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Updated Subsurface Investigation
Block 3
Heritage Hills Development
124 Battersea Crescent
Ottawa, Ontario

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Project: 62970.01

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1.0 INTRODUCTION

This report presents the results of a subsurface investigation carried out at the site of a proposed Heritage Hills residential development at 124 Battersea Crescent in Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of test pits and boreholes. Based on the factual information obtained, engineering guidelines were to be provided on the geotechnical aspects of the design of the project, including construction considerations which could influence design decisions.

Our initial subsurface investigation report issued in March 2011, and revised in December 2013, was prepared relative to the construction of eight (8) residential blocks, including a garbage and recycling facility, at the site. The overall development plans for this site were revised following issuance of our initial reports. This updated report includes geotechnical guidelines for the construction of one (1) residential block within the northwest portion of the site.

2.0 BACKGROUND

2.1 Project Description

Plans are being prepared to construct one (1) additional residential block within the Heritage Hills development located at the southwest corner of the intersection of Kanata Avenue and Richardson Side Road in Ottawa, Ontario (refer to Figure 1). It is understood that the proposed residential block will consist of a 4 storey structure with one level of underground parking and will be located within the northwest portion of the site. The structure will be of wood frame construction with concrete foundation walls. For the purposes of this report, we are referring to the residential block as Block 3.

Based on the information provided to us, it is understood that an underside of footing elevation of 113.3 metres is proposed for Block 3. The finished grades around Block 4 will be at about 117.0 metres. At its closest point, the proposed underground parking facility is to be located 4 metres from an existing 3 storey building (i.e., Block 2).

At grade asphaltic concrete surfaced parking areas and access roadways will be provided. Storm sewers, sanitary sewers and watermains will be installed to the proposed building.

Precambrian bedrock is exposed at ground surface at various locations across the site; however, more predominantly within the centre and west areas of the site.

2.2 Review of Geology Maps

Surficial geology maps of the Ottawa area indicate that a portion of the site, being the southeast area of the proposed development, is underlain by marine deposits composed of silty clay. Thin or discontinuous overburden deposits are mapped within the west and centre portions of the site. Bedrock geology maps indicate that the bedrock consists of gabbro, diorite, and quartzite.

3.0 SUBSURFACE INVESTIGATION

3.1 Test Pit Investigation

The first phase of the field work for this investigation was carried out on February 25, 2008. At that time, ten (10) test pits, numbered 1 to 10, inclusive, were advanced at the site to depths of between about 0.02 and 4.5 metres below existing ground surface using track mounted, hydraulic excavation equipment supplied by a local excavating company. All of the test pits, with the exception of test pit 7, were advanced to practical refusal to excavating on the apparent bedrock surface. The subsurface conditions encountered in the test pits were classified based on visual and tactile examination of the materials exposed on the sides and bottom of the test pits and the excavated materials. Standpipes were installed in selected test pits to measure the groundwater levels. The groundwater conditions in the other test pits were observed at the time of excavating. The field work was supervised throughout by a member of our engineering staff.

Descriptions of the subsurface conditions logged in the test pits are provided on the Record of Test Pit sheets in Appendix A. The approximate locations of the test pits are shown on the Site Plan, Figure 2.

The test pit locations and elevations were determined by Novatech. It is understood that the elevations are referenced to geodetic datum.

The locations of the test pits and boreholes were selected based on a development plan which has since been superseded.

3.2 Borehole Investigation

The second stage of the investigation was carried out between May 1 and 5, 2008. During that time three (3) boreholes, numbered 1 to 3, inclusive, were advanced within the east part of the site using a track mounted, hollow stem auger drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario.

Details for the boreholes are provided below:

- Two (2) boreholes, numbered boreholes 1 and 3, were advanced to practical refusal at depths of 4.7 and 2.3 metres below ground surface, respectively, to identify the inferred depth of the bedrock. No soil sampling was carried out in the boreholes. The soil types in borehole 1 were inferred based on examination of the auger cuttings.
- One (1) borehole, numbered borehole 2, was advanced to 18.9 metres below ground surface. The borehole was advanced to the bedrock and the underlying bedrock was cored using NQ size diamond drilling equipment to identify the type and quality of the bedrock. Shear wave velocity testing was carried out in a casing grouted into the borehole to identify the seismic Site Class.

Standard penetration tests were carried out where possible within the overburden deposits in borehole 2 and samples of the soils encountered were recovered using 50 millimetre diameter drive-open sampling equipment. At borehole 2, the bedrock was cored using NQ size rotary diamond drilling equipment to facilitate the installation of a 50 millimetre diameter PVC casing for subsequent shear wave velocity testing.

Following the borehole drilling work, the soil and bedrock samples were returned to our laboratory for examination by the project engineer.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix B. The approximate locations of the boreholes are shown on the Site Plan, Figure 2.

The field work was supervised throughout by a member of our engineering staff, who located the boreholes relative to existing site features, logged the samples and boreholes and observed the in-situ testing.

3.3 Shear Wave Velocity Testing

In situ shear wave velocity testing was carried out in borehole 2. To carry out the testing, a shear wave was mechanically induced at surface using a steel plate and hammer, and was picked up by a geophone installed in a casing grouted into the borehole. The signal from the geophone was recorded by means of a laptop computer at surface. The shear wave velocity with depth was then computed by dividing the travel distance by the travel time of the shear wave. A schematic of the test arrangement and the results of the testing are provided in Appendix C.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil, bedrock and groundwater conditions identified in the test pits and boreholes are given on the Record of Test Pit and Borehole sheets in Appendices A and B. The test pits and borehole logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the conditions encountered, the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the test pits and boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the test pits and boreholes advanced during this investigation.

4.2 West Part of the Site (Test Pits 1 to 5, 9 and 10)

4.2.1 General Overview

The subsurface within the west part of the site is characterized by shallow depth Precambrian bedrock overlain by thin discontinuous deposits of topsoil/peat and glacial till.

4.2.2 Topsoil and Peat

Test pits 1, 3, 5, 9 and 10 encountered a surficial layer of topsoil. The topsoil has a thickness ranging from about 0.02 to 0.2 metres.

A layer of peat was encountered from surface in test pits 2 and 4 to depths of about 0.5 and 0.7 metres, respectively.

4.2.3 Glacial Till

A deposit of glacial till was encountered above the underlying in bedrock in test pits 2, 5 and 9. The glacial till is a heterogeneous mixture of all grain sizes but may be generally described as silty sand with variable amounts of clay, gravel, cobbles and boulders. At these locations, the thickness of the glacial till ranges from about 0.3 to 1.1 metres.

4.2.4 Bedrock

Refusal to further excavating was encountered on or within the apparent bedrock in test pits 1 to 5, 9 and 10, at depths ranging from 0.02 to 1.2 metres below ground surface (elevation 110.6 to 122.1 metres, geodetic datum). At all test pits, with the exception of test pit 9, practical refusal to excavating was encountered on the upper surface of apparent sound bedrock. Practical refusal at test pit 9 was on apparent fractured bedrock.

4.2.5 Groundwater Conditions

Groundwater inflow was observed in test pits 2, 4, 5 at depths of between 0 and 0.5 metres below ground surface upon completion of excavation. No groundwater inflow was observed in test pits 1, 3, 9 and 10 during the short period that they were left open following excavation.

It should be noted that the groundwater levels do not necessarily represent the stabilized groundwater conditions and that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of heavy precipitation.

4.3 East Part of the Site (Test Pits 6, 7, 8 and Boreholes 1 to 3)

4.3.1 General Overview

The subsurface within the east part of the site is characterized by topsoil followed by deposits of silty clay of marine origin. The silty clay is underlain by thin, discontinuous deposits of glacial till and then Precambrian bedrock.

4.3.2 Topsoil and Peat

Test pits 6, 7, and 8 and borehole 1 and 2 encountered a surficial layer of topsoil. The topsoil has a thickness ranging from about 0.1 to 0.5 metres. (Note that the soil conditions in borehole 3 were not logged and, as such, the thickness of the topsoil at this location is not known.)

4.3.3 Silty Sand

A deposit of grey brown silty sand was encountered below the topsoil in boreholes 6, 7 and 8. The thickness of the silty sand ranges from about 0.2 metres (test pit 8) to 0.6 metres (test pit 7).

4.3.4 Silty Clay

Native deposits of silty clay were encountered in test pits 6, 7 and 8, and in boreholes 1 and 2. The silty clay has a thickness of between 0.7 metres (test pit 6) to 4.1 metres (borehole 1) and extends to depths of about 1.1 to 4.6 metres below ground surface (elevation 105.0 to 109.9 metres, geodetic datum).

The silty clay is weathered grey brown at all of the test pit and borehole locations. Standard penetration tests carried out in the weathered, grey brown silty clay encountered in borehole 2 gave N values ranging from 6 to 18 blows per 0.3 metres of penetration, which reflect a very stiff consistency. Based on the material recovered in the bucket of the shovel, the silty clay encountered at depth in test pits 7 and 8 had an estimated stiff to very stiff consistency.

4.3.5 Glacial Till

A deposit of glacial till was encountered below the silty clay in test pits 6, 7 and 8 and boreholes 1 and 2. The glacial till is a heterogeneous mixture of all grain sizes but may be generally described as silty sand with variable amounts of clay, gravel, cobbles and boulders. At these locations, the thickness of the glacial till ranges from about 0.1 to 4.8 metres.

4.3.6 Bedrock

Refusal to further excavating was encountered on or within the apparent bedrock in test pits 6, 7, and 8, at depths ranging from 2.3 to 4.5 metres below ground surface (elevation 104.7 to 108.8 metres, geodetic datum).

Practical auger refusal was encountered on the inferred surface of the bedrock in boreholes 1 and 3 at depths of 4.7 and 2.3 metres below ground surface, respectively (elevation 107.0 and 106.2 metres, geodetic datum).

Bedrock was encountered and cored at the location of borehole 2 at a depth of about 8.2 metres below ground surface (elevation 102.0 metres, geodetic datum). The bedrock consists of competent, hard, grey quartzite bedrock.

4.3.7 Groundwater Conditions

Groundwater inflow was observed in test pit 7 at a depth of about 1.3 metres below ground surface at the time of excavation. The groundwater in the standpipes installed in test pits 6 and 8 was frozen at the time of the investigation (March 2008).

It should be noted that the groundwater levels do not necessarily represent the stabilized groundwater conditions and that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of heavy precipitation.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off site sources are outside the terms of reference for this report.

5.2 Excavation

5.2.1 Overburden Excavation

In overburden, the excavations for the proposed building and site services will be carried out through surficial topsoil, peat, and possibly fill material. The sides of the excavations in

overburden should be sloped in accordance with the requirements of the Ontario Occupational Health and Safety Act. The native soils can be classified as Type 3 soils. As such, for overburden excavations greater than 1.2 metres depth should be carried out using 1 horizontal to 1 vertical, or flatter, side slopes extending from the bottom of the excavation. In areas where space constraints dictate, the sides of the service trenches could be supported by a steel trench box designed for this purpose.

5.2.2 Bedrock Excavation

The Precambrian bedrock at this site is known to be hard and abrasive on drilling steel and pneumatic hoe ram equipment. Most of the bedrock removal at this site will likely require drill and blasting techniques. Excavations in bedrock should stand near vertically; however, the sides of the excavations should be scaled to remove any loose bedrock material.

Any blasting should be carried out under the supervision of a blasting specialist engineer. As a guideline for blasting, a maximum peak particle velocity of 50 millimetres per second could be used as the vibration criteria at the nearest structure or service. It is pointed out that this criteria, although conservative, was established to prevent damage to existing buildings and services; more stringent criteria may be required to prevent damage to freshly placed (uncured) concrete.

The bedrock in this area is known to contain random joints. To reduce, not prevent, over break and under break of bedrock in the excavation, line drilling on close centres is suggested along the design limits of the excavation. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting vibration criteria.

Recommendations for bedrock removal adjacent to existing structures (i.e., Block 2) are provided in Section 5.2.4 below.

5.2.3 Groundwater Pumping

Groundwater seepage into excavations is expected and should be controlled, as necessary, by pumping from within the excavations. It is expected that the groundwater pumping requirements during construction will be less than 50,000 litres per day. As such, it is anticipated that Environmental Activity and Sector Registry (EASR) or a Permit to Take Water (PTTW) will not be required for this project.

It is not expected that groundwater pumping will affect structures or services on adjacent properties.

5.2.4 Excavation Adjacent to Block 2

At its closest point, the proposed underground parking facility is to be located 4 metres north of an existing 3 storey building (i.e., Block 2). Based on the information provided to us, it is understood that Block 2 is founded at about elevation 115.8 on a thin levelling layer of granular

material over bedrock (i.e., about 2.3 metres above the proposed underside of footing level for Block 3).

Although feasible, bedrock removal using drill and blasting techniques within about 6 metres of the existing structure would require additional measures to reduce the risk of damage to the existing structure and may be cost prohibitive. In these areas, consideration should be given to trimming to the design limits using a combination of hoe ramming techniques and line drilling. For this case, it is suggested that the perimeter of excavation adjacent to the existing structure be defined by line drilling on close centers in order to reduce over break and/or under break of the bedrock. For the bedrock at this site, it is suggested that allowance be made for line drilling 75 to 100 millimetre diameter holes on 200 to 300 millimetre centres. As previously indicated, the Precambrian bedrock at this site is known to be hard and abrasive on pneumatic hoe ram equipment. In addition to line drilling along the perimeter of the excavation, drilled holes, positioned on a grid pattern within the footprint of the planned excavation, could be used to assist in breaking the bedrock using hoe ram equipment.

We recommend that the edge of the excavation adjacent to Block 2 be setback at least 1.2 metres from the edge of the existing footings. Further, it will be necessary remove a portion of the existing foundation wall backfill material to allow for side slopes of 1 horizontal to 1 vertical, or flatter (in accordance with the requirements of the Ontario Occupational Health and Safety Act). Where this is not possible, some form of shoring system may be required to support the existing backfill material. Geotechnical guidelines to assist with assessing the general shoring requirements could be provided, if required.

The vibration effects of hoe ramming are usually minor and localized. Monitoring of the hoe ramming could be carried out, at least initially, to measure the vibrations to ensure that they are below the acceptable threshold value.

5.3 Foundation Design

5.3.1 Subgrade Type

Based on the results of the investigation, together with the planned underside of footing elevation 113.3 metres, the expected subgrade conditions for Block 3 are summarized in Table 5.1.

Table 5.1 – Excepted Subgrade Conditions

Block	Expected Subgrade Conditions
Block 3	Precambrian Bedrock or Engineered Fill over Precambrian Bedrock.

5.3.2 Foundation Bearing Pressures

Spread footing foundations bearing on or within competent Precambrian bedrock could be sized using a geotechnical reaction at Serviceability Limit States (SLS) of 500 kilopascals and a factored geotechnical resistance at Ultimate Limit States (ULS) of 1000 kilopascals. The above bearing values assume that all soil and any loose or fractured bedrock is removed from the bearing surface, but take into account some minor blast induced damage to the bedrock below founding level.

Provided that no significant fractured bedrock or soil filled seams exist below founding level, the post construction settlement of the spread footings at SLS should be negligible.

The Precambrian bedrock surface is known to be highly irregular and can be irregular after blasting. As such, allowance should be made for additional formwork and concrete for footings bearing on bedrock. Furthermore, the surface of the bedrock in all pad footings should be made relatively flat before forming; this may require localized bedrock removal depending on the size of the irregularities.

As an alternative to the above, consideration could be given to founding the structure on a leveling layer of compacted granular material above the bedrock (engineered fill). The engineered fill should consist of at least 150 millimetres of OPSS Granular B Type II throughout the building footprint, and should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value. To allow adequate spread of load below the footings, the engineered fill should extend at least 0.3 metres beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter. Spread footings founded on compacted engineered fill above competent bedrock could be sized using Serviceability Limit State (SLS) and Ultimate Limit State (ULS) bearing pressures of 200 and 500 kilopascals, respectively. In this case, the post constructed total and differential settlement of the footings should be less than 25 and 20 millimetres, respectively, provided that all loose or disturbed soil is removed from below the footings.

5.3.3 Frost Protection of Foundations

At least 1.5 metres of earth cover should be provided for frost protection purposes for spread footings that are backfilled with well graded, non-frost susceptible sand or sand and gravel. Isolated footings constructed in areas that are to be cleared of snow during the winter period should be provided with at least 1.8 metres of earth cover. Where less than the required depth of soil cover can be provided, the footings can be protected from frost by using a combination of earth cover and extruded polystyrene insulation. Details regarding the insulation could be provided if necessary. The bearing pressures provided in Section 5.3.2 of this report may have to be reduced, depending on the type of insulation placed below the footings.

The depth of frost cover could also be reduced for footings founded on competent bedrock that is free of soil filled joints and seams. The frost susceptibility of the bedrock would have to be assessed by geotechnical personnel at the time of construction.

5.3.4 Seismic Design of Proposed Structure

The results of the shear wave velocity tests carried out in borehole 2 are provided in Appendix C. A summary of the shear wave velocities measured in the bedrock at borehole 2 are provided in Table 5.2.

Table 5.2 – Summary of Shear Wave Velocity Measurements

Strata	Measured Velocity (m/s)
Bedrock	900 to 2700 (upper fractured) 2100 to 3500 (competent)

Based on the planned underside of footing elevation of 113.3 metres, Block 3 will be founded on Precambrian bedrock, or a thin layer levelling layer of engineered fill above Precambrian bedrock. Based on the results of the shear wave velocity testing, Site Class A could be used for design purposes.

5.3.5 Basement Foundation Wall Backfill and Drainage

The basement foundation walls for the structures should be damp proofed and backfilled with free draining, non-frost susceptible granular materials, such as sand or sand and gravel conforming to Ontario Provincial Standard Specifications (OPSS) for Granular B Type I or Granular B Type II. Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the Granular B Type I backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment. Light walk behind compaction equipment should be used next to the foundation walls to avoid excessive compaction induced stress on the foundation walls. A nonwoven geotextile meeting OPSS 1860 Class II requirements should be provided between any sand backfill material and open graded rock fill.

A perforated plastic foundation drain with a surround of clear crushed stone should be installed below the level of the underside of the basement slab on the exterior of the foundation walls. The drain should outlet by gravity to a storm sewer or a sump from which the water is pumped. To avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the buildings), a nonwoven geotextile should be placed between the clear stone and any sand backfill material.

The static at rest thrust (P_o) acting on basement foundation walls that are backfilled with sand or sand and gravel should be calculated using the following formula:

$$P_o = 0.5\ K_o\ \gamma\ H^2$$

where;

- P_o: Static at rest thrust component (kilonewtons);
- γ: Moist material unit weight (kN/m^3);
- K_o: "At Rest" earth pressure coefficient;
- H: Wall height (metres).

Seismic shaking can increase the forces on the retaining wall. The total at rest thrust acting on the wall (P_{oe}) during a seismic event is composed of a static component (P_o) and a dynamic component (P_e), that is:

$$P_{oe} = P_o + P_e$$

The dynamic at rest thrust component (P_e), which acts only during seismic loading conditions, should be calculated using the following formula:

$$P_e = 0.5\ (K_{oe} - K_o)\ \gamma\ H^2$$

where;

- P_e: Dynamic at rest thrust component (kilonewtons);
- γ: Moist material unit weight (kN/m^3);
- K_o: "At Rest" earth pressure coefficient;
- K_{oe}: Dynamic at rest earth pressure coefficient;
- H: Wall height (metres).

The static thrust component (P_o) acts at a point located H/3 above the base of the wall. During seismic shaking, the dynamic at rest thrust component (P_e) acts at a point located about 0.6H above the base of the wall.

For design purposes, the soil parameters provided in Table 5.3 can be used to calculate the at rest thrust components acting on the walls.

Table 5.3 – Soil Parameters

Parameter	OPSS Granular B Type I or II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
"At Rest" Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.38
Dynamic At Rest Earth Pressure Coefficient, K_{oe}, assuming horizontal backfill behind the structure	0.38[1]

Notes:

1) According to the 2012 Ontario Building Code, the peak ground acceleration (PGA) for Ottawa is 0.32 for firm ground conditions (i.e., for Site Class C). For this particular site, the corrected PGA can be taken as 0.24 (Site Class A). The dynamic at rest earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal seismic coefficient, k_h, of 0.24 (taken as the corrected PGA) and assuming that the vertical seismic coefficient, k_v, is zero. It is noted that for this particular case, the calculated dynamic "At Rest" earth pressure coefficient (K_{oe}) during a seismic event is equal to the static "At Rest" earth pressure coefficient (K_o) calculated using conventional earth pressure theory. During seismic shaking the thrust component acts higher up the wall.

Heavy construction traffic should not be allowed to operate adjacent to the basement foundation walls for the proposed building (say within about 2 metres horizontal) during construction, without the approval of the designers.

5.3.6 Basement Concrete Slab Support

To provide predictable settlement performance of the basement slab, all loose soil or debris should be removed from the slab area. The base for the floor slab should consist of at least 150 millimetres of 19 millimetre clear crushed stone. Any necessary grade raise fill should consists of either 19 millimetre clear crushed stone or OPSS Granular B Type II. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II material. Since the source of recycled material cannot be determined or controlled, it is suggested that any imported Granular B Type II materials be composed of 100 percent crushed rock only.

The clear crushed stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor. The Granular B Type II should be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory equipment.

Underfloor drainage should be provided below the floor slab. If well graded granular material (such as OPSS Granular B Type II) is used below the basement floor slab, we suggest that drainage be provided by means of plastic perforated pipes spaced at about 6 metres horizontally or as required to link any hydraulically isolated areas in the basement. Drains are not considered essential if clear crushed stone is used below the floor slab, provided that the clear stone can outlet to the sump or sewers and drains are installed to link any hydraulically isolated areas in the basement.

Proper moisture protection with a vapour retarder should be used for any slabs where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The "Guide for Concrete Floor and Slab Construction", ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

Based on published information, there is potential for radon gas due to the decay of radioactive material in the Precambrian bedrock in the Ottawa area. The soil gas control methods as outlined in Section 9.13.4 and Supplemental Standard SB-9 of the Ontario Building Code should be used.

5.4 Final Bedrock Slopes

It is understood that the proposed development plans may include a bedrock slope along the west property line (i.e., the shared property line between the parkland to the west and the development). In this area, we recommend that the perimeter of the excavation along the shared property line be defined by line drilling on close centers in order to reduce over break and/or under break of the bedrock. Further, all loose bedrock should be scaled from the face of the bedrock after excavation. Notwithstanding, it is noted that cyclical freezing and thawing of water in open bedrock fractures over time could cause rock pieces to become dislodged. Where the bedrock slope is relatively steep, this could cause gravel, cobbles and boulder size pieces of rock to fall, roll or slide down the slope and potentially cause damage or injury. As such, the following alternatives (or combination thereof) could be considered for the final bedrock slopes at this site:

- Alternative 1: Swale and Fencing
    - The bedrock slopes could be left in place and scaled to remove any obvious loose or overhanging pieces.
    - A swale would be required at the toe of the bedrock slope to prevent cobble or boulder size material from rolling/sliding onto the sidewalk. The swale should be at least 0.5 metres deep, and should be at least 1 metre wide.
    - A fence should be included on the west side of the sidewalk to prevent access to the slope and swale area.

- Some rock material will accumulate in the swale with time, and may have to be removed if the amount becomes excessive.
- Fencing would be required at the top of the slope.

- Alternative 2: Slope Rock Face at about 1.5 Horizontal to 1 Vertical
  - Some of the bedrock slopes would have to be excavated at 1.5 horizontal to 1 vertical using hoe ramming techniques, drill and blasting techniques, or a combination of both.
  - The bedrock slope could be made smooth, where necessary, with compacted, well graded rock fill material.
  - Voids in the rock fill could be covered with a layer of clayey soil and topsoil, and then vegetated.
  - Fencing would be required at the top of the slope.

- Alternative 3: Flexible Retaining Wall at the Toe of the Bedrock Slope
  - A retaining wall system (such as an armourstone wall, or equivalent) could be constructed at the toe of the bedrock slope. The retaining wall could be founded at shallow depth on bedrock or, where required, on a pad of compacted granular material.
  - All obvious, loose bedrock or overhanging bedrock should be removed in advance of the retaining wall construction.
  - The slopes above the retaining wall, if any, could be constructed using 2 horizontal to 1 vertical, or flatter, slopes.
  - Fencing would be required at the top of the slope or retaining wall.

Additional guidelines for retaining wall construction at this site are provided in Section 5.5.

5.5 Proposed Retaining Walls

The type of retaining walls to be used at this site have not been finalized; however, the following general guidelines are provided for preliminary design purposes:

- Based on the results of the investigation, the proposed retaining walls will likely be founded on the Precambrian bedrock surface or on a levelling layer of compacted granular material above the bedrock (engineered fill). The engineered fill should consist of at least 150 millimetres of OPSS Granular B Type II, and should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value. To allow adequate spread of load below the footings, the engineered fill should extend at least 0.3 metres beyond the edge of the wall and down and out from this point at 1 horizontal to 1 vertical, or flatter. The bearing pressures provided in Section 5.3.2 are valid for the proposed retaining walls.

- The frost protection requirements provided in Section 5.3.3 apply equally to the proposed retaining walls.
- The retaining walls should be backfilled with imported, free draining, non-frost susceptible granular material such as that meeting OPSS Granular B Type I or II requirements. The free draining backfill should extend at least 1.2 metres horizontally from the inside face of the retaining wall and at least 0.6 metres horizontally from the outside face of the retaining wall, as shown on OPSD 3121.150.
- Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable compaction equipment. Where future landscaped areas will exist next to the walls and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.
- Where areas of hard surfacing (concrete, sidewalks, pavement, etc.) abut the proposed walls, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.8 metres below finished grade (or the bedrock surface) to the underside of the granular subbase for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.
- To prevent buildup of hydrostatic pressure behind the retaining wall, the wall backfill material should be drained with a perforated pipe that outlets by gravity to a storm sewer.
- The sliding resistance of the retaining walls could be calculated using an unfactored interface friction angle of 29 degrees for walls founded on engineered fill and 35 degrees for walls founded on competent bedrock.
- The horizontal earth pressures on the retaining walls will depend on a number of factors, including the type of retaining wall, the slope of the face of the retaining wall, the slope (inclination) of the backfill material behind the retaining wall, the amount of movement that occurs at the top of the wall etc. Parameters for calculating earth pressures on the walls could be provided as the development plans progress.

5.6 Proposed Services

5.6.1 Pipe Bedding

The bedding for service pipes should consist of at least 150 millimetres of crushed stone meeting OPSS requirements for Granular A. Cover material, from spring line to at least 300 millimetres above the tops of the pipes, should consist of granular material, such as that meeting OPSS Granular A. Some overbreak of the Precambrian bedrock should be expected and allowance should be made for thickening the bedding material, as required.

5.6.2 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (pavement, sidewalk, etc.), acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The depth of frost penetration in exposed areas can normally be taken as 1.8 metres below finished grade. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material, imported granular material conforming to OPSS Granular B Type I, or well shattered and graded blast rock.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Wood, stumps, topsoil, peat and organic materials should be wasted from the trench. If on site blast rock is used as backfill within the service trench, it should be mostly 300 millimetres, or smaller, in size and should be well graded. To prevent ingress of fine material into voids in the blast rock, the upper surface of the blast rock should be blinded with well graded crushed stone, such as OPSS Granular B Type II.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, driveways, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Rock fill should be placed in maximum 500 millimetre thick lifts and compacted with the haulage and spreading equipment. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures.

Some of the overburden materials may have moisture contents above optimum for compaction. Furthermore, the silty clay, silty sand, and glacial till overburden deposits at this site are sensitive to changes in moisture content. Unless these materials are allowed to dry, the specified densities will not likely be possible to achieve and, as a consequence, some settlement of these backfill materials could occur. Consideration could be given to implementing one or a combination of the following measures to reduce post construction

settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction;
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of any roadways (i.e., Superpave 12.5 asphaltic concrete placement) for 6 months, or longer, to allow the trench backfill settlement to occur and thereby improve the final roadway appearance; OR
- Reuse any wet materials outside hard surfaced areas and where post construction settlement is less of a concern (such as landscaped areas).

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In order to carry out the work during freezing temperatures and maintain adequate performance of the trench backfill as a roadway subgrade, the service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

5.6.3 Seepage Barriers

Seepage barriers should be installed along the service trenches just inside the property lines to prevent groundwater flow along the granular bedding and backfill materials for the service pipes. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to above the groundwater level, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value.

5.7 Access Roadways and Parking Areas

5.7.1 Grade Raise Filling and Subgrade Preparation

In preparation for the construction of the parking areas, any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface.

Should it be necessary to raise the grades above the native soil or bedrock, the grade raise fill could consist of material which meets OPSS specifications for Granular B Type I, Granular B Type II, Select Subgrade Material, suitable earth borrow, or rock fill. The Granular B Type I, Granular B Type II, Select Subgrade Material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value

using vibratory compaction equipment. It is noted, however, that most of the earth borrow materials in the Ottawa area are sensitive to changes in moisture content, precipitation and frost heaving. As such, unless the earth material placement is planned during the dry period of the year (June to September), precipitation and freezing conditions may restrict or delay adequate compaction of these materials. Based on our experience, the silty clay/clayey silt materials should be compacted within 0 to 4 percent above the optimum moisture content, as determined by the standard Proctor test, to reduce the post construction settlement of the fill material. Depending on the weather conditions, it may be necessary to allow the fill material to dry prior to compaction.

Any rock fill material should consist of well graded and shattered material having a maximum particle size of generally less than 300 millimetres. The rock fill should be compacted in maximum 0.5 metre thick lifts using the haulage and spreading equipment in combination with a large (10 tonne) steel drum roller.

Fill placement should be carried out uniformly across the pavement areas to avoid potential differential frost heaving between different fill materials.

The subgrade should be shaped and crowned to promote drainage of the roadway granular materials.

5.7.2 Pavement Design

It is suggested that parking areas for light vehicles (automobiles, etc.) be constructed using the following minimum pavement structure:

50 millimetres of OPSS Superpave 12.5 asphaltic concrete, over

150 millimetres of OPSS Granular A base, over

300 millimetres of OPSS Granular B Type II subbase

For any heavy use access roadways, the asphaltic concrete surfacing thickness should be increased to 90 millimetres (40 millimetres of Superpave 12.5 over 40 millimetres of Superpave 12.5 asphaltic concrete) and the thickness of the subbase layer increased to 450 millimetres.

In areas where the proposed pavement structure directly overlies at least 0.5 metres of clean blast rock fill material or bedrock, the subbase thickness could be reduced to at least 150 millimetres of OPSS Granular B Type II material. For construction of a pavement structure above the proposed underground parking facility, the granular material should consist of OPSS Granular O, as opposed to OPSS Granular A. From a pavement design perspective, we recommend that the rigid polystyrene insulation used in the parking areas above the underground parking facility consist of DOW Styrofoam High Load 40 or an approved alternate.

The above pavement structure assumes that the trench backfill is adequately compacted and that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the Granular B Type II, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance of the subbase material.

5.7.3 Granular Material Compaction

The granular base and subbase materials should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.

5.7.4 Transition Treatments

In areas where the new pavement structure will abut existing pavements, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

Granular frost tapers in accordance with OPSS 205.030/205.040 should be used where the subgrade below the roadway or parking lots transitions from frost susceptible soil to bedrock/rock fill.

5.7.5 Asphaltic Concrete Type

Performance grade PG 58-34 asphaltic concrete could be specified to reduce the potential for thermal cracking of the asphaltic concrete. The Superpave asphaltic concrete mixes should be designed for Traffic Level B.

5.7.6 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. Where storm sewers are used to convey surface water runoff, catch basins should be provided with minimum 3 metre long rigid perforated stub drains which extend in at least two directions from each catch basin at pavement subgrade level.

Full length subdrains should be used along the edges of any roadways or parking lots that are in cut section or are located at the bottom of a slope.

6.0 ADDITIONAL CONSIDERATIONS

6.1 Winter Construction

In the event that construction is required during freezing temperatures, the subgrade surface below the proposed buildings should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

Provision must be made to prevent freezing of any soil below the level of any footings, slabs or services. Freezing of the soil could result in heaving related damage.

6.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, blasting, foundation construction etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during the construction so that any construction related claims can be dealt with in a fair manner.

6.3 Disposal of Excess Soil

It is noted that the professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination, including naturally occurring source of contamination, are outside the terms of reference for this report. This report does not constitute a Phase II Environmental Site Assessment (ESA) nor does it constitute a contaminated material management plan.

6.4 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

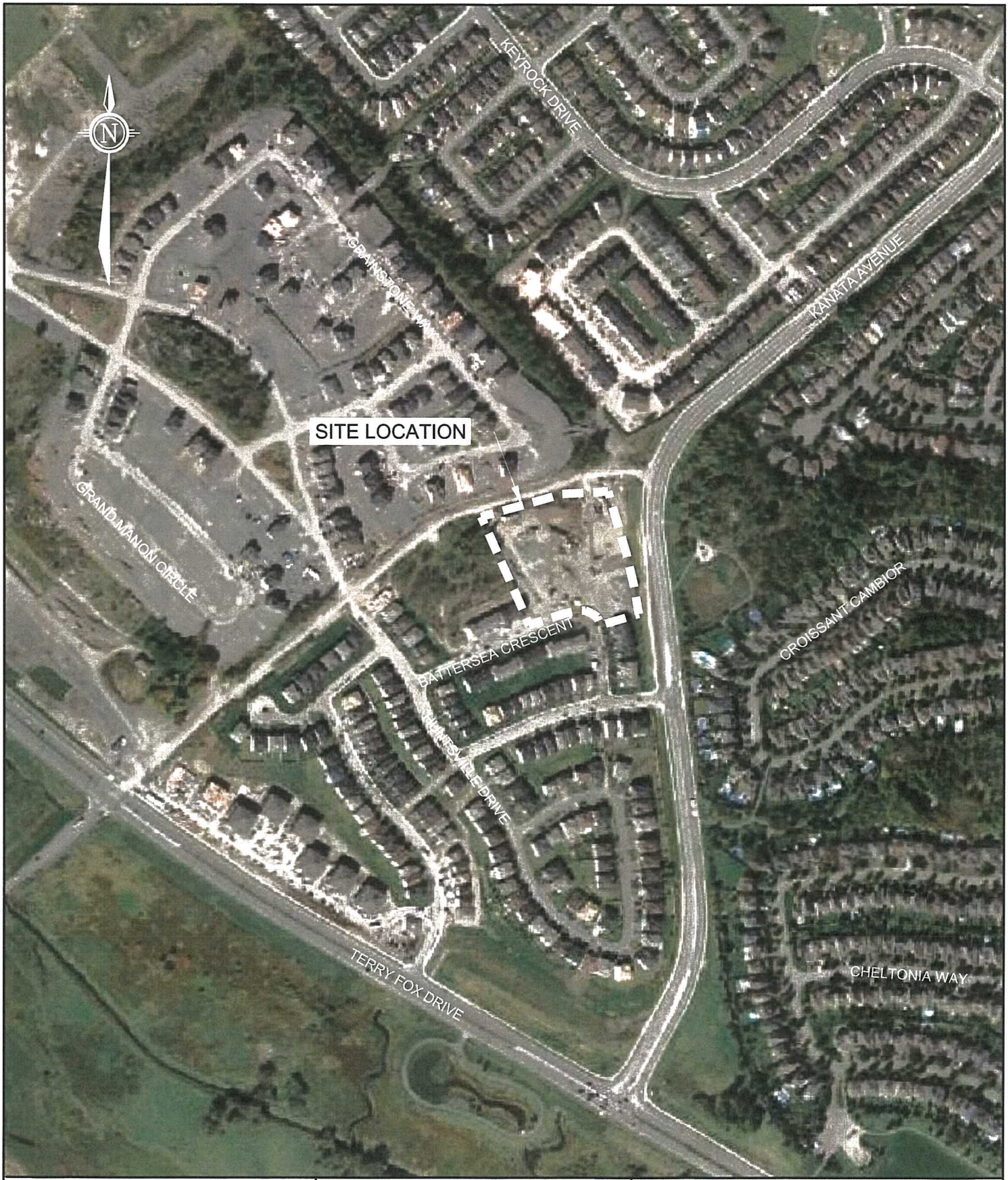
The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not

adversely affect the intent of the design. The subgrade surfaces for all buildings, site services and roadways should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications. In accordance with Ontario Building Code requirements, full time compaction testing is required for engineered fill below buildings.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Johnathan A. Cholewa, Ph.D., P.Eng.

Andrew Chevrier, M.Eng., P.Eng.
Senior Geotechnical Engineer



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Project	Drawing
SUBSURFACE INVESTIGATION HERITAGE HILLS SUBDIVISION 124 BATTERSEA CRESCENT OTTAWA, ONTARIO	KEY PLAN

Drwn By	Chkd By	Date	Project No.	Revision No.	
D.J.R.	J.C.	DECEMBER 2016	62970.01	0	**FIGURE 1**

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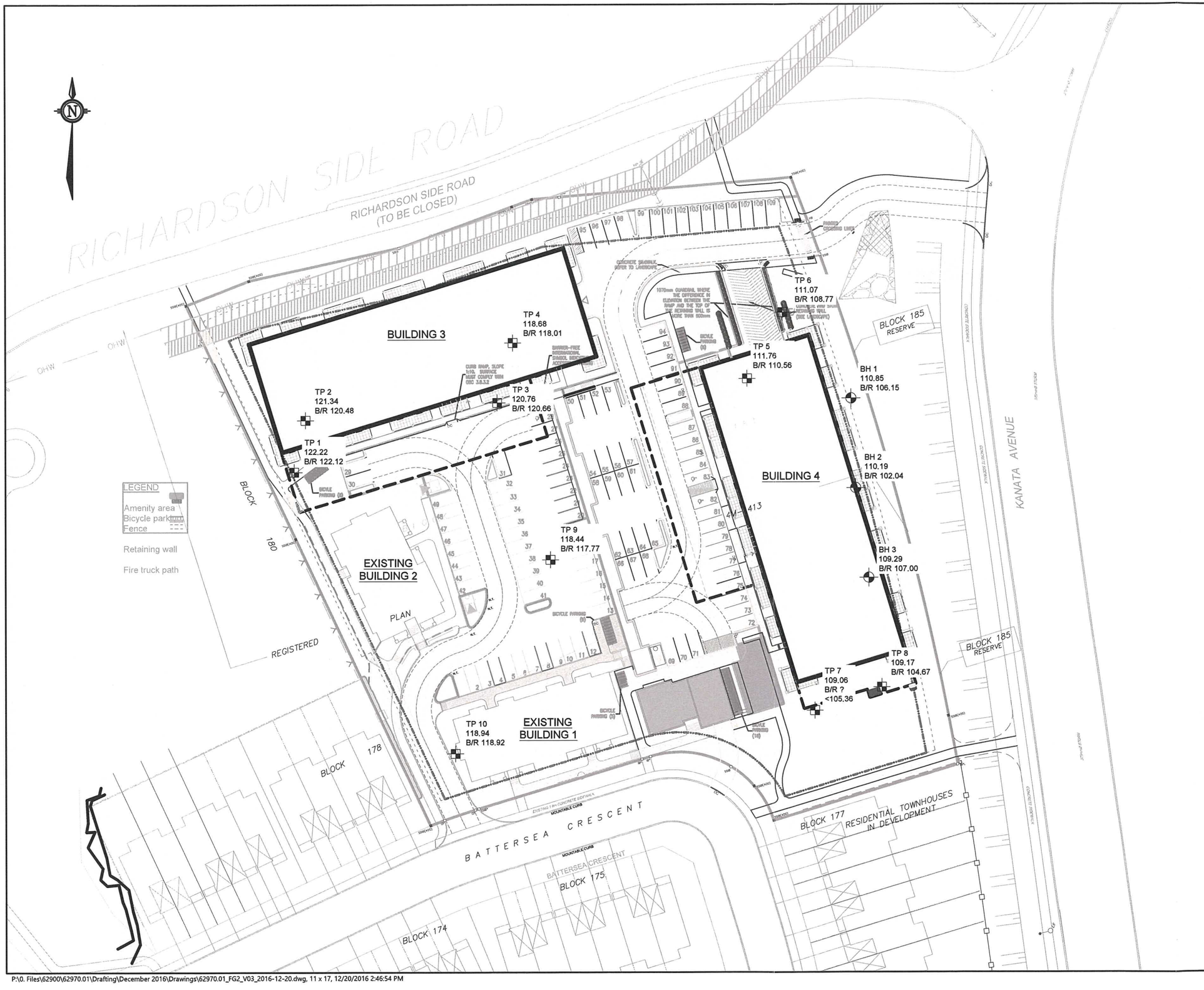

LEGEND

- BH 1 99.99 — BOREHOLE LOCATION IN PLAN (current investigation by Houle Chevrier Engineering Ltd.)
- TP 1 99.99 — TEST PIT LOCATION IN PLAN (current investigation by Houle Chevrier Engineering)
- B/R 99.99 — BEDROCK ELEVATION, IN METRES

Scale 1:750

0 15 30 45m

Houle Chevrier Engineering

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Client: NOVATECH		Project: 62970.01	
Location: HERITAGE HILLS, OTTAWA, ON			
Drwn by: P.C.	Chkd by: J.C.	SITE PLAN	
Date: DECEMBER 2016	Rev.: 0	FIGURE 2	

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APPENDIX A

List of Abbreviations and Terminology
Bedrock Description and Terminology
Record of Test Pit Sheets

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS auger sample
CA casing sample
CS chunk sample
BS Borros piston sample
DO drive open
MS manual sample
RC rock core
ST slotted tube
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimetre required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60° cone attached to 'A' size drill rods for a distance of 300 mm.

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u undrained shear strength
e void ratio
C_c compression index
c_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_P plastic limit
ϕ^1 effective angle of friction
γ unit weight of soil
γ^1 unit weight of submerged soil
σ normal stress

BEDROCK DESCRIPTION TERMINOLOGY

STATE OF WEATHERING

Fresh: no visible sign of weathering.
Faintly weathered: weathering limited to the surfaces of major discontinuities.
Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.
Moderately weathered: weathering extends throughout the rock mass, but the rock material is not friable.
Highly weathered: weathering extends throughout the rock mass and the rock material is partly friable.
Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

CORE CONDITION

Total Core Recovery (TCR): The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR): The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD): The percentage of solid drill core, greater than 100 mm in length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

PROJECT: 11-070

RECORD OF TEST PIT 1

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural, V - + / Remoulded, V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp ⊖ W WI (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		122.22					
	Dark brown silty sand TOPSOIL		122.12 / 0.10					
	Practical refusal to excavating on bedrock End of test pit							Test pit dry upon completion of excavation on February 25, 2008.
1								
2								
3								
4								
5								

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

PROJECT: 11-070

RECORD OF TEST PIT 2

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + / Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		121.34					
	Dark brown amorphous PEAT		120.80					
	Grey brown silty sand, some gravel and cobbles (GLACIAL TILL)		0.54 120.48					
1	Practical refusal to excavating on bedrock End of test pit		0.86					Groundwater inflow from ground surface on February 25, 2008.
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 3

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		120.76					
	Dark brown silty sand TOPSOIL		120.66					
	Practical refusal to excavating on bedrock End of test pit		0.10					Test pit dry upon completion of excavation on February 25, 2008.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

RECORD OF TEST PIT 4

PROJECT: 11-070

LOCATION: Refer to Site Plan, Figure 2

DATE OF EXCAVATION: February 25, 2008

SHEET 1 OF 1

DATUM: N/A

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp ⊢—○ W —⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		118.68					
	Dark brown amorphous PEAT		118.01					
	Practical refusal to excavating on bedrock End of test pit		0.67					Groundwater inflow from ground surface on February 25, 2008.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

RECORD OF TEST PIT 5

PROJECT: 11-070

LOCATION: Refer to Site Plan, Figure 2

DATE OF EXCAVATION: February 25, 2008

SHEET 1 OF 1

DATUM: N/A

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		111.76					
	Dark brown silty sand, some roots TOPSOIL		111.61 / 0.15					
	Grey brown silty sand, some gravel and cobbles (GLACIAL TILL)		111.13 / 0.63					
1	Grey silty sand, some gravel and cobbles (GLACIAL TILL)		110.56 / 1.20					Groundwater inflow at approximately 0.5 metres below ground surface on February 25, 2008.
	Practical refusal to excavating on bedrock End of test pit							
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 6

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural, V - ; Remoulded, V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		111.07					
	Dark brown silty sand TOPSOIL		110.97					
	Grey brown SILTY SAND, some gravel and cobbles		0.10 110.68					
	Grey brown SILTY CLAY (Weathered Crust)		0.39 109.94					
1								
	Grey brown silty sand, some gravel and cobbles (GLACIAL TILL)		1.13 108.77					
2								
	End of test pit Refusal on bedrock		2.30					Groundwater level in standpipe could not be measured on March 20, 2008.
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 7

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp W WI (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		109.06					
	Dark brown silty sand TOPSOIL		108.88 / 0.18					
	Grey brown SILTY SAND, some gravel		108.26 / 0.80					
1	Grey brown SILTY CLAY (Weathered Crust)							
2			107.06 / 2.00					
	Estimated stiff to very stiff, blocky, grey brown SILTY CLAY							
3			105.86 / 3.20					
	Grey brown silty sand, some gravel and cobbles (GLACIAL TILL)		105.36 / 3.70					
	End of test pit							Groundwater inflow at approximately 1.3 metres below ground surface on February 25, 2008.
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 8

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + ; Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp ⊢ W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		109.17					
	Dark brown silty sand TOPSOIL		109.05 / 0.12					
	Grey brown SILTY SAND, some gravel		108.90 / 0.27					
	Grey brown SILTY CLAY (Weathered Crust)							
1								
			107.37 / 1.80					
2	Estimated stiff to very stiff, blocky, grey brown SILTY CLAY							
3								
4			105.02 / 4.15					
	Grey brown silty sand, some gravel and cobbles (GLACIAL TILL)							
			104.67 / 4.50					
	End of test pit Refusal on bedrock							
5								

Groundwater level in standpipe could not be measured on March 20, 2008.

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 9

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + / Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		118.44					
	Dark brown silty sand TOPSOIL		118.28 / 0.16					
	Red brown silty sand, some gravel and boulders			1				
	End of testpit Refuasl on fractured bedrock		117.77 / 0.67					Test pit dry upon completion of excavation on February 25, 2008.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

PROJECT: 11-070

RECORD OF TEST PIT 10

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: N/A

DATE OF EXCAVATION: February 25, 2008

TYPE OF EXCAVATOR: Backhoe

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) — Natural. V - + ; Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) — Wp ⊖ W WI (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		118.94					Test pit dry upon completion of excavation on February 25, 2008.
	Dark brown silty sand TOPSOIL		0.02					
	End of test pit Refusal on bedrock							
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: B.W.

CHECKED: A.C.

TESTPIT LOG 11-070 TP LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 6-19-15

APPENDIX B

Record of Borehole Sheets

PROJECT: 11-070

RECORD OF BOREHOLE 1

SHEET 1 OF 1

LOCATION: Refer to site plan, figure 2

DATUM: N/A

BORING DATE: May 1, 2008

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat. V - +, rem. V - ⊕, Q - ●, U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5} 10^{-4} 10^{-3} 10^{-2}) / WATER CONTENT, PERCENT (Wp, W, Wl) (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		110.85							
		Dark brown sandy silt with roots and organics (TOPSOIL)		110.39							Backfilled with soil cuttings
		Probable weathered SILTY CLAY		0.46							
1											
2											
3											
4				106.28							
		Probable GLACIAL TILL		4.57							
				106.15							
		End of borehole Practical auger refusal on inferred bedrock		4.70							
5											

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

LOGGED: J.M.

CHECKED: A.C.

BOREHOLE LOG 11-070 BH LOGS.GPJ HOULE CHEVRIER 2015.GDT 6-19-15

PROJECT: 11-070

RECORD OF BOREHOLE 2

SHEET 1 OF 1

LOCATION: Refer to site plan, figure 2

DATUM: N/A

BORING DATE: May 1-2, 2008

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat. V - +, rem. V - ⊕, Q - ●, U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5} 10^{-4} 10^{-3} 10^{-2}) / WATER CONTENT, PERCENT (Wp, W, Wl) (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		110.19							Cement backfill
		Dark brown sandy silt with roots and organics (TOPSOIL)		109.73 / 0.46							
1	Power Auger 200 mm Diameter Hollow Stem	Very stiff grey brown SILTY CLAY, trace sand			1	50 DO	12				
2					2	50 DO	18				
3					3	50 DO	6				
				106.84	4	50 DO	3				
4		Very loose to compact grey silty clay with sand and sand seams, trace gravel (GLACIAL TILL)		3.35	5	50 DO	2				
5					6	50 DO	9				64mm diameter PVC standpipe
6					7	50 DO	10				
					8	50 DO	8				
7					9	50 DO	7				
8				102.04							
		Competent grey QUARTZITE BEDROCK		8.15							
9											
10											
11											
12											
13	Rotary Drilling NQ casing										
14											
15											
16											
17											
18											
				91.27							
19		End of borehole		18.92							
20											

DEPTH SCALE

1 to 100

Houle Chevrier Engineering

LOGGED: J.M.

CHECKED: A.C.

BOREHOLE LOG 11-070 BH LOGS.GPJ HOULE CHEVRIER 2015.GDT 6-19-15

PROJECT: 11-070

RECORD OF BOREHOLE 3

SHEET 1 OF 1

LOCATION: Refer to site plan, figure 2

DATUM: N/A

BORING DATE: May 5, 2008

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (20 40 60 80) nat. V - + Q - ● rem. V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5} 10^{-4} 10^{-3} 10^{-2}) / WATER CONTENT, PERCENT (Wp — W — Wl, 20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		109.29							Backfilled with soil cuttings
		Conditions not observed									
1											
2											
				107.00							
		End of borehole Practical auger refusal on probable bedrock		2.29							
3											
4											
5											

DEPTH SCALE

1 to 25

Houle Chevrier Engineering

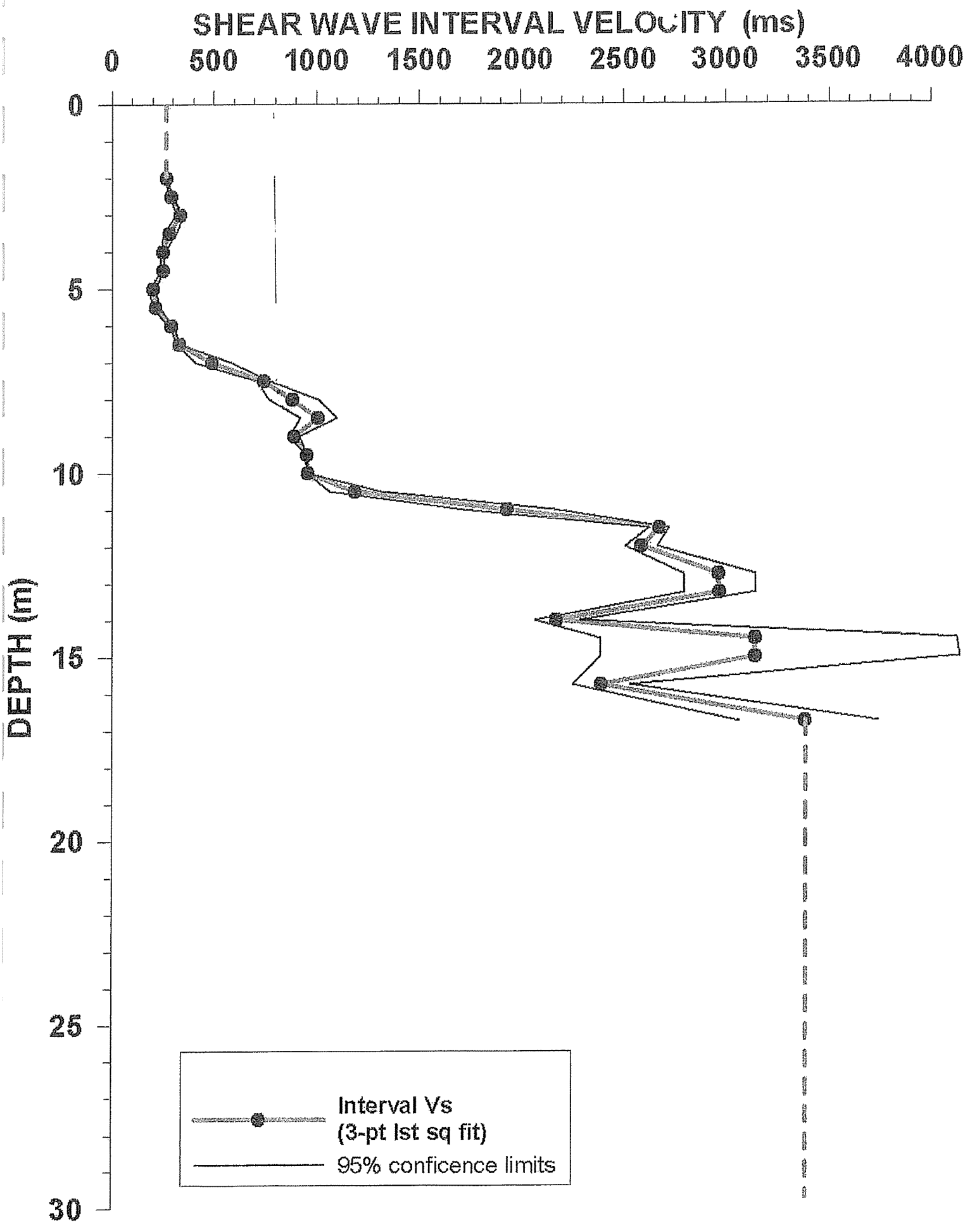
LOGGED: B.W.

CHECKED: A.C.

BOREHOLE LOG 11-070 BH LOGS.GPJ HOULE CHEVRIER 2015.GDT 6-19-15

APPENDIX C

Shear Wave Velocity Test Results


SHEAR WAVE INTERVAL VELOCITY (ms)
0
500
1000
1500
2000
2500
3000
3500
4000
DEPTH (m)
0
5
10
15
20
25
30
Interval Vs
(3-pt lst sq fit)
95% conficence limits