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Geotechnical Investigation

Proposed Mixed-Use Development
2140 Baseline Road
Ottawa, Ontario

Prepared For

Theberge Homes

July 18, 2017

Report PG4184-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Theberge Homes to complete a geotechnical investigation for the proposed mixed-use development to be located at 2140 Baseline Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ☐ Determine the subsurface soil and groundwater conditions by means of boreholes.
- ☐ Provide geotechnical recommendations for the design of the proposed building including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. This report contains the findings and recommendations pertaining to the design and construction of the subject development as 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 for this geotechnical investigation.

2.0 Proposed Development

The proposed development is understood to consist of two multi-storey mixed-use buildings with one level of underground parking. At grade parking areas, access lanes and landscaped areas are also anticipated for the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on June 22, 2017. At that time, 3 boreholes were drilled to a maximum depth of 11.8 m below existing ground surface. The test hole locations were selected in a manner to provide general coverage for the proposed building footprints. A previous study was completed by others for the subject site in 2016. The locations of the boreholes are shown on Drawing PG4184-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled with a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to the laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets and 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden soil thickness was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at all borehole locations. 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 depth increment.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Flexible polyethylene standpipes were installed in the majority of the boreholes to permit the monitoring of groundwater levels subsequent to the completion of the field program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report unless we are otherwise directed.

3.2 Field Survey

The boreholes completed during the field investigation were selected in the field and surveyed by Paterson personnel. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of a fire hydrant located near the southwest corner of Constellation Drive and Gemini Way. An arbitrary elevation of 100.00 m was assigned to the TBM. The locations of the boreholes and the ground surface elevation at each borehole location are presented on Drawing PG4184-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in the laboratory to review the field log results.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample is analyzed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the sample. The results are included in Appendix 1 and are further discussed in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is located at the southwest corner of Baseline Road and Constellation Drive. The site is currently vacant and grass covered with mature trees located in the northeast corner of the property. The site is relatively flat and at a slightly higher grade than Baseline Road, Constellation Drive and Gemini Way. A former roadway crossed the site from the northwest to southeast corners of the site. Based on aerial photographs, the former roadway alignment within the subject site was removed in 2009.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of a topsoil layer underlain by a silty sand to silty clay fill mixed with varying amounts of gravel and cobbles extending to a maximum depth of 3 m. The fill layer is underlain by a native silty clay deposit. The silty clay deposit consists of a stiff to hard brown silty clay crust overlying a firm to stiff grey silty clay. A glacial till layer was encountered below the silty clay deposit in BH 1-17 and BH 3-17. Practical refusal to DCPT was encountered at all borehole locations ranging between 11.3 and 12.7 m depth below existing ground surface. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Rockliffe Formation with an overburden thickness ranging between 10 and 15 m depth.

4.3 Groundwater

Groundwater levels were measured in the boreholes on June 30, 2017. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1. Long-term groundwater levels can also be estimated based on the observed colouring, moisture levels and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected between 4 to 5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface Elevation (m)	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 1-17	99.15	4.72	94.43	June 30, 2017
BH 2-17	99.32	4.92	94.40	June 30, 2017
BH 3-17	99.50	5.08	94.42	June 30, 2017

Note: Ground surface elevations are referenced to a temporary benchmark (TBM). An arbitrary elevation of 100.00 m was assigned to the TBM

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered satisfactory from a geotechnical perspective for the proposed mixed-use development. It is anticipated that the proposed buildings will be founded by end bearing piles extending to the bedrock surface or a raft foundation placed on the undisturbed, silty clay bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from the building footprint, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill placed for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls for frost heave sensitive areas due to frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material 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 beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Raft Foundation

If the above bearing resistance values are insufficient for the proposed building, consideration may be given to placing the proposed building on a raft foundation.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively. It is expected that the base of the slab is located at or below 4 m depth, the long term groundwater level will be at or below 4 m depth, the raft slab is impervious and the basement walls will be provided with a perimeter foundation drainage system.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **225 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **375 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **6 MPa** for a contact pressure of **225 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Piled Foundation

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of 2 to 4 piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

Conventional Shallow Footings

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed over an undisturbed, hard to stiff silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **175 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings. The bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structured is required to provide adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the soil subgrade medium 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 fill of the same or higher capacity as the soil.

Permissible Grade Raise

A permissible grade raise restriction has been determined for the subject site based on the undrained shear strength values completed within the silty clay deposit. Based on the testing results, a permissible grade raise restriction of **1.5 m** above existing ground surface is recommended for the subject site.

To reduce potential long term liabilities, consideration should be given to providing means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the settlement sensitive structures, etc.). It should be noted that building over silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking as compared to unreinforced foundations.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered. Soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and/or fill, containing significant amounts of organic or deleterious materials, within the footprint of the proposed buildings, the native soil will be considered to be an acceptable subgrade surface 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 II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone below the basement floor slab.

All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compacted to, at least, 98% of the material's SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be incorporated to the diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be calculated with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) could be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Minimum Pavement Structure

For design purposes, the pavement structures presented in the following tables could be used for the design of car only parking areas and access lanes.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

Thickness mm	Material Description
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 4 - Recommended Pavement Structure - Access Lanes and Fire Route and Heavy Duty Asphalt Areas

Thickness mm	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or 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 excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dy condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fin subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the subgrade materials, consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended to be provided for the proposed buildings. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls. A drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system with a positive outlet to the site storm sewer is also recommended. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Under floor Drainage

It is anticipated that underfloor drainage will be required to control water accumulation during spring melt and after heavy rain events due to the low permeability of the underlying silty clay subgrade. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration/accumulation can be better assessed.

Concrete Sidewalks Adjacent to Building(s)

To avoid differential settlements within the proposed sidewalks adjacent to the proposed buildings, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks to consist of free draining, non-frost susceptible material such as, Granular A or Granular B Type II, instead of site excavated material which in most cases considered frost susceptible. The granular material should be placed in maximum 300 mm loose lifts and compacted to 95% of the material’s SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the building’s perimeter drainage pipe.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.
The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the material’s SPMDD.

Where hard surface areas are considered 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 reduce the potential 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 SPMDD.

To reduce long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long 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 compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at stratigic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

A temporary Ministry of the Environment and Climate Change (MOECC) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

Long-term Groundwater Control

Any groundwater encountered along the building’s perimeter or sub-slab drainage system will be directed to the proposed building’s sump pit. It is expected that groundwater flow will be low (i.e.- less than 30,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsurface conditions 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 installation of straw, propane heaters and tarpaulins or other suitable means. 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 and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a highly aggressive corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a materials testing and observation services program is required to be completed. The following aspects be performed by the geotechnical consultant:

❑ Observation of all bearing surfaces prior to the placement of concrete.

❑ Sampling and testing of the concrete and fill materials.

❑ Observation of the placement of the foundation insulation, if applicable.

❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❑ Observation of all subgrades prior to backfilling.

❑ Field density tests to determine the level of compaction achieved.

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

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

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, Paterson requests immediate notification to permit reassessment of our recommendations.

The 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 latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 person(s) other than Theberge Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 2140 Baseline Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the corner of Constellation Drive and Gemini Way. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 22, 2017

FILE NO. PG4184

HOLE NO. BH 1-17

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.10
FILL: Brown silty sand, trace gravel	1.07
FILL: Brown silty clay, some sand, gravel and cobbles	1.37
Hard to stiff, brown **SILTY CLAY,** trace sand	5.03
Grey **SILTY SAND,** trace seashell	5.33
Firm to stiff, grey **SILTY CLAY,** trace sand	9.75
GLACIAL TILL: Grey silty clay, some sand, trace gravel	10.36
Dynamic Cone Penetration Test commenced at 10.36m depth. Inferred **GLACIAL TILL**	11.30

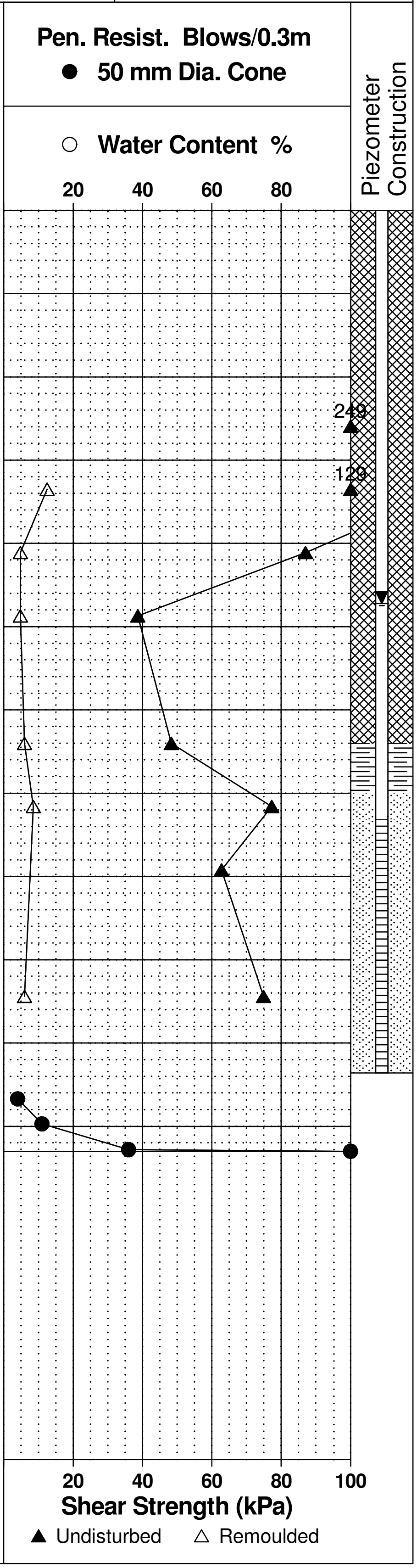
End of Borehole

Practical DCPT refusal at 11.30m depth

(GWL @ 4.72m - June 29, 2017)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	13
SS	3	100	7
SS	4	100	6
SS	5	100	P
SS	6	100	P
SS	7	67	P
SS	8	100	W
SS	9	100	P
SS	10	75	P
SS	11	12	P
SS	12	0	P
SS	13	100	P
SS	14	67	P

DEPTH (m)	ELEV. (m)
0	99.15
1	98.15
2	97.15
3	96.15
4	95.15
5	94.15
6	93.15
7	92.15
8	91.15
9	90.15
10	89.15
11	88.15



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 2140 Baseline Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the corner of Constellation Drive and Gemini Way. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 22, 2017

FILE NO. PG4184

HOLE NO. BH 2-17

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.13
FILL: Crushed stone with silt	0.60
FILL: Brown silty sand, trace clay	0.89
Hard to very stiff, brown **SILTY CLAY,** trace sand	3.81
Brown **SILTY SAND,** trace clay	4.27
Very stiff to stiff, grey **SILTY CLAY,** trace sand	10.36
Dynamic Cone Penetration Test commenced at 10.36m depth	12.60

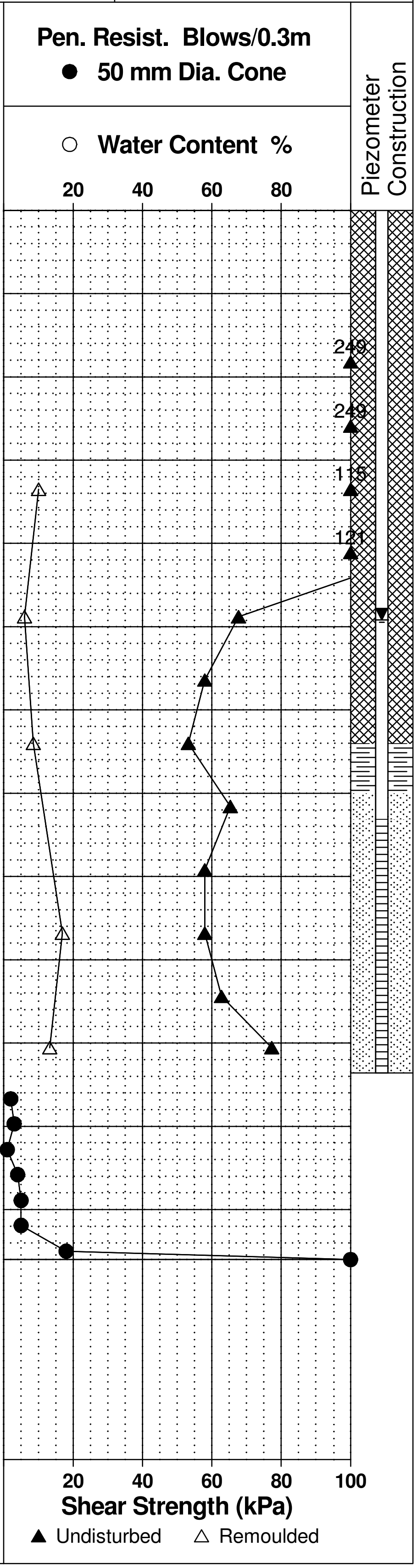
End of Borehole

Practical DCPT refusal at 12.60m depth

(GWL @ 4.92m - June 29, 2017)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	7
SS	3	100	10
SS	4	100	P
SS	5	100	P
SS	6	100	P
SS	7	100	P
SS	8	100	P

DEPTH (m)	ELEV. (m)
0	99.32
1	98.32
2	97.32
3	96.32
4	95.32
5	94.32
6	93.32
7	92.32
8	91.32
9	90.32
10	89.32
11	88.32
12	87.32



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 2140 Baseline Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the corner of Constellation Drive and Gemini Way. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 22, 2017

FILE NO. PG4184

HOLE NO. BH 3-17

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.13

FILL: Brown silty sand, some gravel 0.76

FILL: Brown silty clay, some sand, gravel, trace cobbles 3.05

Very stiff to stiff, brown **SILTY CLAY,** trace sand

- grey by 4.6m depth

10.00

GLACIAL TILL: Grey silty clay, some sand and gravel 10.36

Dynamic Cone Penetration Test commenced at 10.36m depth

Inferred **GLACIAL TILL**

12.73

End of Borehole

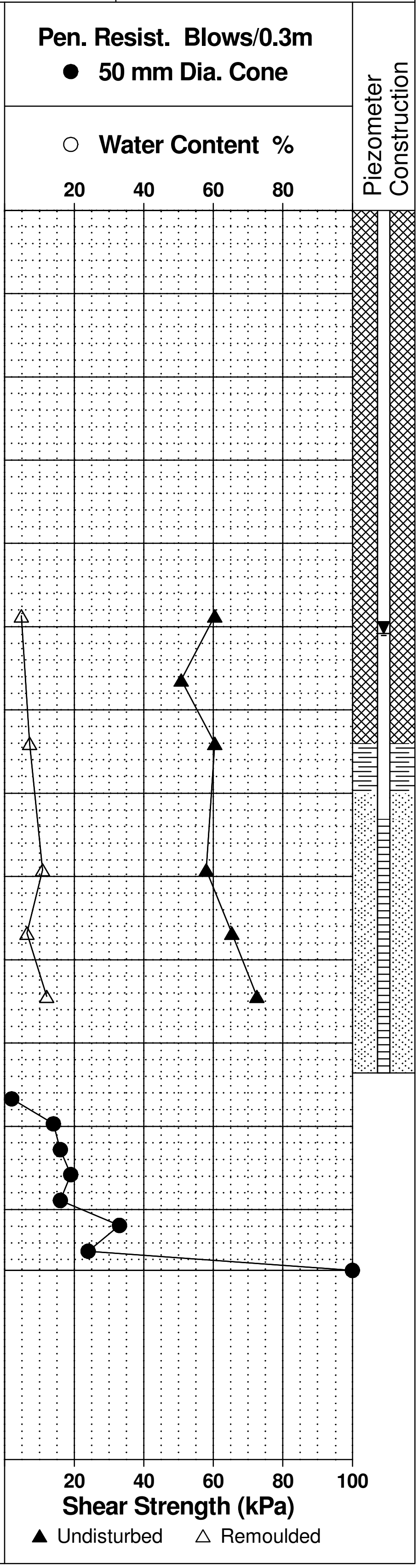
Practical DCPT refusal at 12.73m depth

(GWL @ 5.08m - June 29, 2017)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	5
SS	3	75	5
SS	4	50	3
SS	5	100	4
SS	6	100	P
SS	7	100	P
SS	8	100	1
SS	9	100	P
SS	10	100	P
SS	11	100	P

DEPTH (m)	ELEV. (m)
0	99.50
1	98.50
2	97.50
3	96.50
4	95.50
5	94.50
6	93.50
7	92.50
8	91.50
9	90.50
10	89.50
11	88.50
12	87.50



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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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 x 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)

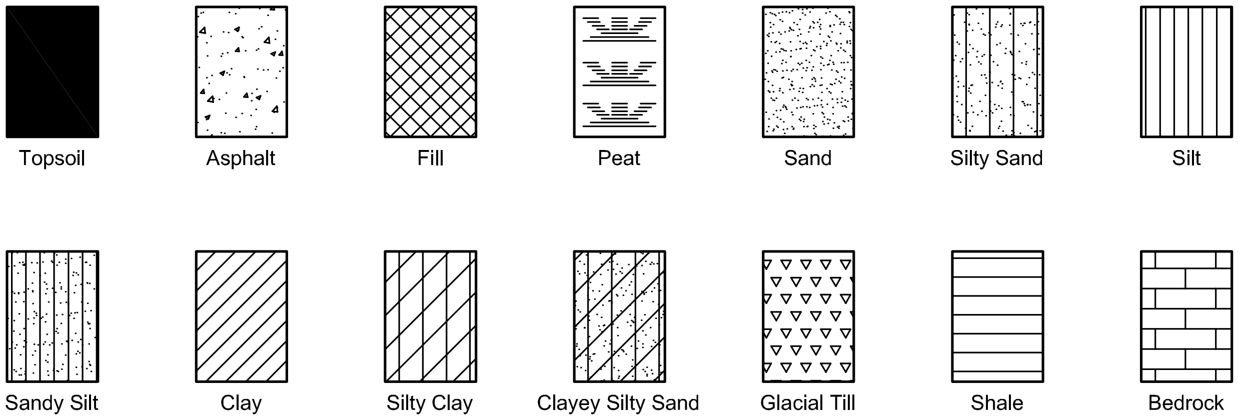
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

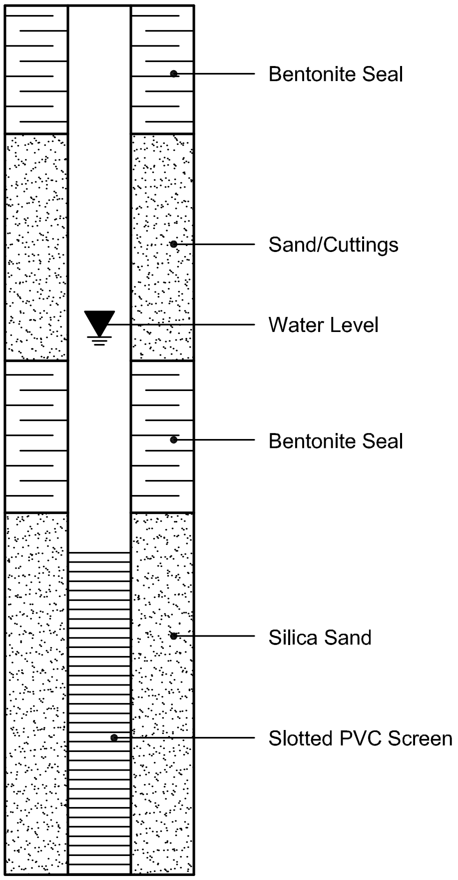
SYMBOLS AND TERMS (continued)

STRATA PLOT

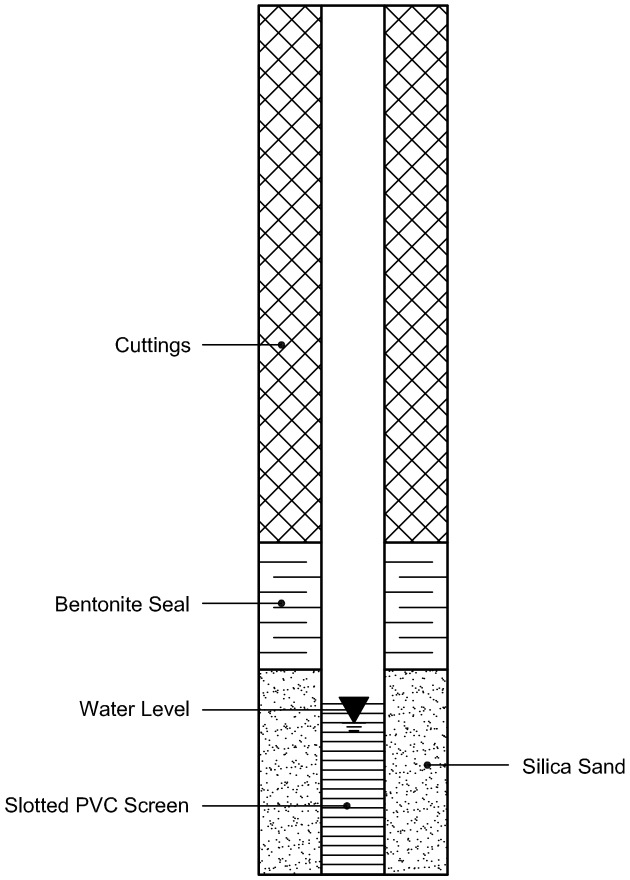


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



SNC·LAVALIN

Borehole ID: BH1

Page 1 of 1

Project No.: 642920
Client: City of Ottawa
Location: 2140 Baseline Rd, Ottawa, ON
Date Completed: Dec 5, 2016
Site Datum: n/a

SLI Supervisor: E. Kelly
Drilling Method: Direct push
Borehole Diameter: 82.5 mm

Drilling Company: Downing
Drilling Equipment: Geoprobe 7822DT
OVM: RKI Eagle 2

DEPTH (ft)	BLOW COUNT (1)	SAMPLE ID	LOCATION	OVM (2)	RECOVERY (%)	DESCRIPTION	ELEVATION (m)
0						Ground Surface	0.00
1	-	BH1-1	◆	<5	54%	Topsoil/Grass Silty Sand Fill dark brown, firm	
3	-	BH1-2	◆	<5	54%	Sand Fill brown, medium, firm	-1.00
5	-	BH1-3	◆	<5	100%	Silty Clay dry to moist, light brown/grey mottled, firm	
7	-	BH1-4		<5	100%		-2.00
9						End of borehole at 2.4 m bgs	-3.00

(1) Blow count per 0.15 m using conventional hammer and split spoons
(2) Organic Vapour Meter (OVM) reading (ppmv unless noted)

The data represented in this borehole log requires interpretation by SNC-Lavalin Environment personnel. Third parties using this log do so at their own risk.

All elevations and locations are approximate.

◆ Sample submitted for laboratory analysis.

BH1-1 submitted for EC, SAR, pH, grain size
BH1-99 (field dup of BH1-1) submitted for EC, SAR
BH1-2 submitted for VOCs/BTEX, PHC F1-F4, PAH, metals
BH1-98 (field dup for BH1-2) submitted for VOC/BTEX, PHC F1
BH1-3 submitted for VOCs/BTEX, PHC F1-F4, PAH, metals
BH1-97 (field dup of BH1-3) submitted for PHC F2-F4

SLE 3

SNC • LAVALIN

Borehole ID: BH2

Page 1 of 1

Project No.: 642920
Client: City of Ottawa
Location: 2140 Baseline Rd, Ottawa, ON
Date Completed: Dec 5, 2016
Site Datum: n/a

SLI Supervisor: E. Kelly
Drilling Method: Direct push
Borehole Diameter: 82.5 mm

Drilling Company: Downing
Drilling Equipment: Geoprobe 7822DT
OVM: RKI Eagle 2

DEPTH (ft / m)	BLOW COUNT (1)	SAMPLE ID	LOCATION	OVM (2)	RECOVERY (%)	DESCRIPTION	ELEVATION (m)
0 / 0						Ground Surface	0.00
0–2 ft	-	BH2-1		<5	67%	Topsoil/Grass Sand and Gravel Fill brown/grey, with silt, firm	
2–4 ft	-	BH2-2	◆	<5	67%		-1.00
4–6 ft	-	BH2-3		<5	67%	Silty Clay dry to moist, light brown/grey mottled, firm	
6–8 ft	-	BH2-4	◆	<5	67%		-2.00
8 ft						End of borehole at 2.4 m bgs	-3.00

(1) Blow count per 0.15 m using conventional hammer and split spoons
(2) Organic Vapour Meter (OVM) reading (ppmv unless noted)

The data represented in this borehole log requires interpretation by SNC-Lavalin Environment personnel. Third parties using this log do so at their own risk.

All elevations and locations are approximate.

◆ Sample submitted for laboratory analysis.

BH2-2 submitted for VOCs/BTEX, PHC F1-F4, PAH, metals
BH2-99 (field dup of BH2-2) submitted for PAH, metals
BH2-4 submitted for EC, SAR, pH, grain size

SLE 3

SNC·LAVALIN

Borehole ID: BH3

Page 1 of 1

Project No.: 642920
Client: City of Ottawa
Location: 2140 Baseline Rd, Ottawa, ON
Date Completed: Dec 5, 2016
Site Datum: n/a

SLI Supervisor: E. Kelly
Drilling Method: Direct push
Borehole Diameter: 82.5 mm

Drilling Company: Downing
Drilling Equipment: Geoprobe 7822DT
OVM: RKI Eagle 2

DEPTH (ft / m)	BLOW COUNT (1)	SAMPLE ID	LOCATION	OVM (2)	RECOVERY (%)
0–3.5 ft / 0–1.07 m	-	BH3-1	◆	<5	92%
3.5–7 ft / 1.07–2.13 m	-	BH3-2	◆	<5	92%
7–10.5 ft / 2.13–3.2 m	-	BH3-3		<5	67%
	-	BH3-4	◆	<5	67%
	-	BH3-5	◆	<5	100%

DESCRIPTION	ELEVATION (m)
Ground Surface	0.00
Topsoil/Grass	
Sand and Gravel Fill grey/brown, firm, with silt, some asphalt/gravel	
Silty Clay Fill dry to moist, light brown/grey mottled, firm	-1.00
Sand Fill brown, coarse	-2.00
Concrete Fill broken/pieces	
Sand Fill brown, coarse	-3.00
Silty Sand moist/wet, grey, soft	
End of borehole at 3.0 m bgs	
	-4.00
	-5.00
	-6.00

(1) Blow count per 0.15 m using conventional hammer and split spoons
(2) Organic Vapour Meter (OVM) reading (ppmv unless noted)

The data represented in this borehole log requires interpretation by SNC-Lavalin Environment personnel. Third parties using this log do so at their own risk.

All elevations and locations are approximate.

◆ Sample submitted for laboratory analysis.

BH3-1 submitted for PAH, metals
BH3-2 submitted for EC, SAR
BH3-4 submitted for VOCs/BTEX, PHC F1-F4, PAH, metals
BH3-5 submitted for EC, SAR, VOCs/BTEX, PHC F2-F4

SLE 3

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21941

Report Date: 30-Jun-2017

Order Date: 26-Jun-2017

Project Description: PG4184

	Client ID:	BH1-SS3	-	-	-
	Sample Date:	26-Jun-17	-	-	-
	Sample ID:	1726086-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.9	-	-	-
General Inorganics					
pH	0.05 pH Units	6.59	-	-	-
Resistivity	0.10 Ohm.m	14.9	-	-	-
Anions					
Chloride	5 ug/g dry	235	-	-	-
Sulphate	5 ug/g dry	263	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4184-1 - TEST HOLE LOCATION PLAN

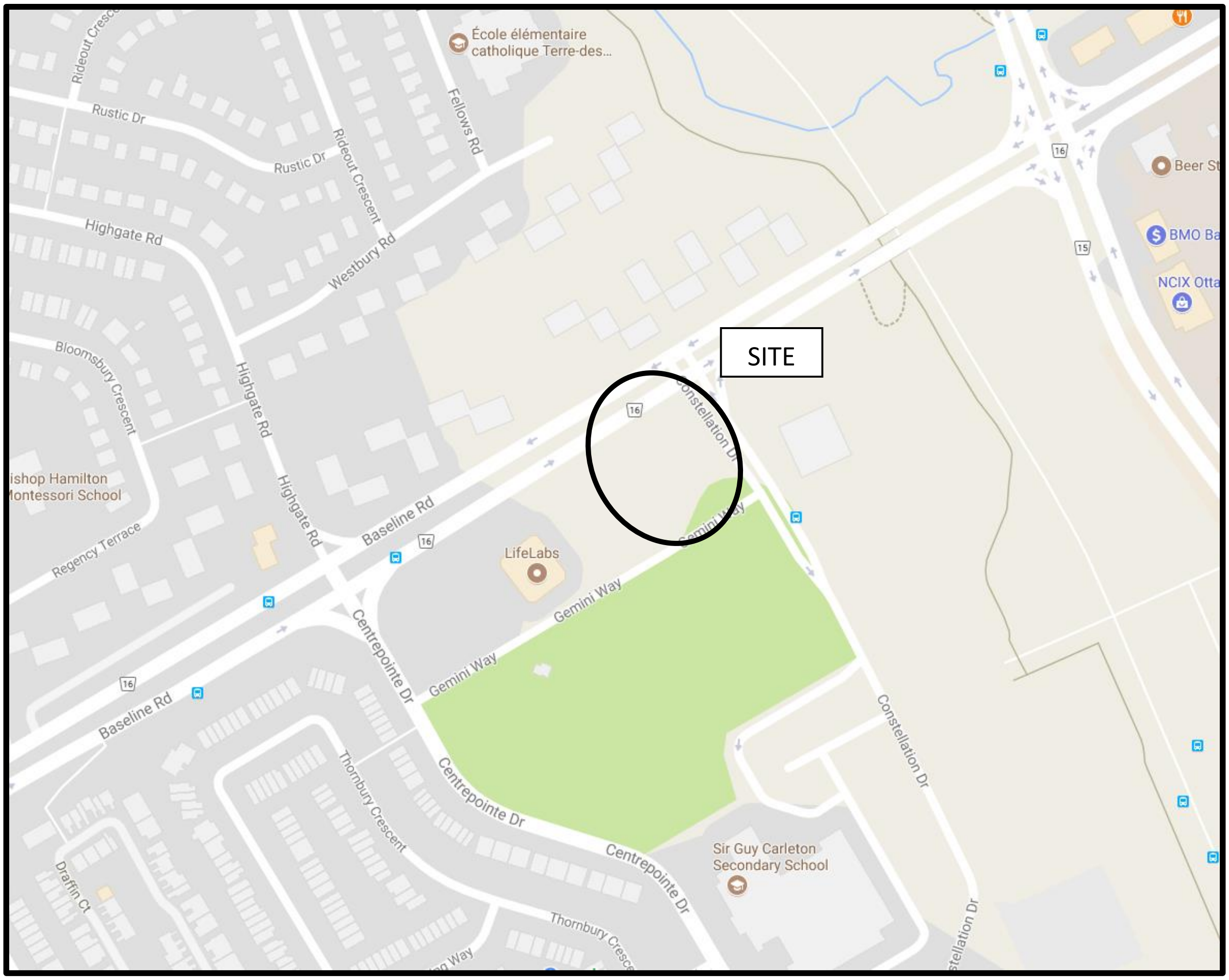
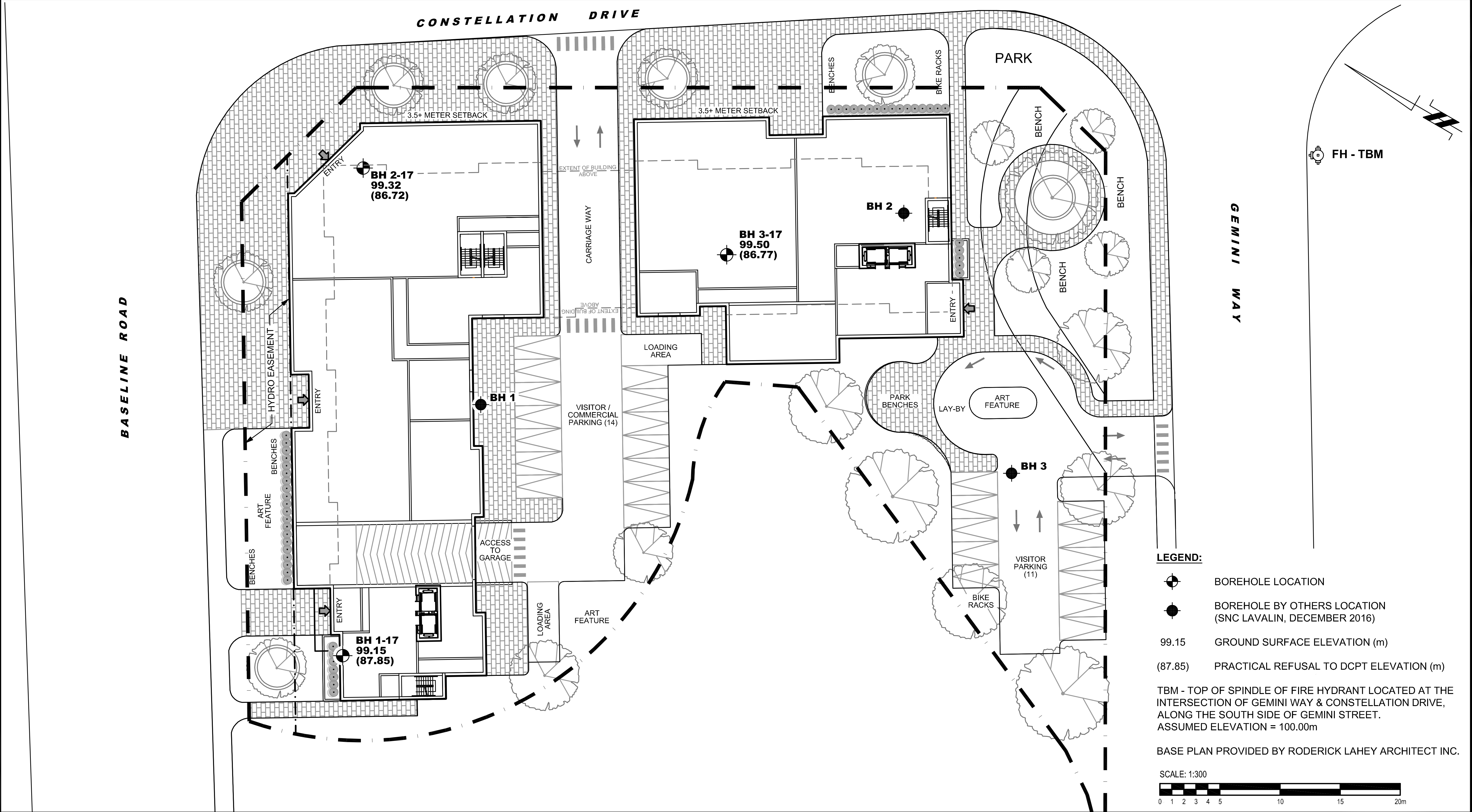


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
●	BOREHOLE BY OTHERS LOCATION (SNC LAVALIN, DECEMBER 2016)
99.15	GROUND SURFACE ELEVATION (m)
(87.85)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

TBM - TOP OF SPINDLE OF FIRE HYDRANT LOCATED AT THE INTERSECTION OF GEMINI WAY & CONSTELLATION DRIVE, ALONG THE SOUTH SIDE OF GEMINI STREET.
ASSUMED ELEVATION = 100.00m

BASE PLAN PROVIDED BY RODERICK LAHEY ARCHITECT INC.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

THEBERGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT - 2140 BASELINE ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: ~1:300	Date: 07/2017
Drawn by: RCG	Report No.: PG4184-1
Checked by: NZ	Dwg. No.: PG4184-1
Approved by: DJG	Revision No.: 0

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