevation 87.0±m	270	215	170	150	140

The above-tabulated factored bearing resistance at ULS values incorporate a geotechnical resistance factor of 0.5 and can be used for the design of footings using factored loads, including the earthquake loading cases.

The above-tabulated bearing resistance values are provided on the assumption that the footings will be placed on **undisturbed in situ soil bearing surfaces**. An undisturbed in situ soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Bearing Resistance Values for Full-Basement Buildings

Full basement buildings are best suited to the north part of the development, where the bedrock is located at relatively shallow depth that is accessible as the bearing medium for footings, or as a subgrade medium for the placement of engineered granular fill to act as the bearing medium.

Bearing Resistance Values for Bedrock

A **clean bedrock bearing surface** can be taken to have a bearing resistance at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance at ultimate limit states (ULS) of **1250 kPa**. A geotechnical resistance factor of 0.5 has been incorporated into the ULS value.

A clean bedrock bearing should be free of all soil, fill and loose or deleterious materials, including severely fractured rock, and be observed and confirmed as such by qualified geotechnical personnel, prior to the placing of concrete.

A **clean surface-sounded bedrock bearing surface** can be taken to have a bearing resistance at SLS of **1,000 kPa** and a factored bearing resistance at ULS of **2,500 kPa**. A geotechnical resistance factor of 0.5 has been incorporated into the ULS value.

A clean, surface-sounded bedrock bearing surface should be free of all soil and loose or deleterious materials, and have no near surface mud seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer, and be observed and confirmed as such by qualified geotechnical personnel, prior to the placing of concrete.

Bearing Resistance Values for Engineered Granular B Type II

Footings can be founded on engineered granular fill. Where the engineered granular fill is placed over in situ soil, a **suitably prepared engineered Granular B Type II bearing surface** can be taken to have bearing resistance values similar to the values applicable for the subgrade material (i.e. the tabulated values).

Where engineered granular fill is placed over an undisturbed glacial till or clean bedrock subgrade, a **suitably prepared engineered Granular B Type II bearing surface** can be taken to have a bearing resistance at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 has been incorporated into the ULS value.

The engineered fill should consist of Ontario Provincial Standard Specifications (OPSS) Granular B Type II, placed over an undisturbed inorganic in situ soil or bedrock subgrade and compacted in thin lifts using suitable compaction equipment for the lift thickness. Fill placed beneath a building, including for footing bearing media and for slab-on-grade support should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD) value.

Bearing Surface Review Observations

On-site bearing surface review observations are recommended as part of the geotechnical field review. In order to confirm that the bearing medium is consistent with the design criteria and that the bearing surface preparation is consistent with our recommendations.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to stiff to firm silty clay and engineered granular fill when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered granular fill of the same or higher capacity as the bearing medium material.

Settlement/Grade Raise

In addition to the shear failure case, consideration should be given to potential post construction settlements when determining the bearing resistance at SLS (allowable bearing pressure) values. The potential settlements can occur due to the compression of the silty clay deposit under the loads from the footings, the grade raise fill pressures and groundwater lowering effects.

The foundation loads to be considered for settlement analyses are the continuously applied or “sustained” loads, which can be taken as the unfactored dead loads and a portion of the unfactored live loads. We have conservatively considered 50% of the live and snow loads as part of the continuously applied loads, for the structures anticipated at this site, to be founded on the silty clay.

To establish the bearing resistance at SLS values, it was assumed that the sustained footing loads are approximately 75% of the unfactored SLS footing loads applicable to the tabulated bearing resistance at SLS values in Tables 4A and 4B.

The analyses also were based on finished floor levels of up to 89.7 m, as compared to a mean original ground surface level of the order of 88.2 m. As such, a maximum permissible grade raise at the building floor slab, of 1.5 m has been considered.

The previous discussion is based on the assumption that tolerable total and differential settlements for the proposed steel-frame structures to be founded on the silty clay are 50 and 20 mm, respectively, recognizing the relatively flexible nature of these structures.

The SLS bearing resistance values provided for the bedrock assume that very conservative potential total settlements of 25 mm and/or differential settlement between footings, both founded on entirely on bedrock, at their appropriate SLS bearing resistance values, of 20 mm are tolerable/serviceable to the proposed structure.

If structures are founded entirely on engineered granular fill, the same settlement criteria as above will apply for subgrades of silty clay, or bedrock, respectively. If structures are founded partly on silty clay and partly on engineered granular fill there will be no concern for elevated differential settlements provided the thickness of the granular fill is not great. If structures are founded partly on bedrock and partly on engineered granular fill, then the differential settlement potential increases to the order of 25 mm between the low compressibility bedrock and the granular fill.

5.4 Recommended Review of Grading Plans

Paterson Group should be consulted to review grading plans to ensure the proposed site grading conforms to the intent of our geotechnical recommendations concerning grading. Our review of the grading plans will be based on an interpretation of the proposed grades and underside of footing levels to ensure our foundation design soil parameters for bearing and settlement (50 mm total and 20 mm differential) are valid for structures founded on the silty clay.

5.5 Design for Earthquakes

Analyses have been conducted (Table 6A and 6B, in Appendix 2) to determine the Site Class for seismic site response for the subject development parcel. Boreholes BH1 and BH4 were analysed for Tables 6A and 6B, respectively.

The Vs_{30} has been calculated for the overburden characteristics at each borehole using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012. For each borehole, the soils and bedrock portions of the profile were assigned representative shear wave velocity (Vs) values. In the case of the grey silty clay, the Vs value was estimated from a formula for Vs in Eastern Ontario clays tested by Hunter, Burns, et al of GSC.

The application of the OBC formula for BH1 is illustrated below for the applicable Vs values, as shown in Table 6A. The thickness of soil is measured from the base of the highest anticipated footing level (i.e. approximately the present ground surface). The average shear wave velocity of the upper 30 m profile, Vs_{30}, is calculated to be **426 m/s** at BH1. Therefore, **Site Class C** is applicable for the conditions at BH1, as per Table 4.1.8.4.A of the OBC 2012.

The application of the OBC formula for BH4, is shown in Table 6B. The average shear wave velocity of the upper 30 m profile, Vs_{30}, is calculated to be **1,242 m/s** at BH4. Therefore, **Site Class B** would normally be applicable for the conditions at BH4.

Please note that the use of **Site Class B** requires that the applicable structure be founded directly on the bedrock, and that the shear wave velocity be confirmed with site-specific testing, such as near-surface seismic reflection/refraction testing.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)} + \frac{Depth_{Layer3}(m)}{Vs_{Layer3}(m/s)} + \frac{Depth_{Layer4}(m)}{Vs_{Layer4}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\sum\left(\frac{2.5m}{200m/s} + \frac{6.4m}{130m/s} + \frac{1.0m}{1{,}500m/s} + \frac{20.1m}{2{,}500m/s}\right)}$$

$$V_{s30} = 426m/s$$

As such, it is our recommendation that **Site Class C** be used for foundation design for subject development unless further testing is conducted to determine the extent of the area that can be classified as **Site Class B**, for structures that would be founded on the bedrock.

The soils underlying the site are not susceptible to seismic liquefaction.

5.6 Slab-on-Grade Floor Slab

With the removal of all topsoil and any unspecified fill within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 150 to 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of their SPMDD.

5.7 Pavement Structures

For design purposes, the pavement structures presented in the following tables could be used for the design of car parking areas (Table 7) and access lanes/fire lanes with occasional truck traffic (Table 8).

Table 7: Recommended Pavement Structure Car Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** SP 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ silty clay or OPSS Granular B Type I or II material placed over in situ silty clay or suitably compacted service trench fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be subexcavated and replaced with OPSS Granular B Type I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, biaxial geogrid, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Table 8: Recommended Pavement Structure
Access Lanes/Fire Lanes - Occasional Truck Traffic

Thickness mm	Material Description
40	**Wear Course -** SP 12.5 Asphaltic Concrete
50	**Binder Course -** SP 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
375	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ silty clay or OPSS Granular B Type I or II material placed over in situ silty clay or suitably compacted service trench fill.	

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

It is recommended that the road structure granular layers be protected from surface water. Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

The pavement structure should maintain a suitable crown or drainage pattern to mirror the surface grading in order to shed water towards the available storm sewer catch basins. Consideration should be given to the placement of subdrains along the pavement edge for access roads, or "stubby" drains, leading into the catch basins at the subgrade level for parking areas.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for each of the proposed structures. The system should consist of a 100 mm to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation drainage is not a requirement for basementless structures. However, the provision of drainage helps with controlling potential frost action around the building (heaving of sidewalks, etc.) as well as reducing the potential for moisture through the perimeter of the interior floor slab.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials, such as clean sand or OPSS Granular B Type I material. The greater part of the site excavated materials will consist of frost susceptible fine-grained soils and, as such, are not recommended for re-use as backfill against the foundations unless a composite drainage system (such as system Platon or Miradrain 6000 or G100N), connected to a foundation drainage system, is provided.

For full-basement structures, consideration should be given to installing a subfloor drainage system under the basement floor slab, consisting of lines of 100 mm diameter perforated pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, with a positive connection to the storm sewer or a sump pit, to ensure that groundwater is adequately controlled.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers and wing walls may require more soil cover or a combination of soil cover and insulation. Details for foundation insulation can be provided upon request if the soil cover is insufficient and needs to be supplemented with insulation.

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through silty clay. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. The lowermost 1.2 m can be vertical provided the material consists of stiff in situ silty clay only. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Unsupported slopes in excess of 3 m in height, if required, should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

General Recommendations

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. Trench details should be as per Detail Drawing Nos. W17, S6 and S7. Further guidelines concerning trench excavations are provided later in this section.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. Where the invert of the excavation is within grey silty clay, or on bedrock, the thickness of the bedding should be increased to 300 mm. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

It should generally be possible to re-use the upper portion of the silty clay, and the glacial till, if applicable, above the cover material if excavation and filling operations are conducted in dry weather. Due to its high natural water content, the wet grey silty clay will be difficult, if not impractical, to compact without an extensive drying period. Native trench backfill that is difficult to compact may be placed and consolidated in layers using ramping techniques.

Where hard surface areas will be located above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Seepage Barriers - Clay Seals

In order to reduce the potential consolidation of the compressible clay deposit, it is very important that no long-term groundwater lowering occur. To prevent the granular pipe bedding and pipe cover from acting as a “french” drain, it is recommended that clay dykes or seals be installed along service trenches situated below the water table.

The clay seals should be as per Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. The seals should be at least 1.5 m long (in the trench direction), as compared to the 1 m minimum in the detail, and should extend from trench wall to trench wall.

Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations, but no more than 60 m intervals apart, in the service trenches.

Trench Support

The installation of the proposed sewers in soils can be carried out safely within the confines of a trench box or in an open cut. An open cut in overburden materials will require that all side slopes be cut back at 1H:1V or shallower to maintain stability. If a shoring system (i.e. trench box) is used to support the walls of the cut, the trench box design should, for safety purposes, allow for additional surcharge pressures associated with construction equipment and stockpiled fill materials above the cut, although stockpiling of materials above excavations is strongly discouraged, considering the presence of deep deposits of sensitive silty clay.

The interpretation of the soil descriptions in the Occupational Health and Safety Act, Regulations for Construction Projects, for purposes such as trench box design, should be undertaken by experienced geotechnical personnel. The information provided in the Soil Profile and Test Data sheets in Appendix 1 can be consulted for this purpose, with due consideration that the information is only accurate at the applicable test hole locations, and in consideration that the contractor's equipment and methods, as well as the depth of the excavation, can have a significant effect on the actual earth pressures in force at the time of construction.

Trench Backfilling Procedures

Native trench fill materials that have just been placed, unless they have been thoroughly compacted using padfoot compaction equipment (which is generally not the case) can be considered to be very weak. As such, the backfilling of the trenches with difficult to compact grey clay native fill materials should be accomplished by ramping with a small dozer or loader, working back and forth on a shallow ramp, placing the native fill in thin lifts.

It is recommended that the bedding and granular cover material be of a uniform density to provide optimum support to the pipe. Compaction of the bedding and cover materials will also enhance the stability of the trench during backfilling.

Improved performance of the pipe installation can be effected by using a combination of the following techniques:

❏ Employ benching techniques to reduce the effective trench depth.

❏ Properly support the excavator with plates and/or beams to distribute equipment surcharges beyond just the trench heading to the sides of the trench.

❏ Keep unsupported trench lengths as short as practical.

❏ Use the drier, upper, soils for trench backfilling and discard the wetter, lower, grey soils for use as general fill in landscaped areas, such as the boulevards. This will provide time for the wetter soil to dry out, while the better soil is used immediately in the trenches.

- ☐ Use layered ramping techniques, with a loader or dozer, supplemented with a padfoot compactor, to backfill the trench. Follow-up as close as practical with the backfilling to the completion of the pipe installation and cover and the moving of the trench box.
- ☐ Bulk up granular cover material against the sides of the trench box to provide granular material to fill the voids created by the walls of the trench box as it is moved forward.
- ☐ Take particular care when moving away from drainage structures, such as manholes, to backfill carefully behind the trench box with the excavator, as backfilling by ramping will not be practical.

Trench Dewatering

Low to moderate rates of groundwater flow into excavations below the water table should be expected in the silty clay. Where trenches extend to the glacial till and/or bedrock, groundwater influx rates could be greater. The contractor should be prepared to pump water from the excavation to enable the installation of the services to be carried out in the dry. It is expected that routine pumping from within the confines of the excavation will suffice where excavations are in silty clay. If onerous groundwater conditions develop, more elaborate dewatering may be required to deal with localized problems.

6.5 Groundwater Control

Due to the relatively low permeability of the silty clay material, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Where deep excavations are required, other dewatering means or cutoff barriers could be required.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

The developer may need to register with the Ontario Ministry of Environment and Climate Change's (MOECC's) Environmental Activity and Sector Registry (EASR) process for this project if more than 50,000 L/day (and less than 400,000 L/day) are to be pumped during the construction phase (routine flows). Paterson can assist with this process.

Pumping of more than 400,000 L/day requires a temporary MOECC permit to take water (PTTW). At lead time of 4 to 6 months should be allowed for completion of the application and the review and issuance of the permit by the MOECC.

6.6 Winter Construction

Precautions should be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and insulated tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of the analytical testing of a soil samples from this investigation, are provided in Appendix 2, and are summarized in Table 9, on the following page.

The results of analytical testing show that the sulphate content is less than 0.1% (1 mg/g). This results are indicative that Type GU (general use) cement, as per CSA A23.1 Section 4.2.1.1.2, is appropriate for this site.

The chloride content is less than 2000 mg/g and the pH of the sample is greater than 5. These results indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site. The resistivity values are less than 1,500 ohm-cm and are indicative of an aggressive to very aggressive corrosive environment.

Table 9: Corrosion Potential and Sulphate

Parameter	**Laboratory Results**	**Threshold For Concern**	**Commentary**
	BH6-SS3		
Chloride	590 µg/g	Chloride content more than 2000 mg/g	Negligible Concern
pH	7.53	pH value less than 5.0	Neutral Soil
Resistivity	43.9 ohm.m 4390 ohm.cm	Resistivity less than 1,500 ohm.cm	Moderate to Slightly Aggressive Corrosion of Metals
Sulphate	41 µg/g	Sulphate value more than 1 mg/g	Negligible Concern

The appropriate concrete exposure class is “N”, for soil contact based on chloride content, where freezing and thawing (F-1 or F-2 exposure class) is not an issue.

6.8 Landscaping Considerations

The subject site is located in an area of sensitive silty clay deposits for tree planting. The silty clay has a high plasticity, which makes it susceptible to drying shrinkage. The City of Ottawa’s standard separation distance of 7.5 m from the tree to the nearest foundation will be applicable unless the combination of tree species and mature tree height, foundation depth and/or bearing medium indicates a reduction is justified.

In our opinion, tree planting for this subject development should be limited to low to moderate water demand trees. Low water demand species include beech, birch, mulberry, cedar, fir, pine and spruce. Moderate water demand trees include ash, cherry, hawthorn, hornbeam, sugar and red maples, and mountain ash.

The minimum permissible distance from the foundation to the tree will depend on the nature of the tree, the depth of the clay crust and the final grade raise in relation to the permissible grade raise. In our opinion, with the use of low water demand trees of low to medium mature height, and with adequate volume provided for the root ball, the development should be provided with a tree to foundation clearance of 6.0 metres. In critical areas, the minimum permissible tree planting distance could be improved by installing various tree damage preventative measures such as:

☐ Exfiltration trenches with a moisture retention barrier
☐ Root barrier systems with water delivery systems
☐ Separation barriers
☐ Additional foundation reinforcement and support

It is well documented in the literature, and is our experience, that fast-growing (i.e. high water demand) trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Field Review and Materials Testing Services

The following field review and materials testing program should be performed by the geotechnical consultant:

☐ Geotechnical review of all grading plans, with particular consideration of the characteristics of the structures proposed.

☐ Observation of all bearing surfaces prior to the concreting of footings.

☐ Field review of the placement of foundation insulation, if and where required.

☐ Sampling and testing of the concrete and granular fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3.0 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling and follow-up field density tests to ensure that the specified level of compaction has been achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon demand, based on the completion of a satisfactory materials testing and field review program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We have reviewed a preliminary grading plan, and provided comments in this report.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes. A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The geotechnical recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by persons or entities other than Black Sheep Developments, or their agent(s), is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

AJ Tovell

Andrew J. Tovell, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Nov. 22-17
A. J. TOVELL
46812509
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

BH 1 to BH 6, Inclusive (Current Investigation)
BH 19 and BH 20 (2005 Investigation)

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Pop. Commercial Development - Mer Bleue Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Atrel Engineering Ltd.

REMARKS

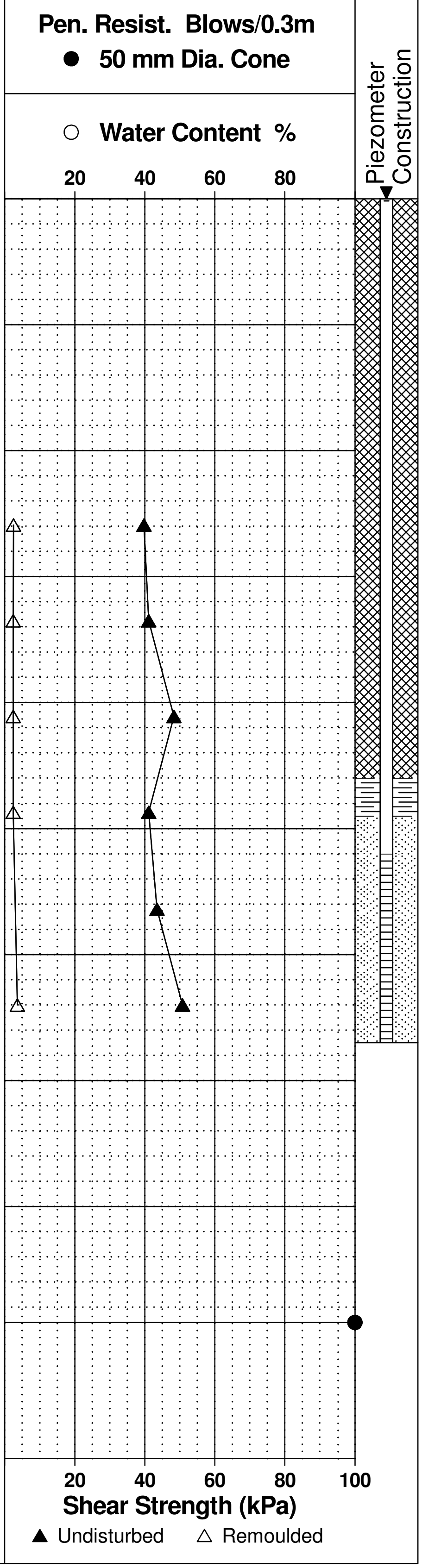
BORINGS BY CME 55 Power Auger

DATE October 19, 2017

FILE NO. PG4286

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	88.25
TOPSOIL 0.33	AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	100	10	1	87.25
	SS	3	100	5		
2.00					2	86.25
					3	85.25
	SS	4	100	P		
Firm to stiff, grey **SILTY CLAY**					4	84.25
	SS	5	100	P	5	83.25
					6	82.25
6.70	SS	6	100			
Dynamic Cone Penetration Test commenced at 6.70m depth. Cone pushed to 8.8m depth.					7	81.25
					8	80.25
8.92						
End of Borehole Practical DCPT refusal at 8.92m depth (GWL @ ground surface - Nov. 8, 2017)						



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Geotechnical Investigation
Pop. Commercial Development - Mer Bleue Road
Ottawa, Ontario

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REMARKS

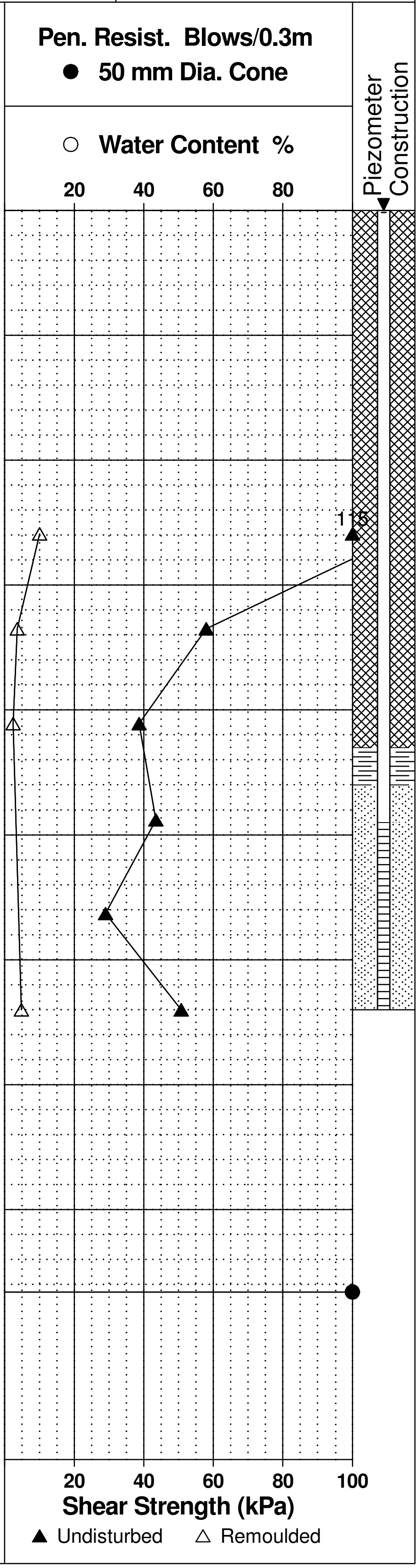
BORINGS BY CME 55 Power Auger

DATE October 19, 2017

FILE NO. PG4286

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.00
TOPSOIL 0.36		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	83	9	1	87.00
		SS	3	100	7	2	86.00
		SS	4	100	P	3	85.00
3.70							
Firm to stiff, grey **SILTY CLAY**		SS	5	100	P	4	84.00
						5	83.00
		SS	6	100	P	6	82.00
6.40							
Dynamic Cone Penetration Test commenced at 6.40m depth. Cone pushed to 8.5m depth.						7	81.00
						8	80.00
8.66							
End of Borehole Practical DCPT refusal at 8.66m depth (GWL @ ground surface- Nov. 8, 2017)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Pop. Commercial Development - Mer Bleue Road
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2017

FILE NO. PG4286

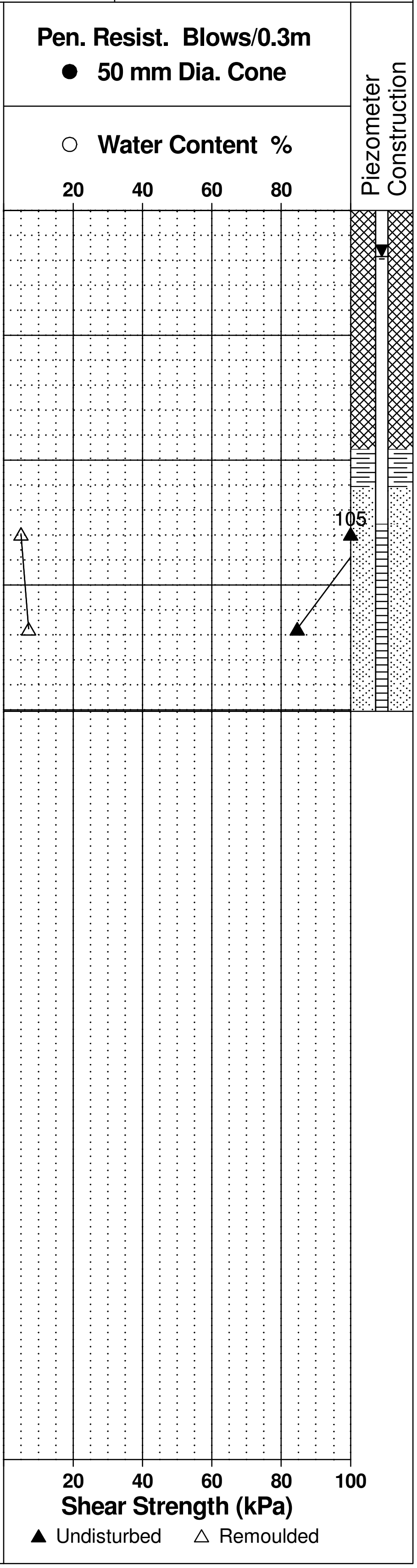
HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	88.25
TOPSOIL 0.28	AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	100	10	1	87.25
	SS	3	100	8	2	86.25
					3	85.25
3.58	SS	4	100			
GLACIAL TILL: Brown silty clay, trace sand, gravel, cobbles and boulders 4.01	SS	5	29	50+	4	84.25

End of Borehole

Practical refusal to augering at 4.01m depth

(GWL @ 0.37m - Nov. 8, 2017)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Pop. Commercial Development - Mer Bleue Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Atrel Engineering Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2017

FILE NO. **PG4286**

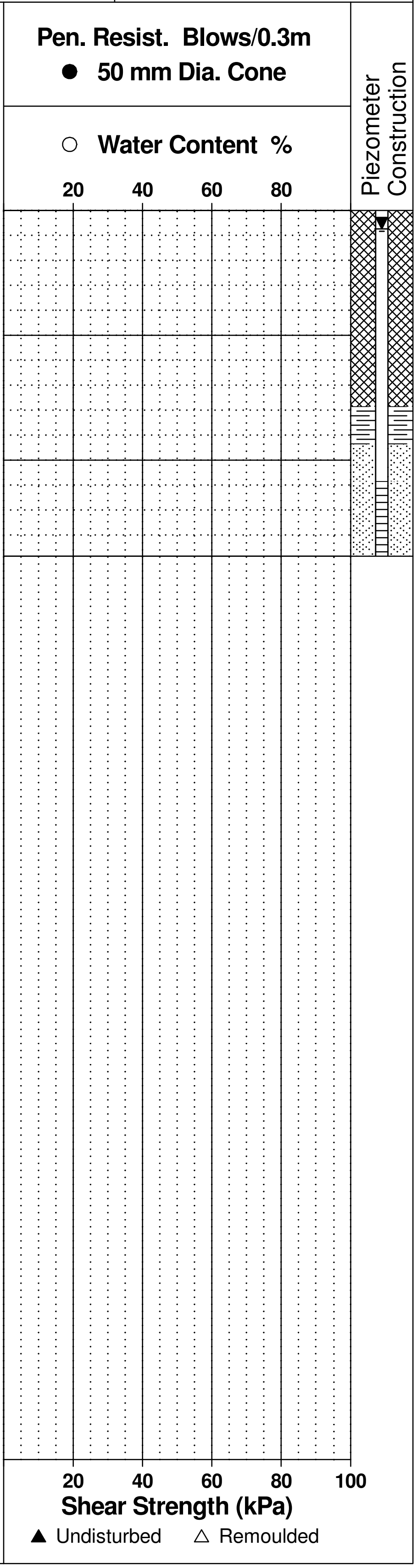
HOLE NO. **BH 4**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.20
TOPSOIL 0.33		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	100	9	1	87.20
		SS	3	100	7	2	86.20
2.44 **GLACIAL TILL:** Brown silty clay, trace sand, gravel, cobbles and boulders 2.77		SS	4		50+		

End of Borehole

Practical refusal to augering at 2.77m depth

(GWL @ 0.15m - Nov. 8, 2017)



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Ground surface elevations provided by Atrel Engineering Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2017

FILE NO. PG4286

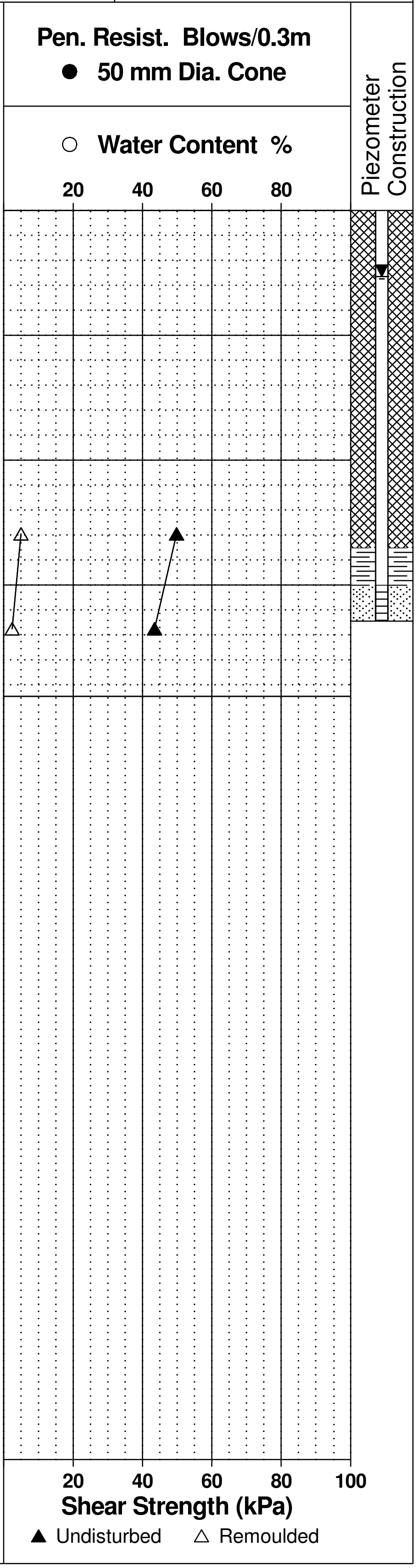
HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	88.35
TOPSOIL 0.28	AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	92	9	1	87.35
	SS	3	100	6	2	86.35
					3	85.35
3.35 **GLACIAL TILL:** Brown silty clay, trace sand, gravel, cobbles and boulders	SS	4	100	6		
3.89	SS	5	67	50+		

End of Borehole

Practical refusal to augering at 3.89m depth

(GWL @ 0.53m - Nov. 8, 2017)



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Ground surface elevations provided by Atrel Engineering Ltd.

FILE NO. PG4286

REMARKS

HOLE NO. BH 6

BORINGS BY CME 55 Power Auger

DATE October 19, 2017

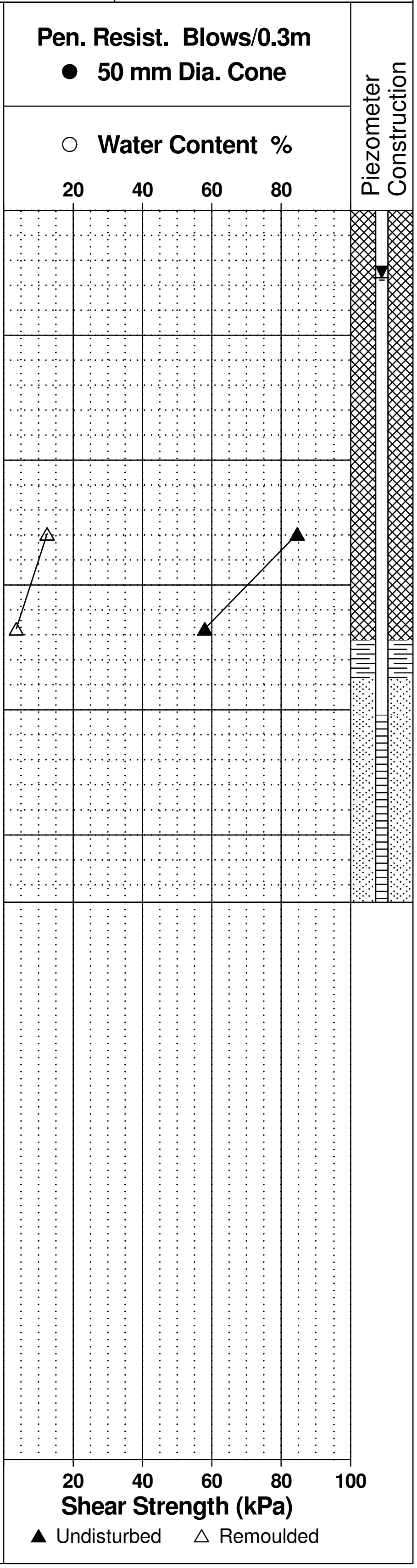
SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	100	10
		SS	3	100	9
	3.73	SS	4	100	3
GLACIAL TILL: Grey silty clay, some sand, gravel, cobbles and boulders		SS	5	100	W
		SS	4	46	4
	5.54	SS	5	67	50+

End of Borehole

Practical refusal to augering at 5.54m depth

(GWL @ 0.54m - Nov. 8, 2017)

DEPTH (m)	ELEV. (m)
0	88.15
1	87.15
2	86.15
3	85.15
4	84.15
5	83.15



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigtion
Proposed Development, Mer Bleue Road
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 20 Oct 05

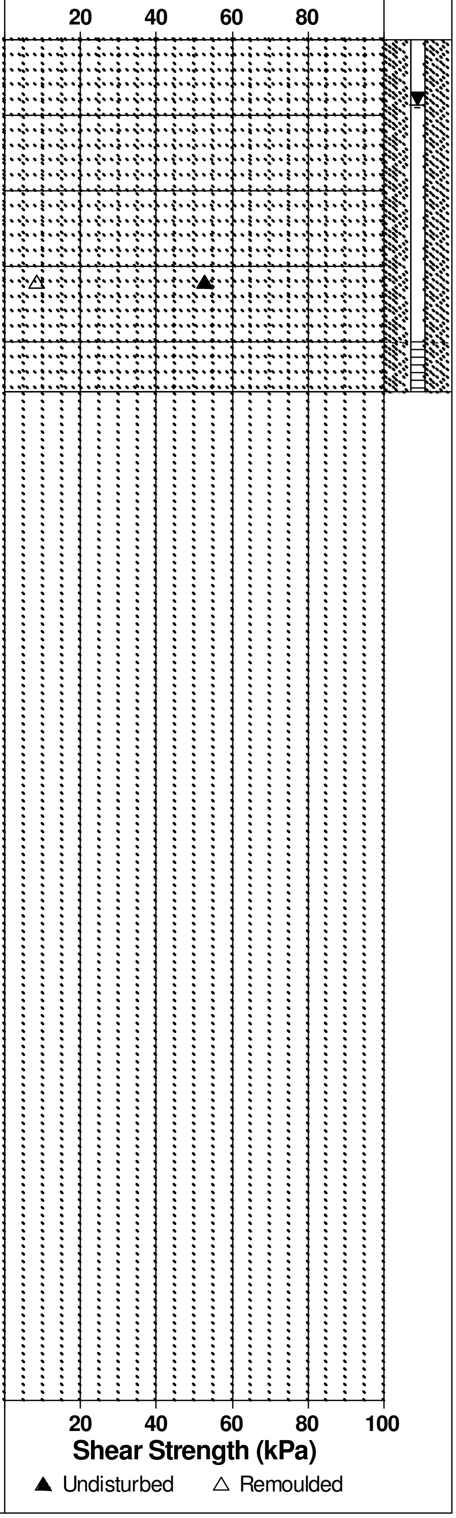
FILE NO. PG0713

HOLE NO. BH19

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE							0	88.26
TOPSOIL	0.28							
Brown **SILTY CLAY** with sand	0.76							
			SS	1	96	12	1	87.26
Very stiff to stiff, brown **SILTY CLAY**			SS	2	100	10	2	86.26
- stiff and grey by 2.9m depth	3.35		SS	3	100	5	3	85.26
GLACIAL TILL: Loose, grey silty sand with clay, gravel, cobbles and boulders			SS	4	50	9	4	84.26
	4.65		SS	5	0			
End of Borehole Practical refusal to augering @ 4.65m depth (GWL @ 0.86m-Oct. 28/05)								

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigtion
Proposed Development, Mer Bleue Road
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 20 Oct 05

FILE NO. PG0713

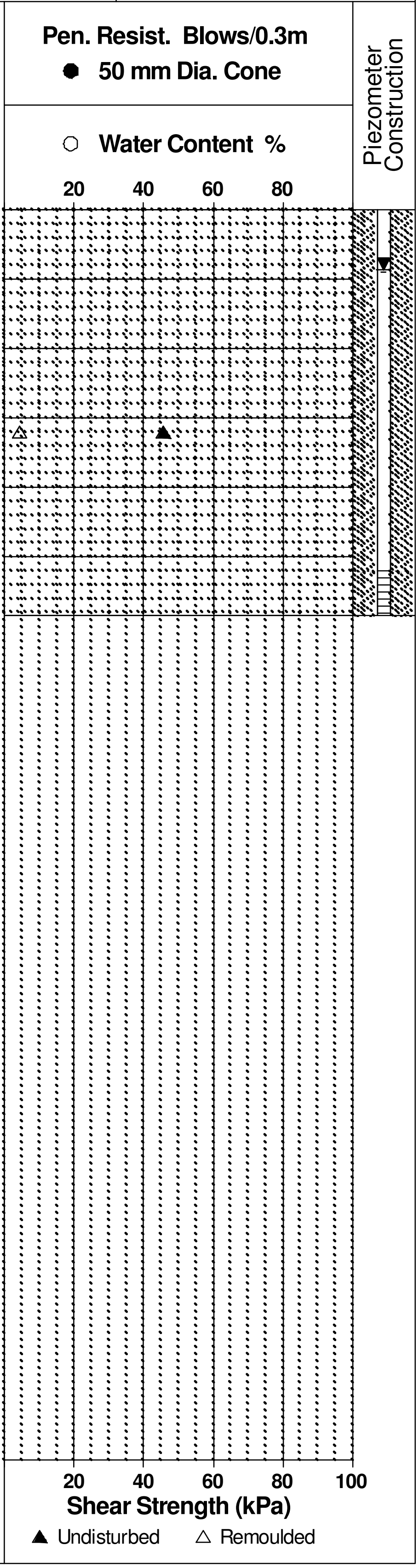
HOLE NO. BH20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.15
TOPSOIL 0.25							
Very stiff to firm, brown **SILTY CLAY**		SS	1	75	20	1	87.15
		SS	2	100	9	2	86.15
		SS	3	100	3	3	85.15
3.66							
GLACIAL TILL: Loose to dense, grey silty sand with gravel, cobbles and boulders - with some clay in upper 0.25m		SS	4	17	7	4	84.15
		SS	5	12	6	5	83.15
5.84		SS	6	10	56+		

End of Borehole

Practical refusal to augering @ 5.84m depth

(GWL @ 0.86m-Oct. 28/05)



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

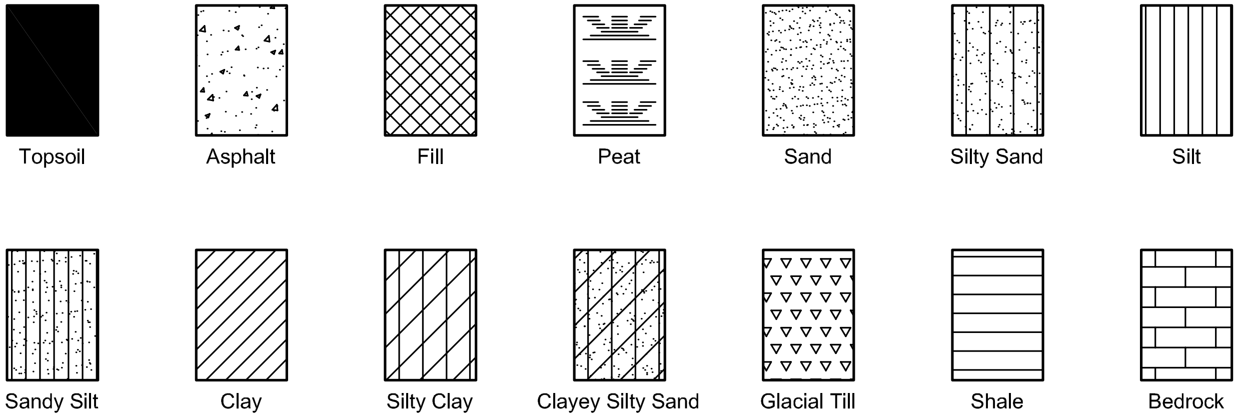
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

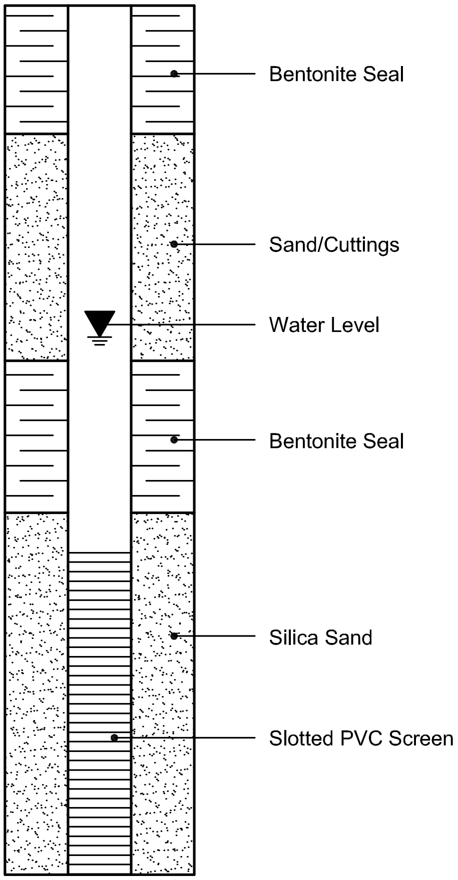
SYMBOLS AND TERMS (continued)

STRATA PLOT

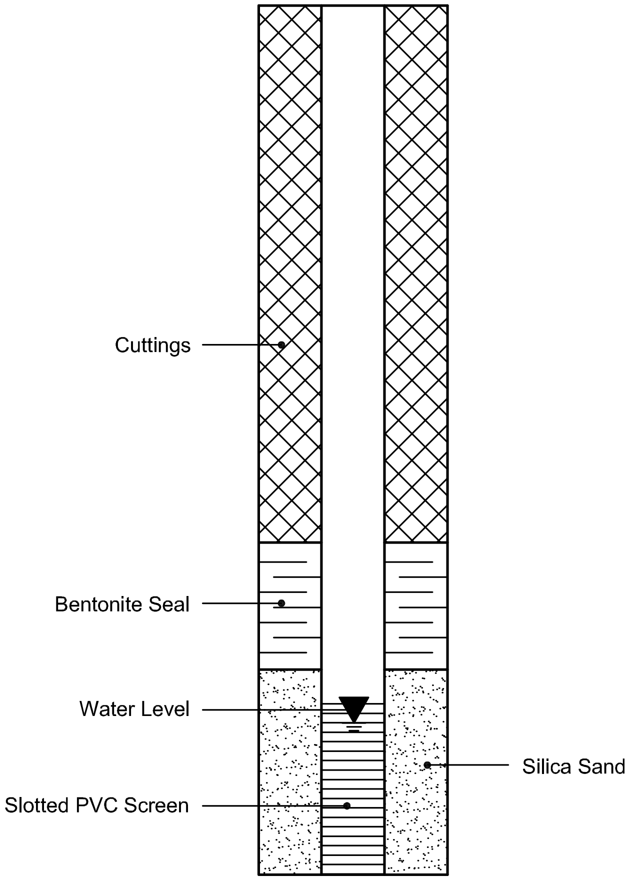


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

ANALYTICAL TEST RESULTS

TABLE 3: COLUMN LOADS FOR STEEL-FRAME STRUCTURES

TABLES 6A AND 6B: SEISMIC SITE CLASS - OBC 2012

	Client ID:	BH6-SS3	-	-	-
	Sample Date:	19-Oct-17	-	-	-
	Sample ID:	1743249-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.53	-	-	-
Resistivity	0.10 Ohm.m	43.9	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	41	-	-	-

TABLE 3:
TYPICAL COLUMN LOADS FOR STEEL-FRAME BUILDING CONFIGURATIONS
(FOR GENERAL PLANNING PURPOSES ONLY)
Mixed-Use Development - Mer Bleue Road, Ottawa (Cumberland)

Load Combination ▾ for Column Grid ▸	**1 Storey Building**			**2 Storey Building (Upper Level Restaurant)**		
	6 m x 6 m	**6 m x 7.5 m**	**6 m x 9 m**	**6 m x 6 m**	**6 m x 7.5 m**	**6 m x 9 m**
Breakdown Int. Col's:						
Dead Load, D	187	234	281	187	234	281
Snow Load, S	84	105	125	84	105	125
Live Load, L	on slab-on-grade	on slab-on-grade	on slab-on-grade	173	217	260
Interior Columns:						
D + S + L	271	339	406	444	555	666
D + 0.5L + 0.5S	229	287	344	316	395	474
1.25D + 1.5L + 0.5S	276	345	414	535	670	804
Exterior Columns:						
D + S + L	145	181	217	231	289	347
D + 0.5L + 0.5S	125	155	187	169	210	251
1.25D + 1.5L + 0.5S	149	185	222	279	349	419

Notes:
1. Sustained loading for purposes of settlement analyses uses the load combination of D + 0.5L + 0.5S.
2. The load combination for comparison to the Factored Bearing Resistance at ULS value is 1.25D + 1.5L + 0.5S.
3. The net footing and pier weight have not been included and should be added as appropriate.
4. The load combination incorporating earthquake (seismic) loading has not been included for this evaluation.

Table 6A: BH1 - Seismic Site Class - OBC 2012

Project:	Mixed-Use Development - Mer Bleue Road - Black Sheep Developments				
File No:	PG4286-REP.01		**Date:**	22-Nov-17	
PGA	0.32		**Region:**	Ottawa (Cumberland)	

Layer Description For BH1 (2005)	**Layer Properties**		**Cumulative**	**Thickness**
	Vs	**Thickness**	**Thickness**	**Vs**
silty clay crust	200	2.5	2.5	0.0125
grey silty clay (Vs by eqn)	130.2	6.4	8.9	0.0492
Vs = 125 + 1.1667*Z				
compact sand	250	0.0	8.9	0.0000
post-glacial clay	200	0.0	8.9	0.0000
glacial till	400	0.0	8.9	0.0000
weak or weathered bedrock	1500	1.0	9.9	0.0007
sound bedrock	2500	20.1	30.0	0.0080
Totals	**N/A**	**30.0**	**N/A**	**0.0704**
Average Shear Wave Velocity =				**426.3**
Site Class for Seismic Response =			**Class**	**C**

Site Class	**Description**	**Vs Min.**	**Vs Max.**	**N60 Range**	**Cu Range**
A	Hard rock	1500	>1500	N/A	N/A
B	Rock	760	1500	N/A	N/A
C	Soft rk VD soil	360	760	N>50	Cu>100
D	Stiff soil	180	360	15<N<50	50<Cu<100
E	Soft soil	0	180	N<15	Cu<50

Table 6B: BH4 - Seismic Site Class - OBC 2012

Project:	Mixed-Use Development - Mer Bleue Road - Black Sheep Developments		
File No:	PG4286-REP.01	**Date:**	22-Nov-17
PGA	0.32	**Region:**	Ottawa (Cumberland)

Layer Description For BH1 (2005)	**Layer Properties**		**Cumulative**	**Thickness**
	Vs	**Thickness**	**Thickness**	**Vs**
silty clay crust	200	2.4	2.4	0.0120
grey silty clay (Vs by eqn)	126.4	0.0	2.4	0.0000
Vs = 125 + 1.1667*Z				
compact sand	250	0.0	2.4	0.0000
post-glacial clay	200	0.0	2.4	0.0000
glacial till	400	0.4	2.8	0.0010
weak or weathered bedrock	1500	1.0	3.8	0.0007
sound bedrock	2500	26.2	30.0	0.0105
Totals	**N/A**	**30.0**	**N/A**	**0.0241**
Average Shear Wave Velocity =				**1242.4**
Site Class for Seismic Response =			**Class**	**B**

Site Class	**Description**	**Vs Min.**	**Vs Max.**	**N60 Range**	**Cu Range**
A	Hard rock	1500	>1500	N/A	N/A
B	Rock	760	1500	N/A	N/A
C	Soft rk VD soil	360	760	N>50	Cu>100
D	Stiff soil	180	360	15<N<50	50<Cu<100
E	Soft soil	0	180	N<15	Cu<50

APPENDIX 3

FIGURE 1: KEY PLAN

DRAWING PG4286-1: TEST HOLE LOCATION PLAN

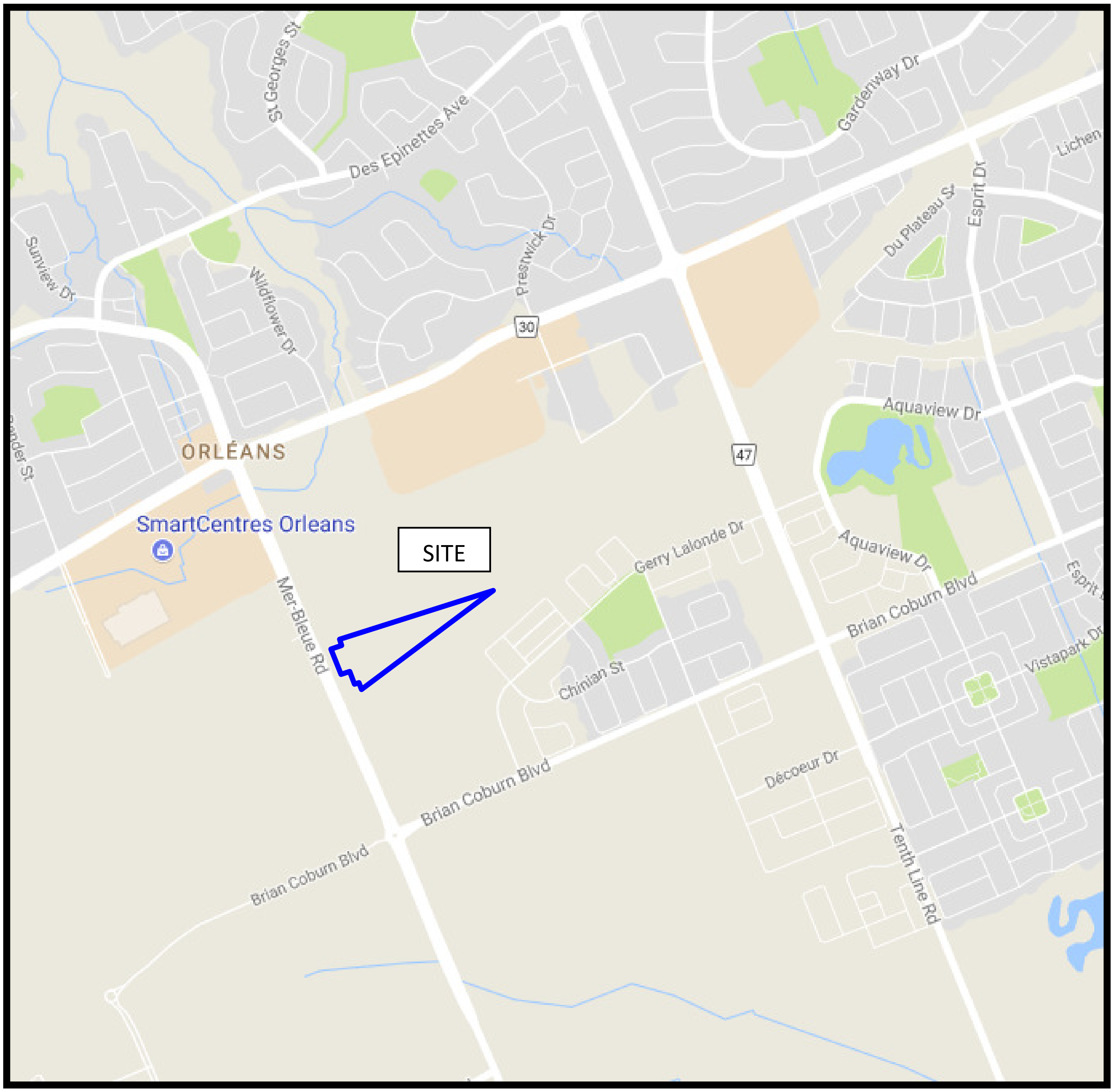
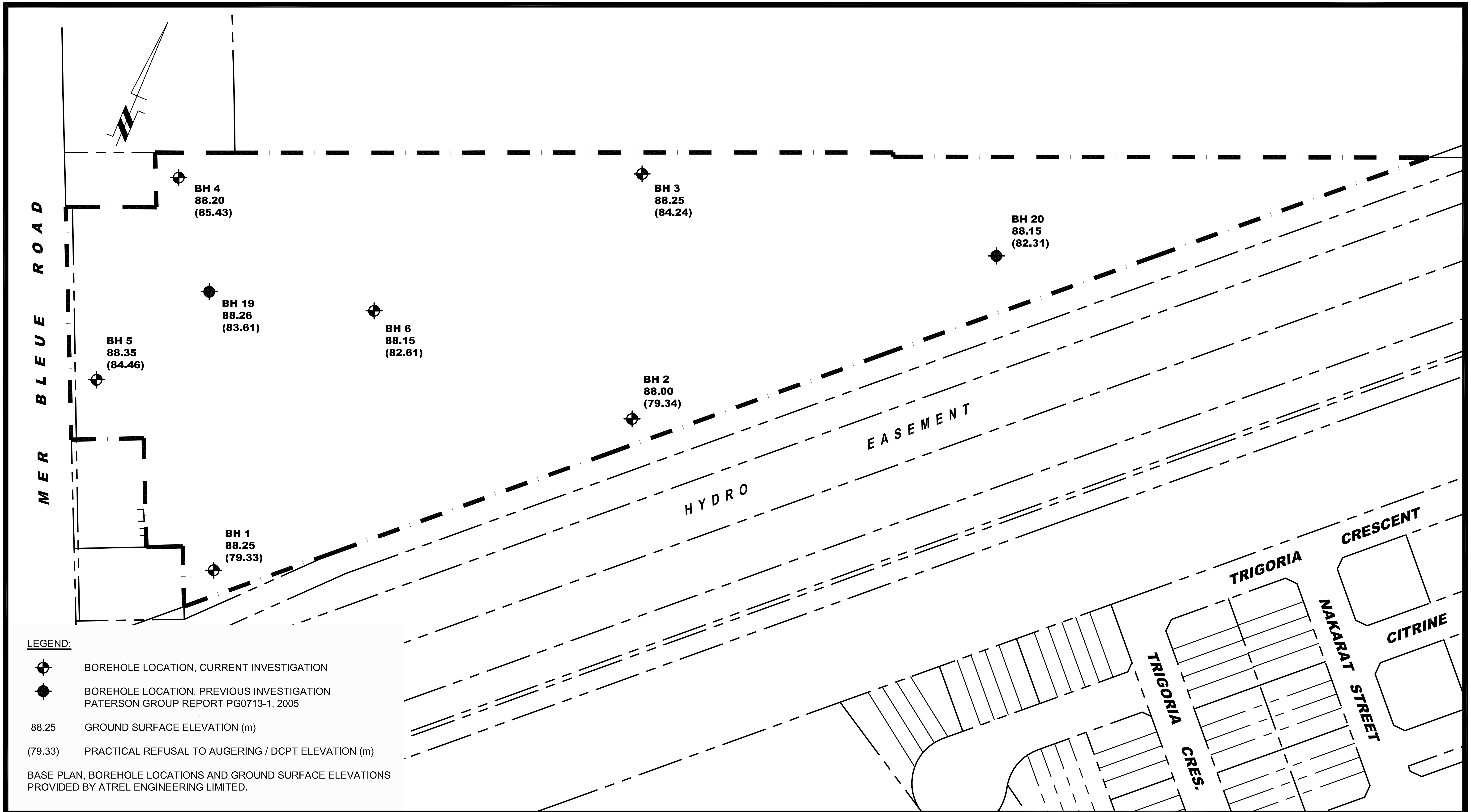


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
●	BOREHOLE LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG0713-1, 2005
88.25	GROUND SURFACE ELEVATION (m)
(79.33)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

BASE PLAN, BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ATREL ENGINEERING LIMITED.

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consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

BLACK SHEEP DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROP. MIXED-USE DEVELOPMENT - MER BLEUE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500
Date: 11/2017
Drawn by: MPG
Report No.: PG4286-1
Checked by: AJT
Dwg. No.: **PG4286-1**
Approved by: AJT
Revision No.: 0

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