

Geotechnical
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Geotechnical Investigation

Proposed Multi-Storey Building
1050 Canadian Shield Avenue, Ontario

Prepared For

Canadian Rental Development Services Inc.

May 19, 2021

Report: PG5371-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Canadian Rental Development Services Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1050 Canadian Shield Avenue in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development 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. 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 the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of one (1) multi-storey building with two (2) underground parking levels. Associated access lanes, hardscaped areas, and walkways are also anticipated as part of the proposed development. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on May 28, 2020 and consisted of advancing a total of 20 boreholes to a maximum depth of 11.2 m or bedrock refusal below existing ground surface. The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5371-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance 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 drilling to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the test pits were recovered from the auger flights. The samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory.

The thickness of the silty clay layer was evaluated during the course of the current geotechnical investigation by dynamic cone penetration testing (DCPT) completed at BH3, BH6, and BH9. 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.

Subsurface conditions observed in the test pits 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 pits locations.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5371-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant with a gravel covered parking area within the central area of the site. The ground surface across the subject site is generally flat and gently slopes down from the northwest portion (at approximate elevation of 102.7m) to the southeast portion (at approximate elevation of 99.0m). Also, the subject site was observed to be surrounded by trees along the southern and western borders of the site, and by a wide grassed area along the northern site border. Based on historical aerial photographs, it appears that the site was undergone topsoil removal and that piles of fill were stored within the subject site during the construction works at the adjacent eastern property in 2008.

The site is bordered by Campeau Drive to the north, Great Lakes Avenue to the west, Canadian Shield Avenue to the south, and by a multi-storey residential building and associated parking and landscaped areas to the east.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations within the northwest portion of the site consists of fill overlying brown to grey silty clay deposit. The silty clay deposit was inferred based on investigations completed within the surrounding areas of the subject site to range from very stiff to stiff silty clay. The silty clay deposit is underlain by compact glacial till and/or bedrock.

It should be noted that the overburden encountered at the test hole locations within the southeast portion of the site were observed to consist of brown silty sand and/or silty sand with crushed stone fill overlying the bedrock.

Practical refusal to augering or DCPT on inferred bedrock was encountered in all the boreholes at depths varying between 1.0 to 12.4m. A glacial till layer was encountered below the silty clay layer at a depth of 8.2m in BH 8. Also, a layer of brown silty sand was encountered below ground surface in BH7. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Bedrock

Based on available geological mapping, and refusal to augering/DCPT, the bedrock in the subject area consists of Paragneiss Migmatic bedrock of granitic origin, with an overburden thickness of 1 to 10 m depth.

4.3 Groundwater

Paterson reviewed the groundwater levels during the geotechnical field investigation and surrounding sites and groundwater levels logged from public wells. The groundwater was also estimated based on the colouring, consistency and moisture levels of the recovered samples. Based on these observations, and our knowledge of the area, the long-term groundwater table is estimated to be found at a depth ranging from 3 to 4 m depth below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed multi-storey building. The foundation support system required is dependent on the design building loading and depth of foundation. Several foundation support options are listed below and discussed in the following sub-sections:

- Conventional footings placed on very stiff to stiff brown silty clay, silty sand, glacial till bearing surface and/or a clean, surface sounded bedrock bearing surface.
- Conventional footings placed on vertical, zero entry lean concrete filled trenches extended to the underlying bedrock bearing surface where depths to bedrock are considered feasible for this application.
- End bearing piled foundations that extend down a clean, surface sounded bedrock bearing surface where the depth of bedrock is considered too deep for lean concrete filled trenches.

Based on the current conceptual drawing, it is expected that bedrock removal will be required for portions of the underground parking levels of the proposed structure. Line drilling and controlled blasting could be used where large quantities of bedrock are needed to be removed. The blasting operations should be planned and conducted under the guidance of a professional engineer with experience in blasting operations.

Due to the presence of a silty clay deposit within the subject site, the subject site will be subjected to a permissible grade raise restriction. However, due to the lack of vane testing within the clay deposit, additional testing will be required within the clay deposit in order to provide a sufficient permissible grade raise restriction. It should be noted that this option would not be required if the proposed building will be fully founded over a bedrock bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, all existing fill material will be excavated from within the proposed building footprint. Bedrock removal will be required for portions of the underground parking levels.

Topsoil and deleterious fill, such as those containing organic materials, or construction debris/remnants should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing. The 1 m horizontal ledge set back can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Bedrock Excavation Face Reinforcement

A bedrock stabilization system consisting of a combination of horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs. This system is usually considered where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Fill Placement

Fill placed for grading beneath the building areas 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 imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for pavements. Where the fill is open-graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

5.3 Foundation Design

Several foundation design options are available for the proposed building depending on the design loading and foundation depth. The following foundation options are recommended:

Conventional Shallow Foundation

Using continuously applied loads, footings for the proposed building placed over an undisturbed silty clay, silty sand, glacial till or bedrock bearing surface can be designed using the bearing resistance values presented in Table 1.

Table 1 - Bearing Resistance Values		
Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Silty Clay	150	225
Silty Sand	120	200
Glacial Till	200	300
Bedrock	-	1500
Note: Strip footings, up to 2 m wide, and pad footings, up to 5 m wide, placed over a silty clay bearing surface can be designed using the above noted bearing resistance values.		

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces or clean, surface-sounded bedrock bearing surface.

An undisturbed 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.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating zero entry, vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**20 MPa** 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, a test pit should be undertaken to assess the water infiltration issues and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. Where strip footings are butting against a vertical bedrock face, the extent of the vertical, in-filled trench can be stopped at the bedrock face. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**.

Deep Foundation - End Bearing Piles

A deep foundation method, such as end bearing piles, can be considered for the central north portion of the proposed structure where the bedrock layer extends to a depth well below the design underside of footing elevation. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 1. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 1. 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 calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four 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 will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - End Bearing Pile Foundation Design Data					
Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	975	1460	10	35.9
245	12	1100	1650	10	42
245	13	1175	1760	10	45.4

Settlement

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

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 native soil and engineered fill bearing media when a plane extending down and out from the bottom edges 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 that of the bearing medium. In unfractured bedrock, a plane with a slope of 1H:6V can be used.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations constructed at the subject site. A higher site class, such as Class A or B may be provided for foundations placed on or near the bedrock surface. However, the higher site class will need to be confirmed by a site-specific seismic shear wave velocity test. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made 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 deleterious fill within the footprint of the proposed building, the native soils and bedrock surface will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

Where silty sand or glacial till subgrade is encountered below the basement slab, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings). It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

If the floor slab is constructed in the areas of shallow bedrock, it is recommended that a minimum 300 mm thick layer (native soil plus crushed stone layer) be present between the floor slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

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 bulk (drained) unit weight of 20 kN/m³.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using 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/m³)
- H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above 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 used in conjunction 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 maintain a minimum separation of 0.3 m 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}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

The earth force component (P_o) under seismic conditions can be calculated using

$$P_o = 0.5 K_o \gamma H^2, \text{ where } K_o = 0.5 \text{ for the soil conditions noted above.}$$

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 Design

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil and possibly bedrock. The proposed pavement structures are presented in Tables 3 and 4.

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 I or II material.

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, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

Table 4 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Wear Course – HL-8 or Superpave 19 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
450	SUBBASE – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. 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 compaction equipment.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the preliminary information provided, it is expected that a portion of the proposed building foundation walls located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which breaches the primary ground infiltration control system.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level and the following is suggested for preliminary design purposes:

- Place a suitable waterproofing membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- Pour foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 6-9 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

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 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

The footings located along parking garage entrance may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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 subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away 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.

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.

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 designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of 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 temporary shoring system may be calculated with the following parameters.

Table 5 – Soils Parameter for Shoring System Design	
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
Unit Weight (γ), kN/m ³	20
Submerged 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 and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate 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.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) 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 MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically 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 MECP review of the PTTW application.

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day/building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the anticipated groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

The subsoil conditions at this site consist of frost susceptible materials. In the 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 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.

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. Additional information could be provided, if required.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- Review the bedrock stabilization and excavation requirements.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Sampling and testing of the concrete and fill materials used.
- 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 that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection 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 Canadian Rental Development Services Inc. 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.



Maha Saleh, P.Eng (Prov)



Faisal I. Abou-Seido, P.Eng

Report Distribution:

- Canadian Rental Development Services Inc. (3 copies)
- Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS SYMBOLS AND TERMS

DATUM Geodetic


FILE NO. **PG5371**

REMARKS

HOLE NO. **BH 1**

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	102.65						
Brown SILTY CLAY with gravel						1	101.65						
End of Borehole													
Practical refusal to augering at 1.80m depth													

1.80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

DATUM Geodetic


REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE						0	102.65						
Brown to grey SILTY CLAY with sand, trace gravel						1	101.65						
						2	100.65						
						3	99.65						
						4	98.65						
						5	97.65						
						6	96.65						
						7	95.65						
						8	94.65						
						9	93.65						
						10	92.65						
						11	91.65						
End of Borehole													
Practical refusal to augering at 11.23m depth													

11.23

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %						
GROUND SURFACE								20	40	60	80			
Brown to grey SILTY CLAY , some sand						0	102.33							
						1	101.33							
						2	100.33							
						3	99.33							
						4	98.33							
						5	97.33							
						6	96.33							
						7	95.33							
						8	94.33							
						9	93.33							
						10	92.33							
						11	91.33							
				12	90.33									
Dynamic Cone Penetration Test commenced at 9.14m depth. Cone pushed to 11.3m depth.														
End of Borehole														
Practical DCPT refusal at 12.42m depth														



DATUM Geodetic


FILE NO. **PG5371**

REMARKS

HOLE NO. **BH 4**

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	101.93						
Brown SILTY CLAY , some sand and gravel						1	100.93						
End of Borehole	1.98												
Practical refusal to augering at 1.98m depth													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

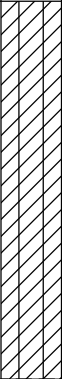
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 5**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	101.78						
Brown SILTY CLAY						1	100.78						
						2	99.78						
						3	98.78						
End of Borehole													
Practical refusal to augering at 3.89m depth													

3.89

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

DATUM Geodetic


FILE NO. **PG5371**

REMARKS

HOLE NO. **BH 6**

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %						
GROUND SURFACE								20	40	60	80			
FILL: Brown silty sand with crushed stone						0	100.88							
						1	99.88							
						2	98.88							
						3	97.88							
End of Borehole														
Practical refusal to augering at 3.71m depth														

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 7**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80			
GROUND SURFACE						0	99.18							
Brown SILTY SAND , some gravel						1	98.18							
						2	97.18							
						3	96.18							
End of Borehole														
Practical refusal to augering at 3.48m depth														

3.48

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

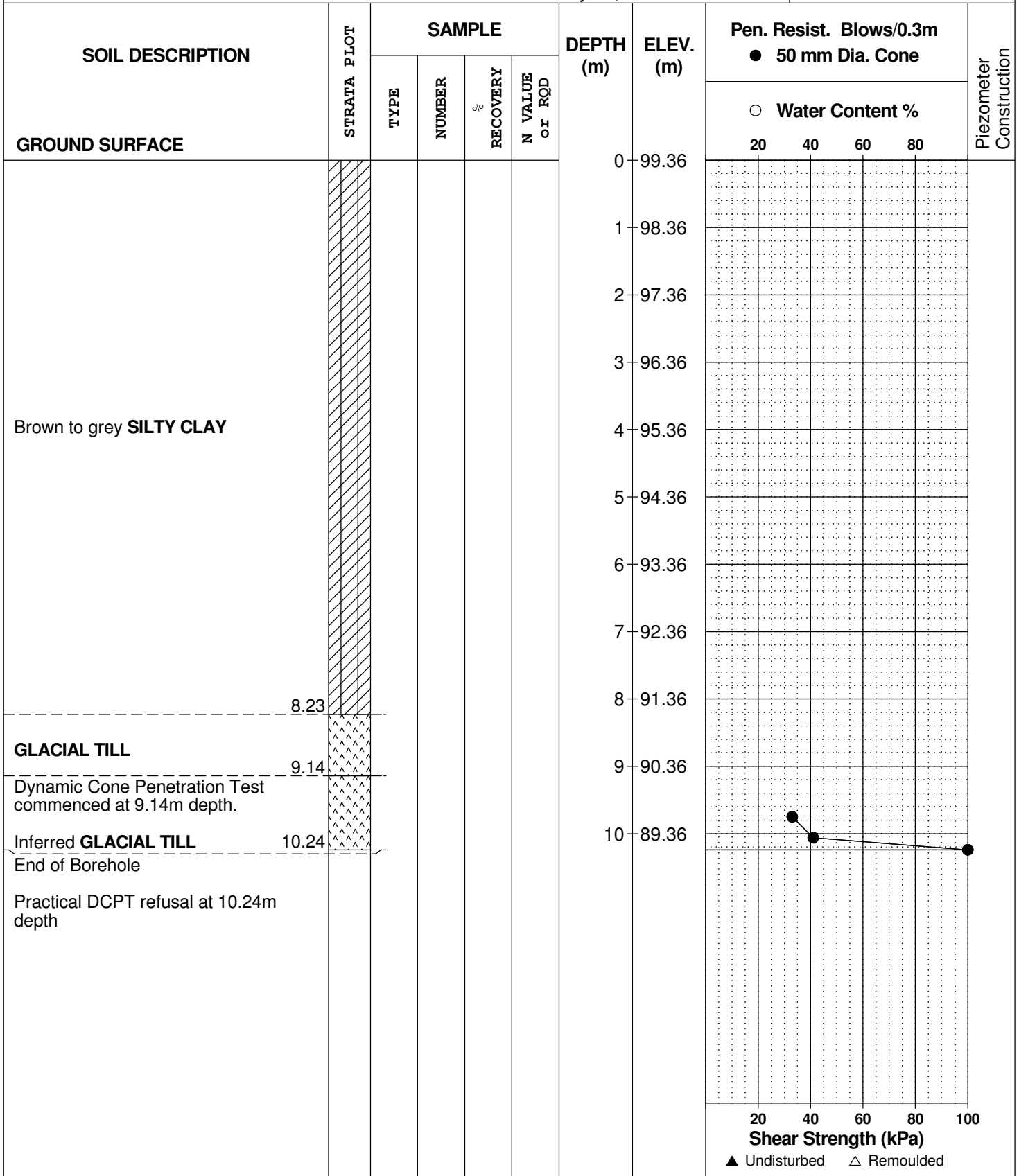
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 8**



DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH 9**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	99.60						
FILL: Crushed stone	0.60												
Brown to grey SILTY CLAY						1	98.60						
						2	97.60						
						3	96.60						
						4	95.60						
						5	94.60						
						6	93.60						
Dynamic Cone Penetration Test commenced at 6.10m depth	6.10												
End of Borehole	6.38												
Practical DCPT refusal at 6.38m depth													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH10**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %					
GROUND SURFACE								20	40	60	80		
FILL: Brown silty sand with gravel	0.60					0	100.55						
Brown to grey SILTY CLAY	5.03					1	99.55						
						2	98.55						
						3	97.55						
						4	96.55						
						5	95.55						
End of Borehole													
Practical refusal to augering at 5.03m depth													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH12**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
GROUND SURFACE								20	40	60	80		
FILL: Brown silty clay, some gravel and crushed stone	0.60					0	99.66						
Grey SILTY CLAY	7.60					1	98.66						
						2	97.66						
						3	96.66						
						4	95.66						
						5	94.66						
						6	93.66						
						7	92.66						
End of Borehole													
Practical refusal to augering at 7.60m depth													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

FILE NO. **PG5371**

REMARKS

HOLE NO. **BH14**

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	98.81						
FILL: Brown silty sand with crushed stone						1	97.81						
End of Borehole	1.50												
Practical refusal to augering at 1.50m depth													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH15**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	98.93						
FILL: Brown silty sand with clay and crushed stone						1	97.93						
End of Borehole													
Practical refusal to augering at 1.04m depth													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic


REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 28, 2020

FILE NO. **PG5371**

HOLE NO. **BH16**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %						
GROUND SURFACE								20	40	60	80			
FILL: Brown silty sand with crushed stone						0	98.90							
						1	97.90							
						2	96.90							
						3	95.90							
End of Borehole														
Practical refusal to augering at 3.15m depth														

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic


REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

FILE NO. **PG5371**

HOLE NO. **BH17**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %					
GROUND SURFACE								20	40	60	80		
FILL: Brown silty sand with crushed stone						0	99.05						
						1	98.05						
						2	97.05						
End of Borehole													
Practical refusal to augering at 2.77m depth													

2.77

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

DATUM Geodetic


REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

FILE NO. **PG5371**

HOLE NO. **BH18**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
FILL: Brown silty sand with crushed stone						0	99.62					
						1	98.62					
						2	97.62					
						3	96.62					
End of Borehole	3.28											
Practical refusal to augering at 3.28m depth												

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic


FILE NO. **PG5371**

REMARKS

HOLE NO. **BH19**

BORINGS BY CME-55 Low Clearance Drill

DATE May 29, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY	N VALUE or RQD			○ Water Content %						
GROUND SURFACE								20	40	60	80			
FILL: Brown silty sand with crushed stone						0	99.98							
						1	98.98							
						2	97.98							
						3	96.98							
End of Borehole														
Practical refusal to augering at 3.45m depth														

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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		Overconsolidation 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.
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SYMBOLS AND TERMS (continued)

STRATA PLOT



Topsoil



Asphalt



Fill



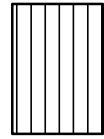
Peat



Sand



Silty Sand



Silt



Sandy Silt



Clay



Silty Clay



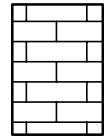
Clayey Silty Sand



Glacial Till



Shale



Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5371-1 – TEST HOLE LOCATION PLAN

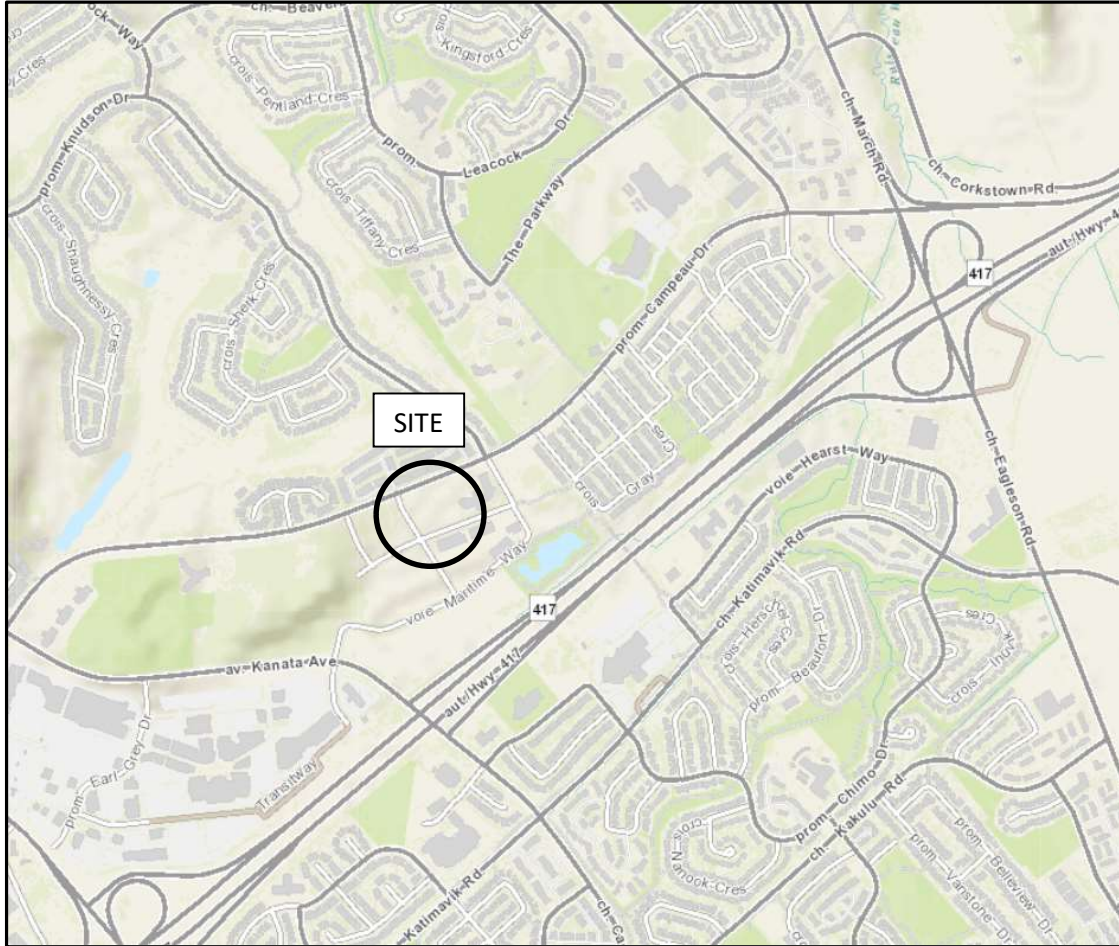
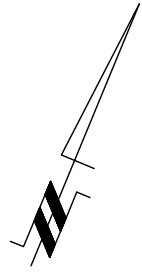
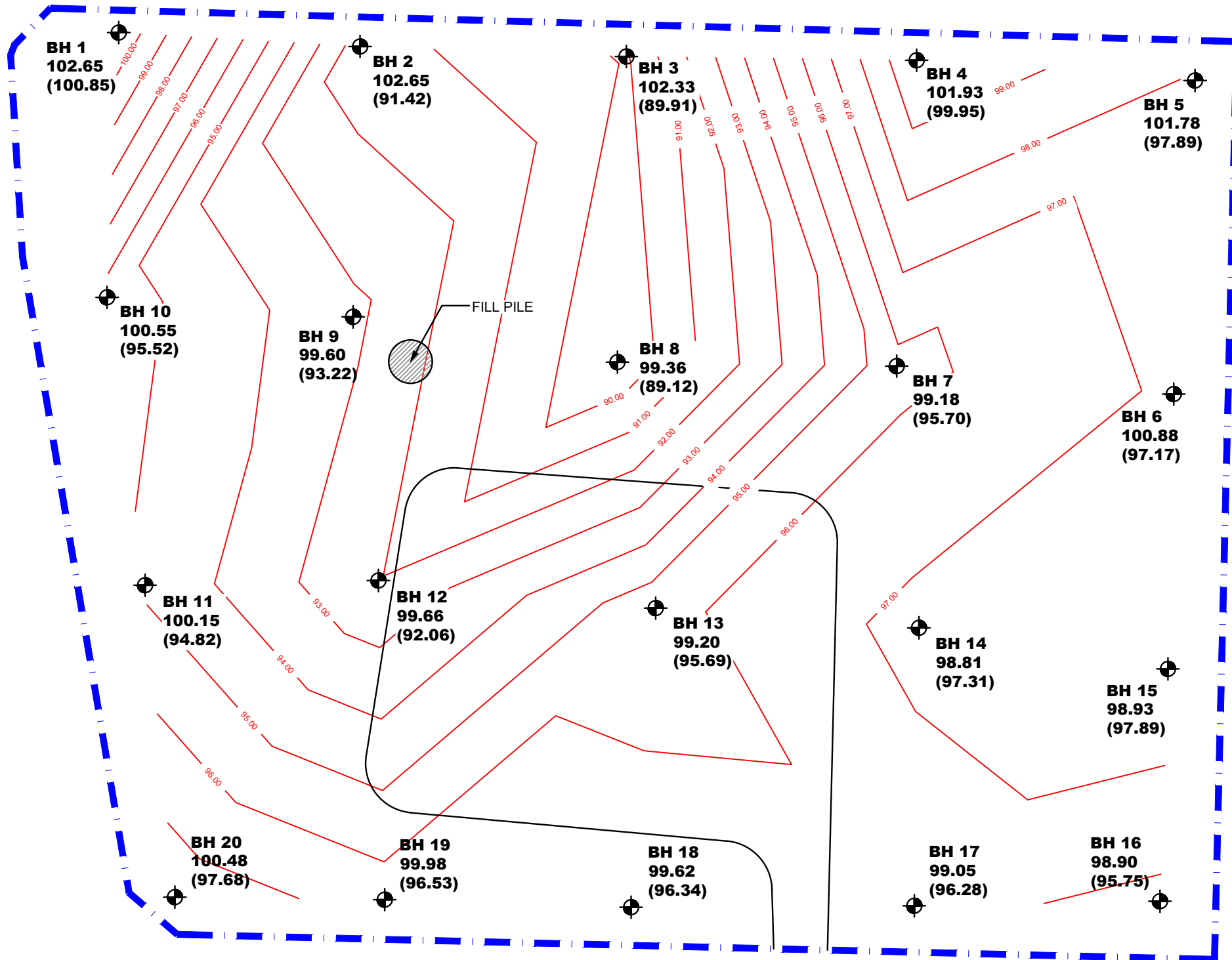


FIGURE 1

KEY PLAN





GREAT LAKES AVENUE



CANADIAN SHIELD AVENUE

LEGEND:

-  BOREHOLE LOCATION
- 102.65 GROUND SURFACE ELEVATION (m)
- (100.85) PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
-  96.00 BEDROCK CONTOUR

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATION ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500



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NO.	REVISIONS	DATE	INITIAL

CANADIAN RENTAL DEVELOPMENT SERVICES
BEDROCK SURFACE DELINEATION INVESTIGATION
1050 CANADIAN SHIELD AVENUE

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	06/2020
Drawn by:	YA	Report No.:	PG5371-1
Checked by:	KP	Dwg. No.:	PG5371-1
Approved by:	DG	Revision No.:	