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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
485 RICHMOND ROAD
OTTAWA, ONTARIO

Submitted to:

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed residential building at 485 Richmond Road 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 boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the proposed building and site services, including construction considerations that could influence design decisions.

This investigation was carried out in accordance with our proposal dated October 4, 2010.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct a residential building at 485 Richmond Road in Ottawa, Ontario (see Key Plan, Figure 1). The site is currently occupied by an asphaltic concrete surfaced parking lot which services the adjacent office building at 495 Richmond Road. At the time of this report, the preliminary plans included a twenty-four (24) storey structure, which includes an above ground parking facility consisting of three (3) levels. One (1) level of underground parking is also included. It is understood that surface parking is not included in the scope of the project.

The following buildings currently exist adjacent to the proposed site (see Borehole Location Plan, Figure 2):

- Multi-storey office building located west of the proposed site at 495 Richmond Road.
- Single-storey commercial building east of the proposed site at 471 Richmond Road
- Multi-storey condominium building located south of the proposed site at 491 Richmond Road.

The preliminary plans indicate that the proposed residential building will be constructed within about 2 to 4 metres of the existing office building at 495 Richmond Road.

Based on the preliminary information provided to us, the proposed building will be founded at about elevation 60.35 metres, geodetic datum.

2.2 Previous Investigation by Morey Houle Chevrier Engineering Ltd.

A previous geotechnical investigation was carried for the construction of the existing condominium building at 491 Richmond Road by Morey Houle Chevrier Engineering Ltd. At that time, seven (7) boreholes were advanced at the site. The boreholes showed that the depth of the bedrock surface ranged between 0.9 and 1.4 metres below ground existing ground surface. In addition, one (1) borehole, numbered borehole 1, was advanced at the site of the proposed residential building to about 8.8 metres below ground surface. The borehole showed that the site of the proposed residential building is underlain by fill material, glacial till and then grey limestone bedrock. The depth of the bedrock was found to be 4.9 metres below ground surface.

The approximate location of borehole 1 is shown on the Borehole Location Plan, Figure 2. Copies of relevant borehole logs from that investigation are provided in Appendix B for reference.

2.3 Review of Geology Maps

Based on available surficial geology maps for the Ottawa area, the overburden in the vicinity of the subject site consists of deposits of glacial till. Fill material associated with previous construction activities on the site is also anticipated.

Bedrock geology maps indicate that the overburden deposits are underlain by limestone bedrock of the Gull River formation at depths ranging from about 2 to 3 metres. In addition, bedrock geology maps show that northwest-southeast aligned bedrock faults exist north and south of the site. Based on geological records, the shallow bedrock faults in the Ottawa area are currently not active.

3.0 SUBSURFACE INVESTIGATION

3.1 Geotechnical Investigation

The field work for this investigation was carried out between October 26 and 29, 2010. During that time, seven (7) boreholes, numbered 10-1 to 10-7, inclusive, were advanced at the site using a truck mounted, drill rig supplied and operated by supplied and operated by George Downing Estate Drilling Ltd. of Hawkesbury, Ontario. Details for the boreholes are provided below:

- Four (4) boreholes, numbered 10-2, 10-4, 10-6, and 10-7, were advanced to between 15.2 and 15.4 metres below ground surface for foundation design purposes. Boreholes 10-2, 10-4, 10-6, and 10-7 were advanced using both hollow stem auger and rotary diamond drilling techniques.

- Three (3) boreholes, numbered boreholes 10-1, 10-3, and 10-5, were advanced to practical refusal on or within inferred bedrock to provide additional information on the depth of the bedrock. Boreholes 10-1, 10-3, and 10-5 were advanced using hollow stem auger drilling techniques.

Standard penetration tests were carried out where possible within the overburden deposits and samples of the soils encountered were recovered using drive open sampling equipment. The underlying bedrock was cored in boreholes 10-2, 10-4, 10-6, and 10-7 using N size rotary diamond drilling equipment to identify the type and quality of the bedrock. Well screens were sealed in the bedrock at boreholes 10-4 and 10-7, and in the overburden at boreholes 10-3 and 10-6 to measure the groundwater levels, carry out hydraulic conductivity testing, and to allow groundwater sampling. Three samples of the groundwater were recovered from boreholes 10-4, 10-6, and 10-7 and sent to Exova Accutest for basic chemical testing relating to corrosion of buried concrete and steel. The field work was supervised throughout by a member of our engineering staff.

Following the borehole drilling work, the soil and bedrock samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content and grain size distribution. Selected samples of the bedrock were tested to determine its unconfined compressive strength.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the chemical analysis of the groundwater samples relating to corrosion are provided in Appendix D.

The borehole locations and elevations were measured using global positioning system surveying techniques by Houle Chevrier Engineering Ltd. personnel. The elevations are referenced to Geodetic datum.

3.2 Hydraulic Conductivity Testing

Rising head tests were carried out on November 9, 2010 in the well screens sealed in the bedrock at boreholes 10-4 and 10-7, and in the well screens sealed in the overburden at borehole 10-6. Rising head testing was not carried out in borehole 10-3 because of the limited quantity of water observed in the well screen. The rising heads tests were carried out in order to estimate the hydraulic conductivity of the soil and bedrock within the anticipated depth of the excavation for the proposed building. The well screens, having an internal diameter of about 32 millimetres, were installed within a surround of filter sand. Above the surround of filter sand, bentonite pellets were used to seal the bedrock surface (boreholes 10-4 and 10-7) or to seal the underlying bedrock (boreholes 10-6). Details of the well screens are provided on the Record of Borehole sheets in Appendix A.

The well screens were purged using a D-25 waterra foot valve pump for about 2 to 5 minutes and the rate of groundwater flow from the surrounding soil or bedrock into the borehole was calculated by recording the time for the groundwater level in the borehole, following pumping, to rise to a given level (relative to the static groundwater level). The results of the hydraulic conductivity testing are provided in Appendix C.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the subsurface conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The 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 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 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 boreholes advanced during this investigation.

4.2 Pavement Structure in the Parking Lot

All of the boreholes were advanced within the existing asphaltic concrete surfaced parking lot and encountered a pavement structure composed of 20 to 80 millimetres of asphaltic concrete followed by 0.4 to 1.2 metres of base/subbase material. At boreholes 10-1, 10-2, 10-3, 10-6, and 10-7, the base/subbase material consists of grey, crushed sand and gravel. At boreholes 10-4 and 10-5, the base/subbase material consists of grey brown sand and gravel.

The water content of the base/subbase material ranges from 3 to 4 percent.

4.3 Fill Material

Fill material, having a thickness of between 0.3 and 1.1 metres, was encountered below the parking lot pavement structure in boreholes 10-1, 10-2, 10-3, 10-6, and 10-7. At boreholes 10-2, 10-3, and 10-6, the fill material is composed of brown sand with variable amounts of gravel. At boreholes 10-1 and 10-7, the fill is composed of dark brown, brown, and grey brown sandy silt and silty sand with variable amounts of gravel and clay.

A standard penetration test carried out in the fill material encountered in borehole 10-6 gave an N value of 6 blows per 0.3 metres of penetration, which reflects a loose relative density.

The water content of the fill material ranges from 4 to 19 percent.

4.4 Silty Clay

The fill material at borehole 10-7 is underlain by a native deposit silty clay. The silty clay is weathered grey brown and has a thickness of 0.9 metres. A standard penetration test carried out in the silty clay gave an N value of 16 blows per 0.3 metres of penetration, which reflects a very stiff consistency.

The water content of a sample of the silty clay recovered from borehole 10-7 was 34 percent.

4.5 Glacial Till

A deposit of glacial till was encountered below the fill material in boreholes 10-2, 10-3, and 10-6, below the base/subbase materials in borehole 10-4, and below the silty clay in borehole 10-7, at depths of about 1.1 to 2.1 metres below ground surface (elevation 58.6 to 61.4 metres, geodetic datum). The glacial till can be generally described as silty sand with variable amounts of clay and gravel. In boreholes 10-2 and 10-6, the glacial till is grey brown and grey in colour, respectively. In boreholes 10-3, 10-4, and 10-7, the glacial till transitions from grey brown to grey at between 2.7 and 2.9 metres below ground surface (elevation 58.7 to 59.4 metres, geodetic datum). Cobbles and boulders should also be expected in the glacial deposit. The glacial till deposit has a thickness of about 1.1 to 3.8 metres.

Standard penetration tests carried out in the glacial till gave N values of 1 to 33 blows per 0.3 metres of penetration, which reflect a variable, very loose to dense relative density. The lower N values (i.e., between 1 and 10 blows per 0.3 metres of penetration) were measured in the grey glacial till. Standard penetration test results of 50 blows or greater for less than 0.3 metres of penetration in boreholes 10-2 and 10-7 likely reflect the presence of cobbles and boulders in the glacial till.

Grain size distribution curves for samples of the glacial till recovered from boreholes 10-3 and 10-7 are provided on Figure A1 in Appendix A. The water content of the glacial till ranges from 8 to 14 percent.

4.6 Bedrock

Practical refusal to further advancement of the hollow stem auger occurred in boreholes 10-1 and 10-5 at depths of between about 0.8 and 1.6 metres (elevation 61.1 to 61.3 metres, geodetic datum). At borehole 10-3, practical refusal to further advancement of the hollow stem auger occurred at 5.4 metres below ground surface (elevation 56.2 metres, geodetic datum). It should be noted that practical auger refusal can sometimes occur within cobbles and boulders and may not necessarily be representative of the upper surface of the bedrock.

Bedrock was encountered and cored in boreholes 10-2, 10-4, 10-6, and 10-7 at depths of 3.0 to 5.0 metres below ground surface (elevation 55.8 to 60.3 metres, geodetic datum). The following observations were made with respect to the bedrock:

- The upper 0.7 to 1.2 metres of the bedrock in boreholes 10-2 and 10-6 is fractured, as indicated by a total core recovery (TCR) values of 69 to 96 percent, solid core recovery (SCR) values of 39 to 46 percent, and rock quality designation (RQD) values of 0 to 16 percent.

- At borehole 10-2, the bedrock consists of fresh, medium to thickly bedded, grey, greenish grey, and dark grey interbedded limestone, dolostone, and shale with occasional near vertical joints. The bedrock core recovered below the fractured zone in borehole 10-2 is of fair to excellent quality as indicated by TCR values of 97 to 100 percent, SCR values of 80 to 100 percent, and RQD values of 55 to 100 percent. A mud seam, having a thickness of about 15 millimetres, was encountered in borehole 10-2 at 14.9 metres below ground surface (elevation 48.3 metres, geodetic datum).

- At boreholes 10-4, 10-6, and 10-7, the bedrock consists of fresh, thinly to thickly bedded, grey and dark grey interbedded limestone and shale with occasional near vertical joints, followed by fresh, medium to thickly bedded, greenish grey and dark grey interbedded dolostone and shale. The transition from interbedded limestone and shale bedrock to interbedded dolostone and shale bedrock occurs between about elevation 51.6 to 52.2 metres, geodetic datum. The bedrock core recovered from boreholes 10-4, 10-7, and below the fractured zone in borehole 10-6 is of fair to excellent quality as indicated by TCR values of 95 to 100 percent, SCR values of 76 to 100 percent, and RQD values of 69 to 100 percent.

The unconfined compressive strength of twelve samples of the interbedded limestone, dolostone, and shale bedrock are summarized in the following table:

Elevation (metres, geodetic datum)	Borehole	Lithology	Unconfined Compressive Strength (MPa)
57.1	10-7	Limestone	96
54.3	10-7	Shale	53
53.9	10-4	Dolostone	95
52.8	10-2	Limestone	102
52.6	10-4	Limestone	77
51.7	10-2	Dolostone	63
50.6	10-4	Dolostone	128
50.3	10-6	Dolostone	88
50.2	10-7	Dolostone	84
49.2	10-6	Dolostone	70
49.1	10-7	Dolostone	83
48.5	10-7	Shale	54

Photographs of the bedrock core recovered from boreholes 10-2, 10-4, 10-6 and 10-7 are provided on Figures 3 to 6, inclusive.

4.7 Groundwater Levels

The groundwater levels measured in the well screens installed in the overburden (boreholes 10-3 and 10-6) ranged between 2.2 and 4.2 metres below ground surface (elevation 57.3 to 58.5 metres, geodetic datum) on November 15, 2010. The groundwater levels measured in the well screens installed in the bedrock (boreholes 10-4 and 10-7) ranged between 3.7 and 4.8 metres below ground surface (elevation 57.2 to 57.8 metres, geodetic datum) on November 15, 2010.

The groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

4.8 Groundwater Chemistry Relating to Corrosion

The results of chemical testing on groundwater samples recovered from the well screens installed in boreholes 10-4, 10-6, and 10-7 are provided in Appendix C and summarized in the following table:

Parameter	Borehole		
	10-4 (Bedrock)	10-6 (Overburden)	10-7 (Bedrock)
Conductivity (micromhos/centimetre)	3080	5180	2620
pH	8.16	7.69	8.22
Sulphate Content (mg/L)	569	116	412
Chloride Content (mg/L)	516	1340	439

4.9 Hydraulic Conductivity Test Results

During the rising head test carried out in the overburden (borehole 10-6), the groundwater level in the borehole rose to 0.9 metres below the initial groundwater level 10 minutes after the well screen was purged dry. Three hours after the well screen was purged dry, the groundwater level in the borehole recovered nearly to the initial groundwater level (i.e., to about 40 millimetres below the initial groundwater level).

During the rising head tests carried out in the bedrock (boreholes 10-4 and 10-7), the groundwater level in the boreholes rose to between 8.8 and 10.0 metres below the initial groundwater level 10 minutes after the well screens were purged dry. Three hours after the well screens were purged dry, the groundwater level in the borehole recovered to between 3.2 and 3.4 metres below the initial groundwater level.

The results of the hydraulic conductivity testing are provided in Appendix C.

5.0 PROPOSED RESIDENTIAL BUILDING

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 for the Proposed Building

5.2.1 Overburden Excavation

The excavation for the proposed building will be carried out mostly through asphaltic concrete, granular material, fill material, silty clay, glacial till and interbedded limestone, dolostone, and shale bedrock.

The sides of the excavations in overburden should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soil at this site can be classified as Type 3 soil and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending from the bottom of the excavation with soil overburden.

In areas where space constraints dictate, and an open excavation is required, the sides of excavation in overburden could be supported using a shoring system, such as a pile and lagging shoring wall, driven interlocking steel sheet piles, secant concrete pile wall, or a concrete diaphragm wall. The retaining walls should be suitably tied back with tensioned rock anchors. It should be noted that the glacial till contains cobble and boulder obstructions which could affect the shoring installation. For design and costing purposes, allowance should be made to socket the soldier piles for a pile and lagging wall into the bedrock using predrilled holes. Any boulders encountered in the drill holes will have to be broken and removed. For a steel sheet pile wall, hard driving conditions should be expected and some of the sheet piles may terminate within the glacial till on or within cobbles and boulders and it may be necessary to remove the obstructions and redrive the sheet piles.

The piles for a pile and lagging wall should be set back at least 1 metre from the edge of the bedrock excavation, if the piles are terminated 1 metre into the bedrock; a similar set back allowance would be required for a steel sheet pile wall. The setback allowance could be omitted for a pile and lagging wall, if the piles are socketed below the bottom of the excavation.

The type of shoring used on this project should be based on the permissible movement behind the shoring as well as space constraints. Some unavoidable inward horizontal movement and settlement of the ground behind the retaining walls should be anticipated, which could affect existing structures and services located behind the retaining walls. As part of the selection and design of the shoring system(s), the effects of the settlement on surrounding structures and services must be assessed. As a guide for design only, the amount of movement behind tied back pile and lagging walls could be in the order of 0.2 to 0.5 percent of the depth of the excavation in overburden; this assumes that all voids are filled behind the lagging and provided that good workmanship is used. A nonwoven geotextile filter should be installed behind the lagging for a pile and lagging wall, in particular below the groundwater level, to reduce the migration of the very loose to loose glacial till through gaps in the wood lagging. The amount of settlement behind secant concrete pile and diaphragm walls will be much less.

Temporary, tied back pile and lagging and sheet pile walls should be designed for the following lateral earth pressures from the overburden:

$$P_a = K_a (\gamma H + q)$$

Where,

P_a = Active earth pressure at the bottom of the shoring (kilopascals)
K_a = Active earth pressure coefficient (0.30)
γ = Unit weight of backfill material (20 kilonewtons per cubic metre)
H = Height of shoring above bedrock (metres)

q = Uniform surcharge at ground surface behind the wall to take into account traffic, equipment, or stockpiled soil (typically 10 kilopacals or more)

Rock anchors should be installed to provide lateral support for the shoring. The following modes of failure should be considered in the design of the anchors:

- Anchor tendon failure
- Pull out along the tendon/grout contact
- Pull out along the grout/rock contact
- Rock cone pullout

The rock anchor bond length should be calculated using a cement grout to rock bond value of 500 kilopascals. This assumes that 30 megapascal grout is used and that the sides of the drill holes are adequately cleaned in advance of the grouting. Typically, the diameter of the anchor hole should be about 1.7 to 2.5 times the diameter of the anchor. A minimum anchor length of 3 metres into the bedrock should used.

The pull out capacity of the cone of rock depends on the location, spacing, length and the inclination of the anchors, and should take into account jointing and fractures in the bedrock. Details on rock cone pull out could be provided upon request.

The design capacity of the anchors should be confirmed by carrying out proof tests on selected anchors.

If the shoring walls are to be left exposed during the winter period, the exposed face of the shoring should be thermally protected to prevent freezing of the soil behind the shoring. This will avoid possible lateral movement of the shoring and overstressing of the anchors caused by frost action.

Allowance should be made for settlement monitoring behind the shoring walls during the construction.

5.2.2 Bedrock Excavation

Based on the results of the subsurface investigation, and assuming the proposed residential building will be founded at about elevation 60.0 metres, geodetic datum, bedrock excavation is required within the southeast portion of the site (i.e., in the area of boreholes 10-1 and 10-5).

Bedrock removal at this site could be carried out using drill and blasting, hoe ramming techniques in conjunction with line drilling on close centres, or a combination of both. In areas where an upper layer fractured bedrock is encountered, rock removal could likely be carried out using hydraulic excavation equipment.

Any blasting should be carried out under the supervision of a blasting specialist engineer. As a guideline for blasting, the following peak vibration limits are suggested at the nearest structure or service:

Frequency of Virbration (Hz)	Vibration Limits (millimetres/second)
<10	5
10 to 40	5 to 50 (interpolated)
>40	50

It is pointed out that these criteria, although conservative, were established to prevent damage to existing buildings and services; more stringent criteria may be required to prevent damage to freshly placed (uncured) concrete or vibration sensitive equipment or utilities. Monitoring of the blasting should be carried out to ensure that the blasting meets the limiting vibration criteria. Pre-construction condition surveys of nearby structures and existing buried services are considered essential.

The bedrock contains near vertical joints. To reduce, not prevent, over break and under break of bedrock in the excavation, line drilling on close centres is suggested. In areas where the excavation is carried out in within about 5 metres of the existing structures or vibration sensitive service, bedrock blasting should not permitted. In these areas, trimming to the design limits should be carried out using hoe ramming techniques, as discussed below.

Bedrock removal could be carried out using large hydraulic excavation equipment in combination with hoe ramming. In order to reduce over break and/or under break of the bedrock in areas where the excavation will be carried out next to an existing site service and along the perimeter of the excavation next to existing structures, it is suggested that the limit of excavation be defined by line drilling on close centers. 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. 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.

Provided that good bedrock excavation techniques are used, the bedrock could be excavated using vertical side walls. Allowance should be made for rock reinforcement (rock bolts and mesh) in areas where blast induced damage to the side walls occurs or where fractured bedrock is encountered.

It is noted that the bedrock contains near vertical joints and is medium to thickly bedded. Therefore, some vertical and horizontal over break of the bedrock should be expected. The bedrock below founding level will likely break at a horizontal bedding plane below the design depth of the footings, which may necessitate thickening and/or lowering of the footing.

5.2.3 Groundwater Pumping

The overburden deposits below the groundwater level consist of glacial till. One rising head test carried out in the glacial till gave a hydraulic conductivity of about 1×10^{-5} centimetres per second. It is anticipated that the groundwater inflow from the overburden will be relatively small and handled by pumping from within the excavation.

The rising head tests carried out in boreholes 10-4 and 10-7 showed hydraulic conductivities of 1×10^{-6} to 1×10^{-8} centimetres per second which are relatively low. It is noted that these results represent the conditions at the specific test locations only. Based on the groundwater level measurements to date, it is anticipated that the groundwater inflow from the bedrock into the excavation should be handled by pumping from within the excavation. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

Based on the groundwater measurements to date (i.e., the measured groundwater level is below founding level), and assuming there is no increase in the groundwater level or change in excavation depth, the rate of groundwater inflow into the excavation would likely not exceed 50,000 litres per day. As such, a Permit to Take Water (PTTW) is considered necessary for this project.

5.2.4 Excavation Next to Existing Structures

Based on the results of the current investigation in conjunction with the results of the previous investigation carried out by Morey Houle Chevrier Engineering Ltd., the existing structure at 491 Richmond Road is founded on bedrock. The foundation conditions for the building at 495 Richmond Road are not currently available, but it is likely that the structure is founded on bedrock.

As discussed in Section 5.2.2 Bedrock Excavation, bedrock excavation within about 5 metres of existing structures should be carried out by line drilling on close centres in conjunction with hoe ramming techniques. It is suggested that allowance be made for line drilling 75 to 100 millimetre diameter holes on 200 to 250 millimetre centres.

For adjacent existing structures founded on bedrock, the excavation for the proposed building should not encroach within a line extending downwards and outwards from the existing foundation at an inclination of 2 vertical to 1 horizontal. If this is not possible, a detailed examination of the bedrock by geotechnical personnel will be required at the time of construction. Allowance should be made for rock reinforcement using near vertical dowels, rock bolts, and/or mesh reinforcement.

It is recommended that the foundation conditions for the building at 495 Richmond Road be obtained to identify the excavation requirements next to the building.

5.3 Foundation Design

5.3.1 Foundation Alternatives

Based on the results of the subsurface investigation, we have considered the following possible foundation alternatives for the proposed building:

Alternative 1: Conventional spread footings bearing on or within sound bedrock; and/or

Alternative 2: Socketed piers into sound bedrock

Given the proposed height of the structure, it is anticipated that the foundation loads will be relatively high. Furthermore, the glacial till deposit encountered in the boreholes has a limited capacity to support loads from footings. As such, it is not considered practicable to found the structure on spread footing bearing on or within the glacial till.

Below the southeast portion of the proposed structure (i.e., boreholes 10-1, 10-2, and 10-5), the bedrock surface was encountered between elevations 60.3 and 61.3 metres, geodetic datum, which is at or slightly above founding level. As such, in this area, the proposed building could be founded on spread footings bearing on or within competent interbedded limestone, dolostone and shale bedrock.

Below the remaining areas of the building footprint, the bedrock surface was encountered between elevations 57.3 and 59.0 metres, geodetic datum (i.e., about 1 to 5 metres below founding level). In areas where the bedrock surface is located below proposed founding level, consideration could be given to supporting the proposed building on socketed pier (caissons) foundations into the bedrock. Alternatively, the overburden could be subexcavated and the building founded on spread footings bearing on or within bedrock, or on a pad of full strength concrete above interbedded limestone, dolostone and shale bedrock. Consideration could also be given to including an additional basement level (as opposed to subexcavating between 1 and 5 metres of overburden and raising the grade below the proposed building).

The type of foundation selected on this project should be based on the relative cost of these alternatives, availability of equipment, and schedule. The foundation alternatives are discussed below.

Alternative 1: Conventional Spread Footings

Based on the results of the subsurface investigation, the proposed structure could be founded on spread footings bearing on or within competent interbedded limestone, dolostone and shale bedrock. In some areas, the underside of footing level may be above the surface of the bedrock. For this case, consideration could be given to thickening and/or lowering of the footing or, if necessary, raising the grade below the foundations with full strength concrete to the underside of footing level.

Spread footing foundations bearing on or within competent interbedded limestone, dolostone and shale bedrock, or on a pad of full strength concrete above competent interbedded limestone, dolostone and shale bedrock, could be sized using a factored bearing resistance at Ultimate Limit States of 3000 kilopascals.

The above bearing pressure assumes that all soil and any weathered or fractured bedrock is removed from the bearing surface, and that no significant soil filled seams exist in proximity to the bearing surface. It should be noted that a soil filled seam was encountered in borehole 10-2 at elevation 48.3 metres, geodetic datum. Therefore, it is suggested that 50 millimetre diameter percussion drilled probe holes be advanced at selected locations to about 1.5 metre below the proposed founding level to check for the presence of defects in the bedrock. If a thick soil filled seam or zone of fractured bedrock is encountered near founding level, it may be necessary to subexcavate the bedrock to the level of the seam and backfill the area with full strength concrete, or reduce the bearing pressure. The probe holes should be checked by qualified geotechnical personnel.

Provided that no significant fractured bedrock or soil filled seams exist below founding level, the post construction settlement of the spread footings should be less than 10 millimetres.

Alternative 2: Socketed Piers

As an alternative to spread footings, the structure could be founded on reinforced concrete caissons constructed through the overburden and socketed into the bedrock. The caissons could be constructed using churn drilling or augering techniques in conjunction with a temporary steel casing sized to allow cleaning of the rock and inspection of the bearing surface under dry conditions.

Socketed piers that derive support in shear within the bedrock should have a socket length to diameter ratio of at least 2. Assuming that the sockets are constructed using churn drilling or augering techniques and that 30 megapascal strength concrete is used in the piers, the geotechnical shear reaction at SLS could be taken as 500 kilopascals, and the factored geotechnical shear resistance at ULS could be taken as 700 kilopascals.

The settlement of the socketed piers should be small (less than 10 millimetres), provided that the bottom and sides of the piers are cleaned of all soil and disturbed bedrock and that no significant discontinuities exist within the bedrock socket. Groundwater pumping should be carried out in the sockets to allow visual downhole inspection of the bedrock.

Cobbles and boulders should be anticipated in the glacial till. As such, allowance should be made to break boulders, where necessary, within a temporary steel casing using churn drilling techniques or to remove any boulders encountered by the caissons using conventional excavation techniques. Any voids created during removal of boulders should be filled with concrete or suitable compacted soil.

5.3.2 Shear Resistance of the Footings

The resistance of the footings against lateral sliding could be calculated using a ULS factored friction value of 0.3 across the interface between the footing and the bedrock surface. The shear resistance could be increased by providing shear keys, socketed piers, and/or post tensioned rock anchors in the footings. Further information on the design of shear keys, sockets piers, and/or rock anchors could be provided upon request.

5.3.3 Uplift Resistance of the Footings

One method of increasing the uplift resistance of the foundations would be by means of rock anchors, either vertical or inclined.

The following modes of failure should be considered in the design of the anchors:

- Anchor tendon failure
- Pull out along the tendon/grout contact
- Pull out along the grout/rock contact
- Rock cone pullout
- Corrosion of the tendon

Anchor tendon failure, pull out along the tendon/grout contact should be addressed by a structural engineer. Anchor corrosion could be mitigated by using double corrosion protected rock anchors.

The rock anchor bond length should be calculated using a cement grout to rock bond value of 500 kilopascals at SLS and 1000 kilopascals at ULS. This assumes that 30 megapascal grout is used and that the sides of the drill holes are adequately cleaned in advance of the grouting. Typically, the diameter of the anchor hole should be about 1.7 to 2.5 times the diameter of the anchor. A minimum anchor length of 3 metres into the bedrock should used.

The pull out capacity of the cone of rock depends on the location, spacing, length and the inclination of the anchor, the unbonded length of the anchor, jointing and fractures in the bedrock, and the buoyant weight of the bedrock. Details on rock cone pull out could be provided as the design progresses.

If grout loss is encountered during the anchor installation, the anchor tendon should be removed, the grout allowed to harden, and the anchor redrilled. All grouting should be carried out using pumped tremie techniques.

The installation and testing of the anchors should be supervised by geotechnical personnel. The design capacity of the anchors should be confirmed by carrying out proof tests on all of the anchors. Further details on the test procedures can be provided, if required.

If socketed caisson foundations are used, the frictional and adhesive strength of the rock-concrete bond along the length of the socket could be used to increase the uplift resistance of the foundations. Details on the shear resistance along the socket could be provided as the design progresses.

5.3.4 Frost Protection of Foundations

At least 1.5 metres of earth cover should be provided for frost protection purposes for all exterior footings and grade beams for frost protection purposes. Isolated, exterior footings or grade beams 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 for frost protection purposes. Where less than the required depth of soil cover can be provided, the grade beams and footings can be protected from frost by using a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

5.3.5 Seismic Design of Proposed Structure

Based on the results of the bedrock coring, the underside of the proposed building will be within interbedded limestone, dolostone and shale bedrock (Alternative 1 in Section 5.3.1) or on a combination of glacial till and bedrock (Alternative 2 in Section 5.3.1). In accordance with the Ontario Building Code, Site Class C could be used for the seismic design of the proposed residential building.

Consideration could be given to carrying out shear wave velocity testing to evaluate whether a more favourable Site Class (i.e., A or B) can be specified. Further details regarding shear wave velocity testing and the effects of the foundation alternatives on the Site Class could be provided upon request.

5.3.6 Parking Garage Foundation Wall Backfill and Drainage

5.3.6.1 Foundation Wall Alternatives

The following alternatives could be considered for the basement foundation walls for the structure:

1) Foundation walls formed on both sides, damp proofed and backfilled with free draining-non frost susceptible granular materials; OR

2) Foundation walls formed on one side, with a proprietary drainage system placed directly against the bedrock.

Alternative 2 could be considered where space constraints dictate and/or to limit the amount of bedrock excavation.

5.3.6.2 Foundation Drainage

In areas where the foundation walls are formed on both sides, the exterior of the foundation walls should be damp proofed and a perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls below the parking garage floor slab. The drain should outlet by gravity to 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 building), a nonwoven geotextile should be placed between the clear stone and any sand backfill material.

In areas where the foundation walls will be cast directly on the bedrock, drainage could be achieved by means of a prefabricated drainage system, such as Miradrain 5000, fastened to the bedrock. In this case, the drainage system could be connected to a perforated drain below the interior basement floor slab through regularly spaced weep holes in the foundation walls or footings.

As described in Section 5.2.1 Overburden Excavation, a setback allowance of at least 1 metre from the edge of the bedrock excavation is required for a shoring system socketed into the bedrock. As such, in areas where the foundation walls extend above the bedrock surface, the foundation walls could be formed on both sides. In this case, the foundation walls should be damp proofed and drainage could be achieved by means of a prefabricated drainage system, such as Miradrain 5000, fastened to the exterior of the foundation wall and hydraulically linked (spliced) to the drainage system fastened to the bedrock or by means of a free draining granular backfill material.

In areas where the foundation walls will be cast directly on the bedrock and will extend above the bedrock surface, a perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls at the bedrock surface. 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 building), a nonwoven geotextile should be placed between the clear stone and any sand backfill material.

5.3.6.3 Backfill Type

In areas where the foundation walls are formed on both side, the exterior of the foundation walls should be backfilled with non-frost susceptible, sand or sand and gravel conforming to Ontario Provincial Standard Specifications (OPSS) for Granular B Type I or 19 millimetre clear crushed stone. The use of 19 millimetre clear crushed stone is preferred where compaction is not practicable due to the proximity of the foundation wall to the bedrock or shoring system. Any clear stone should be suitably wrapped with a nonwoven geotextile where it is direct contact with other soils.

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. Compaction of the 19 millimetre clear crushed stone backfill is not considered essential.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed building, 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 and native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.8 metres below finished grade to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

5.3.6.4 Earth Pressures on Foundation Walls

Foundation walls that are backfilled with sand or sand and gravel or 19 millimetre clear crushed stone should be designed to resist “at rest” earth pressures calculated using the following formula:

$$P_o = K_o\,(\gamma\,H + q)$$

Where,

P_o = At rest earth pressure at the bottom of the foundation wall (kilopascals)

K_o = At rest earth pressure coefficient (0.50)

γ = Unit weight of backfill material (22 kilonewtons per cubic metre)

H = Height of foundation wall (metres)

q = Uniform surcharge at ground surface behind the wall to take into account traffic, equipment, or stockpiled soil (typically 10 kilopacals)

Where conditions dictate, allowance should be made in the structural design of the foundation walls for active loads due to ground supported vehicles/equipment. For example, the horizontal active load due to a uniform, vertical live load adjacent to the foundation wall could be determined using a horizontal earth pressure coefficient, K_o, of 0.50, times the vertical live load. The effects of other vertical loads (point loads, line loads, etc.) adjacent to or near the foundation walls could be provided, if required.

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

Seismic shaking can increase the forces on the foundation walls backfilled with granular material during or following an earthquake. The increase in pressure during seismic shaking may be estimated using the method suggested by Wood (1973) for non-yielding smooth walls which are restrained against movement. The combined coefficient of static and seismic earth pressure on the back of the foundation walls can be calculated as 0.70. The static thrust component acts at H/3 and the dynamic thrust component acts at approximately 0.63H above the base of the foundation wall (where H is the height of the foundation wall).

In areas where the foundation walls are formed directly on the surface of the bedrock, the lateral earth pressure from the bedrock will be negligible.

5.3.7 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 200 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 parking garage floor slab. If well graded granular material (such as OPSS Granular B Type II) is used below the parking garage 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 parking garage. If clear crushed stone is used below the parking garage floor slab, drains are not considered essential provided that the clear stone can outlet to the sump and drains are installed to link any hydraulically isolated areas in the basement. The drains should outlet by gravity to a sump from which the water is pumped.

The floor slab should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimized shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for any slab on grade 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.

5.4 Proposed Services

5.4.1 Excavation for the Site Services

The excavation for the sewer and watermain services will be carried out through asphaltic concrete, granular material, fill material, silty clay, glacial till and possibly interbedded limestone, dolostone, and shale bedrock.

In overburden, the excavation for service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 Soil. In bedrock, the excavation for the service pipes should be in accordance with OPSD 802.013 for Type 3 soil.

The excavations for the services should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. That is, open cut excavations within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Alternatively, the excavations could be carried out near vertically within a tightly fitting, braced steel trench box designed specifically for this purpose.

Groundwater inflow into the excavations for the proposed services should be handled by pumping from within the excavations. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services. It is noted that the existing sewers and watermains likely have a bedding and surround composed of granular material and that water inflow into the trenches through the bedding and surround could be significant.

5.4.2 Pipe Bedding

The bedding for service pipes located within overburden should be in accordance with OPSD 802.010 for Type 3 Soil. The bedding for service pipes located within bedrock should be in accordance with OPSD 802.013 for Type 3 soil. The pipe bedding material should consist of at least 150 millimetres of granular material meeting Ontario Provincial Standard Specification

(OPSS) for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.

In areas where the subsoil is disturbed or where unsuitable material (such as fill, organic soil, or existing trench backfill material) exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as native sand or material that meets OPSS Granular A or Granular B Type II (50 or 100 millimetre minus crushed stone). To provide adequate support for the pipes in the long term in areas where subexcavation of material is required below design subgrade level, the excavations should be sized to allow a 1 horizontal to 2 vertical spread of granular material down and out from the bottom of the pipes. The use of clear crushed stone as a bedding or subbedding material should not be permitted.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The granular bedding and subbedding materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value.

5.4.3 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 or imported granular material conforming to OPSS Granular B Type I. Well shattered and graded rock fill is acceptable as backfill for the lower portion of the service trenches in areas where the excavation is in bedrock.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Any topsoil or organic soil should be wasted from the trench. If rock fill 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 any rock fill should be blinded with well graded crushed stone.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced to 90 percent of the standard Proctor dry density in areas where the trench backfill is not located below or in close proximity to existing or future roadways, parking areas, sidewalks, etc. and provided that some settlement above the trench is acceptable.

The silty clay and glacial till will likely have moisture contents above optimum for compaction. Furthermore, depending on the weather conditions at the time of construction, some wetting of materials could occur. As such, the specified densities may not be possible to achieve and, as a consequence, some settlement of these backfill materials should be expected. 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 placement of the final lift of the asphaltic concrete for 3 months, or longer, to allow some of the trench backfill settlement to occur and thereby improve the final pavement appearance.

5.5 Access Roadways and Pavement Reinstatement

5.5.1 Subgrade Preparation

In preparation for the construction of the access roadways, any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. The subgrade surface for the pavement areas should be proof rolled with a large (10 tonne minimum) steel drum roller under dry conditions. Any soft areas exposed from the proof rolling should be subexcavated and replaced with imported granular material conforming to OPSS Granular B Type I or II.

The grade within the access roadway areas could be raised, where necessary, using suitable imported granular material conforming to OPSS Granular B Type I. The imported granular material should be placed in maximum 300 millimetre thick lifts and compacted to at least 98 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

5.5.2 Pavement Design

It is suggested that the following minimum pavement structure be used for the access roadways and to reinstate the existing access roadways used by truck traffic and fire trucks:

40 millimetres of HL3 or Superpave 12.5 asphaltic concrete, over
50 millimetres of HL8 or Superpave 19.0 asphaltic concrete, over
150 millimetres of OPSS Granular A base, over
450 millimetres of OPSS Granular B Type II subbase, over
Approved subgrade

In areas where the pavement structure is constructed over the existing base/subbase having a thickness of at least 450 millimetres, the pavement structure could consist of:

40 millimetres of HL3 or Superpave 12.5 asphaltic concrete, over
50 millimetres of HL8 or Superpave 19.0 asphaltic concrete, over
150 millimetres of OPSS Granular A base, over
Existing approved sand and gravel base/subbase

The Superpave asphaltic concrete mixes should be designed for Traffic Level A or B. Performance grade PG 58-34 asphaltic concrete should be specified.

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.

The design life of the pavement should be 20 to 25 years. Allowance should be made for normal crack sealing, as required, and a possible asphaltic concrete overlay in about 12 to 15 years.

5.5.3 Pavement Transitions

In areas where the new pavements will abut existing pavements, the existing asphaltic concrete should be neatly saw cut to facilitate reinstatement. To provide a uniform longitudinal joint overlap between the existing and the proposed pavement during the placement of the asphaltic concrete, the saw cut should be relatively straight.

Where the thickness of the new asphaltic concrete will be greater than the existing asphaltic concrete, the new asphaltic concrete should taper to match the existing asphaltic concrete within 300 millimetres of the limit of asphalt removal. In addition, the joint between the new and existing granular materials could be located slightly beyond the joint in the asphaltic concrete. Furthermore, a stepped or sloped joint (say at 1 horizontal to 1 vertical, or flatter) could be considered to provide a gradual transition and facilitate compaction.

5.5.4 Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. The catch basins should be provided with minimum 3 metre long perforated stub drains which extend in at least two directions from each catch basin at pavement subgrade level. The need for additional subdrains within the granular material should be assessed by geotechnical personnel as part of the design.

5.5.5 Effects of Soil Disturbance and Construction Traffic on the Pavement Design

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 to the subgrade material. The contractor should be responsible for construction access.

The above pavement structures assume that the roadway subgrade surface and trench backfill is prepared as described in this report. If the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation, the Granular B Type II thickness given above may not be adequate and it may be necessary to:

- Increase the thickness of the Granular B Type II subbase,
- Incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or.
- A combination of the above.

The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

5.6 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in samples of the groundwater ranges between 116 and 569 milligrams per litre. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the groundwater can be classified as low in the overburden to moderate in the bedrock. Any concrete that will be in contact with the native soil, bedrock or groundwater should meet CSA A23.1 Class S-3 requirements. This would include the rock anchors, footings, foundation walls and columns below the lowest basement floor slab level, and the parking garage floor slab on grade. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) near the building should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the conductivity and pH of the groundwater, the groundwater can be classified as aggressive to very aggressive toward unprotected steel. It is noted that the corrosivity of the groundwater could vary throughout the year due to the application sodium chloride for de-icing.

5.7 Winter Construction

In the event that construction is required during freezing temperatures, the bedrock below the footings 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 behind shoring walls or below the level of any existing structures or services. Freezing of the soil or bedrock could result in damage to the shoring wall and its tie backs, and heaving related damage to structures or services.

5.8 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, blasting, 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. Assuming that any blasting is carried out in accordance with the guidelines in this report, the magnitude of the vibrations should be less than that required to cause damage to the nearby structures or services that are in good condition, but may be felt at the nearby structures. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during the construction.

5.9 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 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 foundation drawings for 495 Richmond Road and the existing parking garage should be reviewed by us.

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 the proposed building, site services, and access 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. The bedrock support requirements should be assessed by geotechnical personnel during the foundation construction.

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.

Yours truly,

HOULE CHEVRIER ENGINEERING LTD.

Johnathan A. Cholewa, Ph.D., EIT

Andrew Chevrier, M.Eng.P.Eng.
Principal

LICENSED PROFESSIONAL ENGINEER
26 July 2012
A.F. CHEVRIER
797450
PROVINCE OF ONTARIO

References

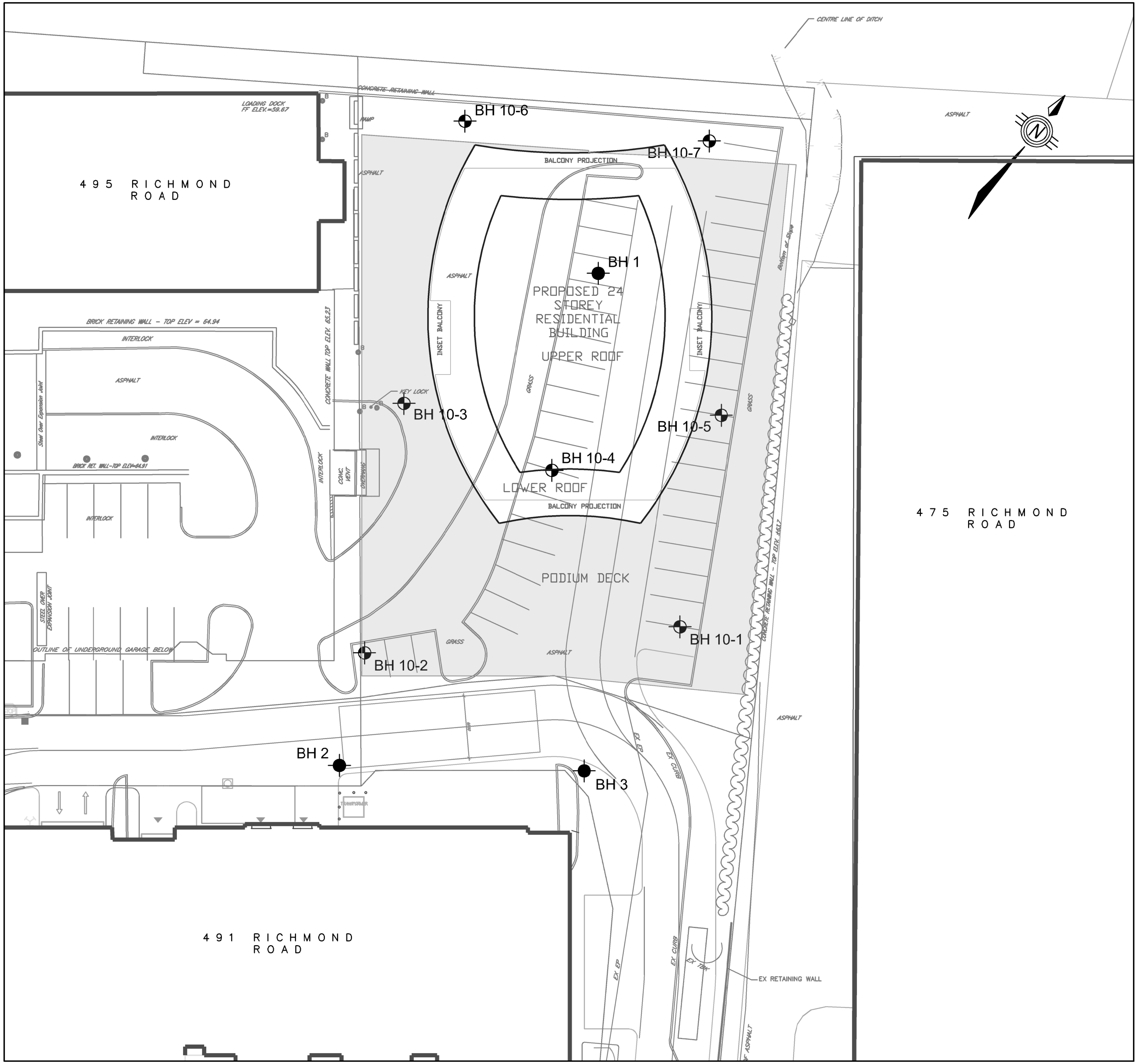
Wood, J. (1973). “Earthquake-induced soil pressure on structures,” Report EERL 73-05, California Institute of Technology, Pasadena, California, 311 pp.

KEY PLAN

FIGURE 1

N.T.S

Date: July 2012
Project: 10-513



LEGEND

⊕ BH 10-1	APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.
● BH 1	APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY MOREY HOULE CHEVRIER ENGINEERING LTD.

Client CANDEREL MANAGEMENT INC.		Location 485 RICHMOND ROAD OTTAWA, ON	Revision 0
Drawn by D.J.R	Approved by A.F.C	Project No. 10-513	Scale 1:1250

Houle Chevrier Engineering

Title
BOREHOLE LOCATION PLAN

Date
July 2012

FIGURE 2

Photograph of Bedrock Core

FIGURE 3

Date: July 2012

Project: 10-513

Borehole: 10-2

Depth: 2.95 to 15.16 metres

Houle Chevrier
Engineering

Photograph of Bedrock Core	FIGURE 4

Date: July 2012

Project: 10-513

Borehole: 10-4

Depth: 3.07 to 15.39 metres

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Engineering

Photograph of Bedrock Core

FIGURE 5

Date: July 2012

Project: 10-513

Borehole: 10-6

Depth: 4.95 to 15.32 metres

Houle Chevrier Engineering

Photograph of Bedrock Core	FIGURE 6

Date: July 2012

Project: 10-513

Borehole: 10-7

Depth: 4.29 to 15.32 metres

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Engineering

APPENDIX A

ABBREVIATIONS AND SYMBOLS,
BEDROCK TERMINOLOGY AND DESCRIPTIONS
RECORD OF BOREHOLE SHEETS
AND FIGURE A1

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS	auger sample
CS	chunk 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 millimetres 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: 10-513

RECORD OF BOREHOLE 10-1

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 29, 2010

SPT HAMMER: 63.5 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 - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) WATER CONTENT, PERCENT Wp ⊢—o W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		62.91							
		Asphaltic concrete		0.03							
		Dense, grey crushed sand and gravel (BASE/SUBBASE)		61.69 1.22	1	50 D.O.	44				
1		Dark brown to grey brown sandy silt, some gravel, trace clay (FILL MATERIAL)		61.33 1.58	2	50 D.O.	50 for 50 mm				
		End of borehole Auger refusal									Borehole dry upon completion
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: AC

PROJECT: 10-513

RECORD OF BOREHOLE 10-2

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 26, 2010

SPT HAMMER: 63.5 kg; drop 0.76 m

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○: 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}

WATER CONTENT, PERCENT: Wp ⊢—○ W —⊣ Wl: 20, 40, 60, 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface	63.24						
		Asphaltic concrete	0.05						
		Dense, grey crushed sand and gravel (BASE/SUBBASE)							
1		Compact, brown sand and gravel (FILL MATERIAL)	62.23 1.02	1	50 D.O.	53			
2		Grey brown silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	61.41 1.83	2	50 D.O.	37			
				3	50 D.O.	50 for 100 mm			
3	Rotary Drill NQ RC	Fractured, fresh, dark grey to grey interbedded LIMESTONE and SHALE	60.29 2.95	4	RC	TCR	= 96%, SCR = 39%, RQD = 16%		
4		Fresh, medium to thickly bedded, grey, greenish grey and dark grey interbedded LIMESTONE, DOLOSTONE and SHALE with occasional near vertical joints Note: 15 mm thick mud seam at 14.94 metres (elevation 48.3 metres)	59.05 4.19	5	RC	TCR	= 100%, SCR = 85%, RQD = 81%		
6				6	RC	TCR	= 97%, SCR = 90%, RQD = 79%		
8				7	RC	TCR	= 100%, SCR = 98%, RQD = 88%		
9				8	RC	TCR	= 100%, SCR = 100%, RQD = 92%		
10								U	
11				9	RC	TCR	= 100%, SCR = 93%, RQD = 92%	U	
12				10	RC	TCR	= 98%, SCR = 98%, RQD = 98%		
14				11	RC	TCR	= 100%, SCR = 100%, RQD = 100%		
15				12	RC	TCR	= 100%, SCR = 80%, RQD = 55%		
15		End of borehole	48.08 15.16						Groundwater conditions not observed

DEPTH SCALE

1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: AC

PROJECT: 10-513

RECORD OF BOREHOLE 10-3

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

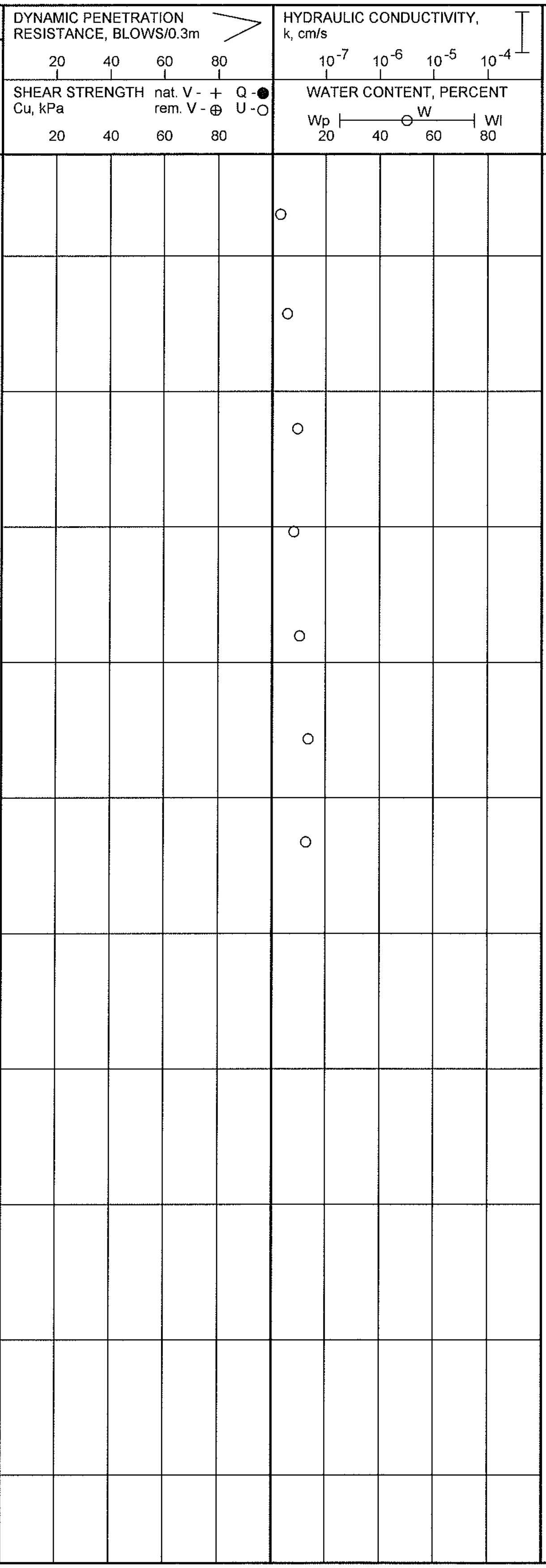
DATUM: Geodetic

BORING DATE: October 29, 2010

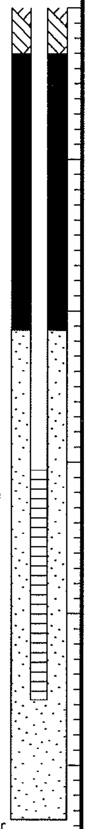
SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	61.55				
Asphaltic concrete	0.05				
Dense, grey crushed sand and gravel (BASE/SUBBASE)	61.12 0.43	1A	C.S.		
Brown sand, trace silt, trace gravel (FILL MATERIAL)	60.03 1.52	1B	C.S.		
Loose to compact, grey brown silty sand, trace to some clay, some gravel, probable cobbles and boulders (GLACIAL TILL)	58.66 2.89	2 3	50 D.O. 50 D.O.	9 15	
Very loose to compact, grey silty sand, trace to some clay, some gravel, probable cobbles and boulders (GLACIAL TILL)	56.19 5.36	4 5 6	50 D.O. 50 D.O. 50 D.O.	10 4 1	MH (See Fig. A1)
End of borehole Auger refusal					



PIEZOMETER OR STANDPIPE INSTALLATION



Cold mix with protective flushmount

Bentonite

Filter sand

1.52 metres long, 38 mm diameter slotted well screen

Groundwater level at 4.2 metres (elevation 57.3 metres, geodetic datum) below ground surface on November 15, 2010

DEPTH SCALE 1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: AC

RECORD OF BOREHOLE 10-4

PROJECT: 10-513

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: October 27, 2010

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 m

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● ; rem. V - ⊕ U - ○ : 20, 40, 60, 80

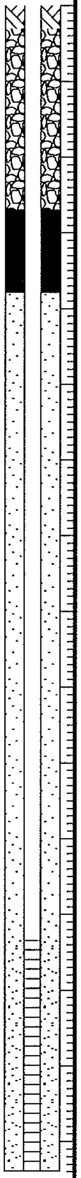
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}

WATER CONTENT, PERCENT: Wp ⊢—○W—⊣ Wl : 20, 40, 60, 80

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	Ground Surface	62.04
Power Auger 200 mm Diameter Hollow Stem	Asphaltic concrete	0.05
	Dense, grey crushed sand and gravel (BASE/SUBBASE)	60.97 1.07
	Loose to dense, grey brown silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	59.35 2.69
	Grey silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	58.97 3.07
Rotary Drill NQ RC	Fresh, thinly to medium bedded, grey and dark grey interbedded LIMESTONE and SHALE	55.01 7.03
	Fresh, medium to thickly bedded, grey and dark grey interbedded LIMESTONE, DOLOSTONE and SHALE with occasional near vertial joints	51.93 10.11
	Fresh, medium to thickly bedded, greenish grey and dark grey interbedded DOLOSTONE and SHALE	46.65 15.39
	End of borehole	

NUMBER	TYPE	BLOWS/0.3m		ADDITIONAL LAB. TESTING
1	50 D.O.	31		
2	50 D.O.	33		
3	50 D.O.	8		
4	50 D.O.	50 for 25 mm		
5	RC	TCR	= 100%, SCR = 95%, RQD = 69%	
6	RC	TCR	= 97%, SCR = 76%, RQD = 71%	
7	RC	TCR	= 100%, SCR = 100%, RQD = 91%	
8	RC	TCR	= 97%, SCR = 97%, RQD = 93%	U
9	RC	TCR	= 100%, SCR = 100%, RQD = 100%	U
10	RC	TCR	= 100%, SCR = 100%, RQD = 91%	U
11	RC	TCR	= 100%, SCR = 100%, RQD = 98%	
12	RC	TCR	= 98%, SCR = 98%, RQD = 95%	
13	RC	TCR	= 100%, SCR = 100%, RQD = 84%	

PIEZOMETER OR STANDPIPE INSTALLATION



Cold mix with protective flushmount

Native backfill

Bentonite seal

Filter sand

3.05 metres long, 38 mm diameter slotted well screen

Groundwater level at 4.8 metres (elevation 57.2 metres, geodetic datum) below ground surface on November 15, 2010

DEPTH SCALE 1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: Ac

PROJECT: 10-513

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: October 29, 2010

RECORD OF BOREHOLE 10-5

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 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 - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) / 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		61.98							
		Asphaltic concrete		0.05							
		Dense, grey crushed sand and gravel (BASE/SUBBASE)			1	C.S.					
1		End of borehole Auger refusal		61.14 0.84							Borehole dry upon completion
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: AC

PROJECT: 10-513

RECORD OF BOREHOLE 10-6

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 28, 2010

SPT HAMMER: 63.5 kg; drop 0.76 m

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

WATER CONTENT, PERCENT: Wp W Wl 20 40 60 80

SOIL PROFILE

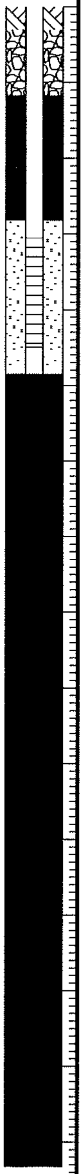
BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)
	Ground Surface	60.73
Power Auger 200 mm Diameter Hollow Stem	Asphaltic concrete	0.02
	Dense, grey crushed sand and gravel (BASE/SUBBASE)	59.66 1.07
	Loose, brown sand, some gravel, trace silt (FILL MATERIAL)	58.60 2.13
	Very loose to loose, grey silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	55.78 4.95
Rotary Drill NQ RC	Fractured, grey and dark grey interbedded LIMESTONE and SHALE	55.12 5.61
	Fresh, medium to thickly bedded, grey and dark grey interbedded LIMESTONE and SHALE with occasional near vertical joints	52.22 8.51
	Fresh, medium to thickly bedded, greenish grey and dark grey interbedded DOLOSTONE and SHALE	45.41 15.32
	End of borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	
1	50 D.O.	23	
2	50 D.O.	6	
3	50 D.O.	8	
4	50 D.O.	4	
5	50 D.O.	4	
6	50 D.O.	65 for 150 mm	
7	RC	TCR	= 69%, SCR = 46%, RQD = 0%
8	RC	TCR	= 95%, SCR = 93%, RQD = 93%
9	RC	TCR	= 100%, SCR = 98%, RQD = 92%
10	RC	TCR	= 100%, SCR = 100%, RQD = 100%
11	RC	TCR	= 97%, SCR = 97%, RQD = 97%
12	RC	TCR	= 100%, SCR = 98%, RQD = 98%
13	RC	TCR	= 100%, SCR = 100%, RQD = 100%
14	RC	TCR	= 100%, SCR = 100%, RQD = 92%

ADDITIONAL LAB. TESTING: U (approx. 10.4 m), U (approx. 11.5 m)

PIEZOMETER OR STANDPIPE INSTALLATION



Cold mix with protective flushmount

Native backfill

Bentonite seal ∇

Filter sand

1.52 metres long, 38 mm Diameter slotted well screen

Bentonite seal

Groundwater level at 2.2 metres (elevation 58.5 metres, geodetic datum) below ground surface on November 15, 2010

DEPTH SCALE 1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: J.C.

CHECKED: AC

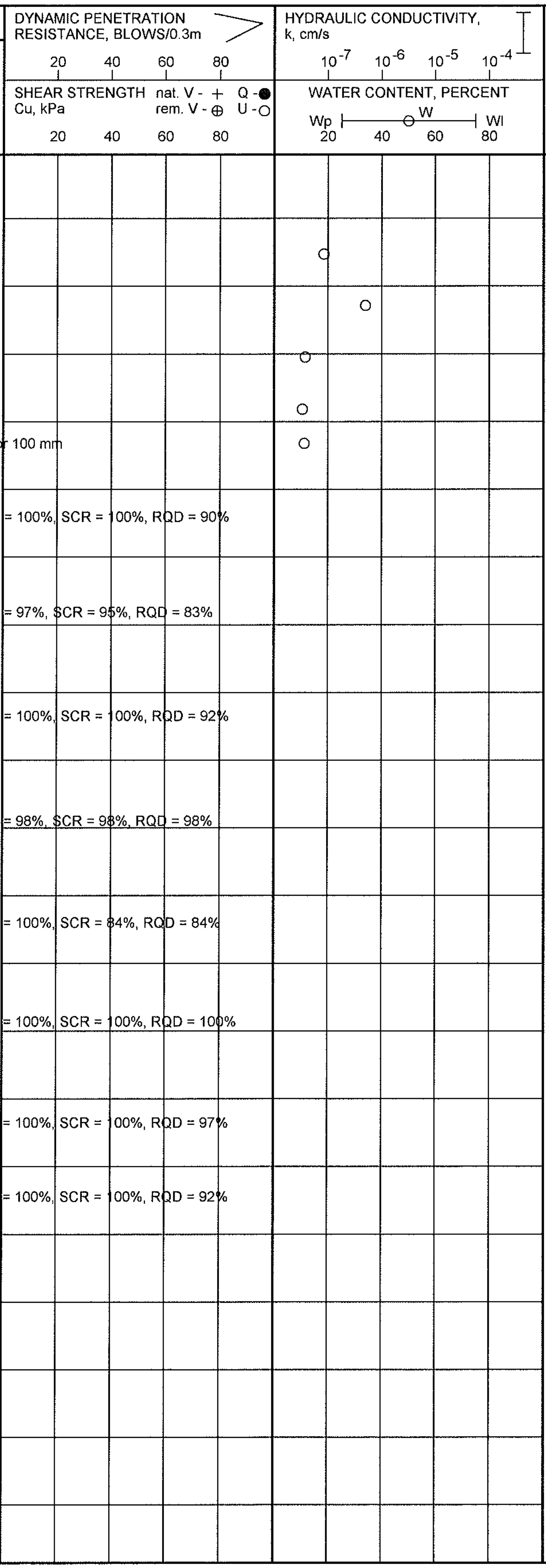
PROJECT: 10-513
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: October 27-28, 2010

RECORD OF BOREHOLE 10-7

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 m

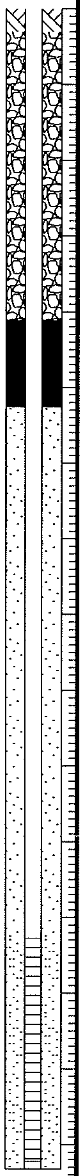
BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
	Ground Surface	61.54
Power Auger, 200 mm Diameter Hollow Stem	Asphaltic concrete	0.08
	Dense, grey crushed sand and gravel (BASE/SUBBASE)	60.47 / 1.07
	Brown to grey brown silty sand, some clay, trace to some gravel (FILL MATERIAL)	1.37
	Very stiff, grey brown SILTY CLAY (weathered crust)	59.26 / 2.28
	Compact, grey brown silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	58.65 / 2.89
	Loose, grey silty sand, some gravel, trace to some clay, probable cobbles and boulders (GLACIAL TILL)	57.25 / 4.29
Rotary Drill, NQ RC	Fresh, medium to thickly bedded, grey and dark grey interbedded LIMESTONE and SHALE	51.61 / 9.93
	Fresh, medium to thickly bedded, greenish grey and dark grey interbedded DOLOSTONE and SHALE	46.22 / 15.32
	End of borehole	

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	13
2	50 D.O.	16
3	50 D.O.	17
4	50 D.O.	8
5	50 D.O.	50 for 100 mm
6	RC	TCR = 100%, SCR = 100%, RQD = 90%
7	RC	TCR = 97%, SCR = 95%, RQD = 83%
8	RC	TCR = 100%, SCR = 100%, RQD = 92%
9	RC	TCR = 98%, SCR = 98%, RQD = 98%
10	RC	TCR = 100%, SCR = 84%, RQD = 84%
11	RC	TCR = 100%, SCR = 100%, RQD = 100%
12	RC	TCR = 100%, SCR = 100%, RQD = 97%
13	RC	TCR = 100%, SCR = 100%, RQD = 92%



ADDITIONAL LAB. TESTING: MH (See Fig. A1); U; U; U; U; U

PIEZOMETER OR STANDPIPE INSTALLATION



Cold mix with protective flushmount

Native backfill

Bentonite seal

Filter sand

2.96 metres long, 38 mm diameter slotted well screen

Groundwater level at 3.7 metres (elevation 57.8 metres, geodetic datum) below ground surface on November 15, 2010

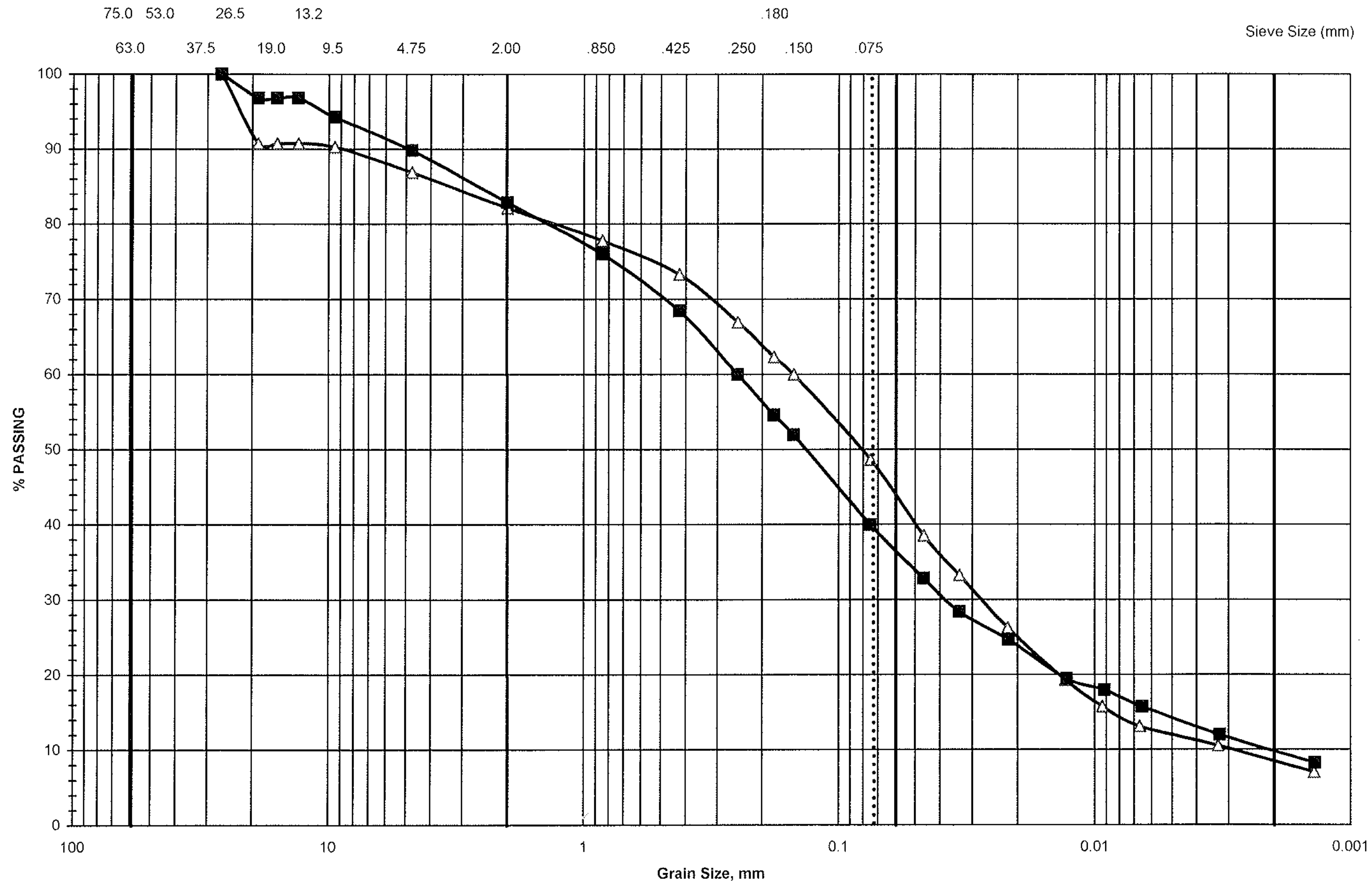
DEPTH SCALE
1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: J.C.
CHECKED: AC

GRAIN SIZE ANALYSIS

FIGURE A1



	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	CLAY
	GRAVEL			SAND			SILT			
Modified M.I.T. Classification										

Bore Hole	Sample	Depth (m)	Legend
10-3	6	4.57 - 5.18	△
10 - 7	4	4.57 - 5.18	■

Houle Chevrier Engineering

Date: November 2010
Project : 10-513

APPENDIX B

RECORD OF BOREHOLE SHEET
PREVIOUS INVESTIGATION BY
MOREY HOULE CHEVRIER ENGINEERING LTD.

PROJECT: 04-178

RECORD OF BOREHOLE 1

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: May 29, 2004

SPT HAMMER: 63.6 kg; drop 0.76 m

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^{-7} 10^{-6} 10^{-5} 10^{-4}

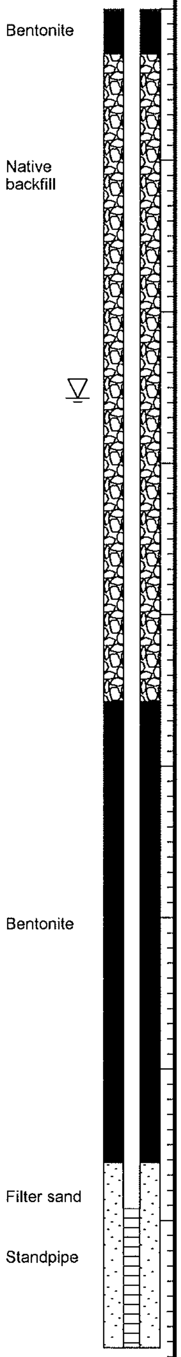
WATER CONTENT, PERCENT: Wp ├──○ W──┤ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)
	Ground Surface	61.61
Power Auger 200 mm Diameter Hollow Stem	ASPHALTIC CONCRETE	0.08
	Grey sand and gravel (BASE MATERIAL)	61.33 0.28
	Grey brown sand and gravel (SUBBASE MATERIAL)	61.15 0.46
	Grey brown silty sand, some gravel, piece of perforated drain (FILL)	60.39 1.22
	Very loose to compact grey silty sand, trace clay, some gravel (GLACIAL TILL)	56.73 4.88
Rotary Drilling NQ Core	Fresh to faintly weathered, thinly to thickly bedded, layered grey LIMESTONE with some shale partings and dark grey SHALE	52.77 8.84
	End of borehole	

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	7
2	50 DO	15
3	50 DO	5
4	50 DO	6
5	50 DO	2
6	50 DO	6
7	RC	TCR=91% SCR=35% RQD=27%
8	RC	TCR=100% SCR=90% RQD=87%
9	RC	TCR=100% SCR=98% RQD=88%

PIEZOMETER OR STANDPIPE INSTALLATION



Groundwater level in standpipe at elevation 59.05 metres on June 29, 2004.

DEPTH SCALE 1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED:

PROJECT: 04-178

RECORD OF BOREHOLE 2

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: May 29, 2004

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: 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 - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) / 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		62.85							
		ASPHALTIC CONCRETE	■	62.80 0.05							
		Grey sand and gravel (SUBBASE MATERIAL)	▨	62.57 0.28							
		Grey brown sand and gravel (SUBBASE MATERIAL)	▨	62.06 0.79							
		Practical auger refusal End of borehole									
1											No groundwater inflow observed on completion of drilling.
2											

DEPTH SCALE

1 to 10

Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED:

PROJECT: 04-178

RECORD OF BOREHOLE 3

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: May 29, 2004

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: 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 - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) 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		63.15							
		ASPHALTIC CONCRETE		63.10 0.05							
		Grey sand and gravel (SUBBASE MATERIAL)		62.87 0.28							
		Grey brown sand and gravel (SUBBASE MATERIAL)		62.18 0.97							
1		Practical auger refusal End of borehole									No groundwater inflow observed on completion of drilling.
2											

DEPTH SCALE

1 to 10

Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED:

APPENDIX C

HYDRAULIC CONDUCTIVITY TESTING
RESULTS AND ANALYSIS
BOREHOLE 10-4
BOREHOLE 10-6
BOREHOLE 10-7

Houle Chevrier
Engineering

Pumping Test - Water Level Data

Page 1 of 1

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road

Pumping Test: BH10-4 Rising Head Test

Pumping Well: BH10-4

Test Conducted by: BP

Test Date: 11/1/2010

Discharge: variable, average rate 5.9889E-5 [m³/min]

Observation Well: BH10-4

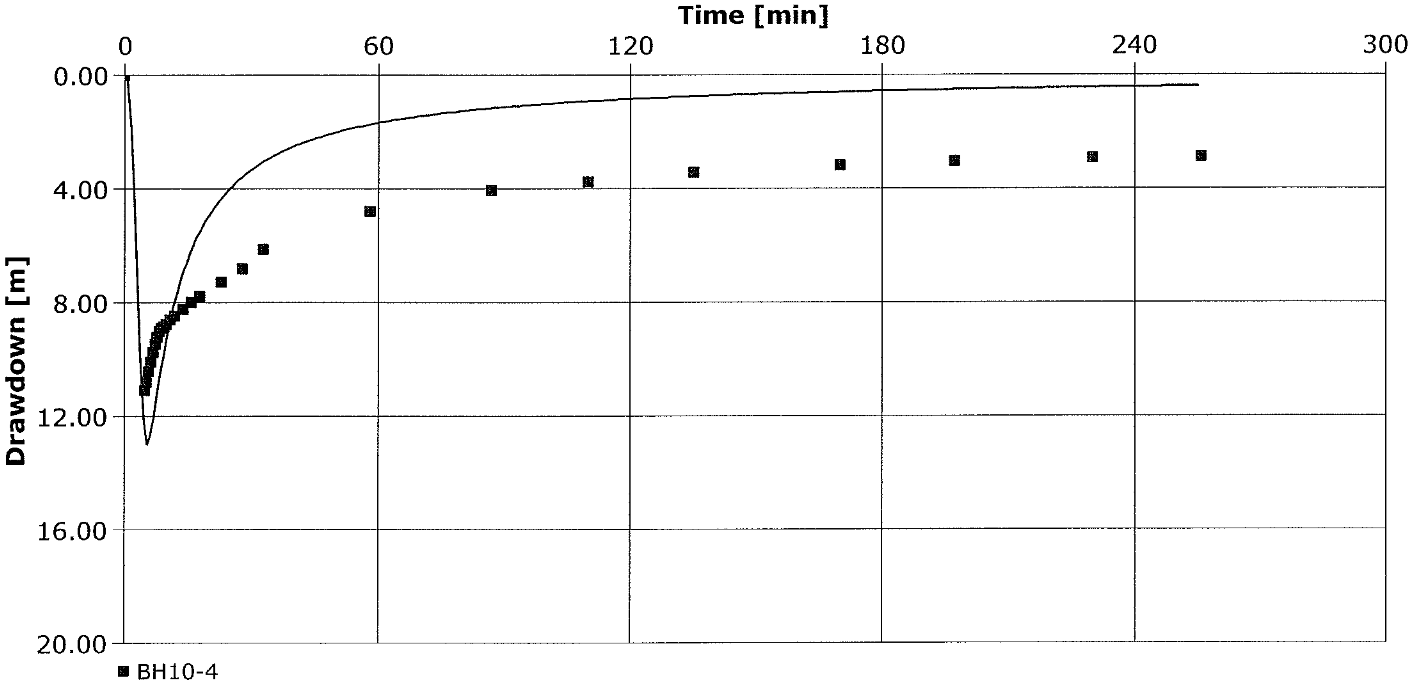
Static Water Level [m]: 2.90

Radial Distance to PW [m]: -

	Time [min]	Water Level [m]	Drawdown [m]
1	0	2.90	0.00
2	4.75	13.98	11.08
3	5.25	13.69	10.79
4	5.75	13.32	10.42
5	6.25	13.01	10.11
6	6.75	12.67	9.77
7	7.25	12.36	9.46
8	7.75	12.13	9.23
9	8.25	11.91	9.01
10	8.75	11.81	8.91
11	9.25	11.74	8.84
12	9.75	11.66	8.76
13	10.75	11.51	8.61
14	11.75	11.37	8.47
15	13.75	11.13	8.23
16	15.75	10.91	8.01
17	17.75	10.69	7.79
18	22.75	10.18	7.28
19	27.75	9.73	6.83
20	32.75	9.04	6.14
21	58	7.69	4.79
22	87	6.94	4.04
23	110	6.65	3.75
24	135	6.34	3.44
25	170	6.07	3.17
26	197	5.94	3.04
27	230	5.84	2.94
28	256	5.78	2.88

Pumping Test Analysis Report
Project: Office Building Geotechnical Investigation
Number: 10-513
Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-4 Rising Head Test	Pumping Well: BH10-4
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by: JM	Theis Analysis	Analysis Date: 11/18/2010
Aquifer Thickness: 12.32 m	Discharge: variable, average rate 5.9889E-5 [m³/min]	



Calculation after Theis

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Storage coefficient	Radial Distance to PW [m]	
BH10-4	2.42×10^{-2}	1.96×10^{-3}		0.02	

Houle Chevrier Engineering

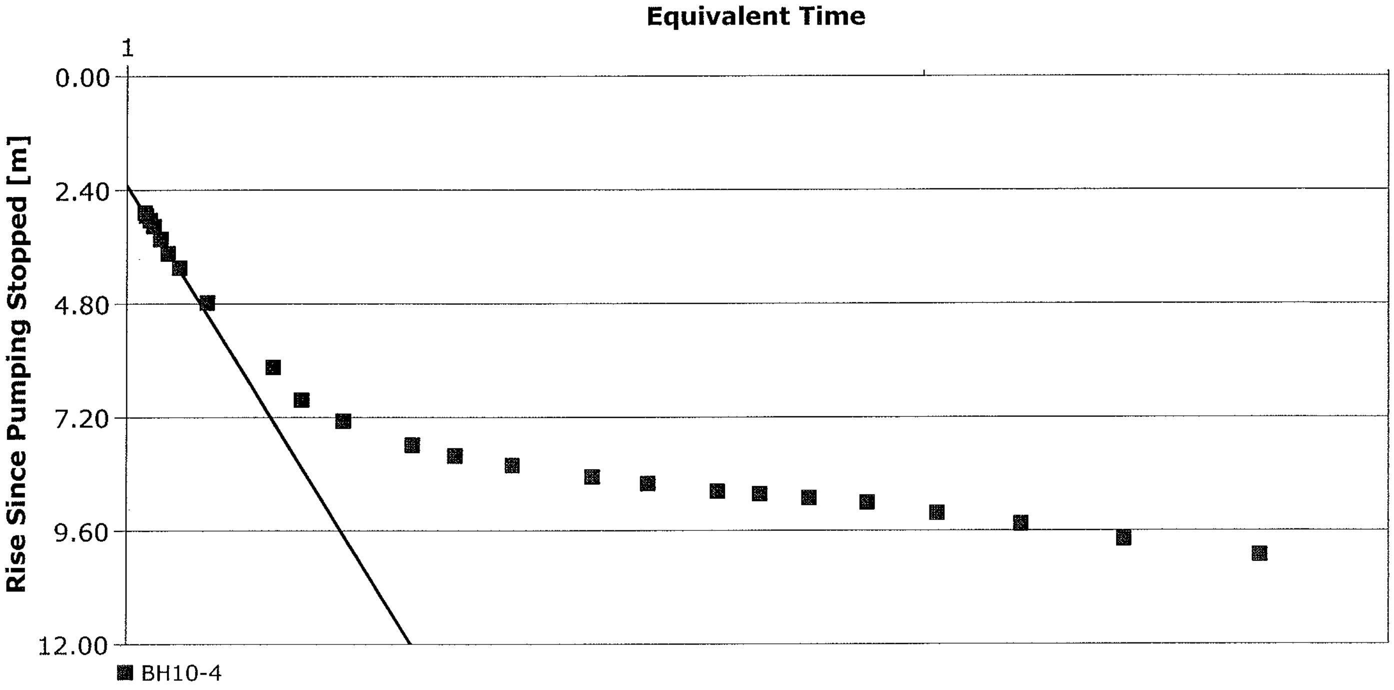
Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-4 Rising Head Test	Pumping Well: BH10-4
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by: JM	Theis Analysis Recovery Period	Analysis Date: 11/18/2010
Aquifer Thickness: 12.32 m	Discharge: variable, average rate 5.9889E-5 [m³/min]	



Calculation after Theis & Jacob

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-4	1.75×10^{-4}	1.42×10^{-5}	0.02	

Houle Chevrier Engineering

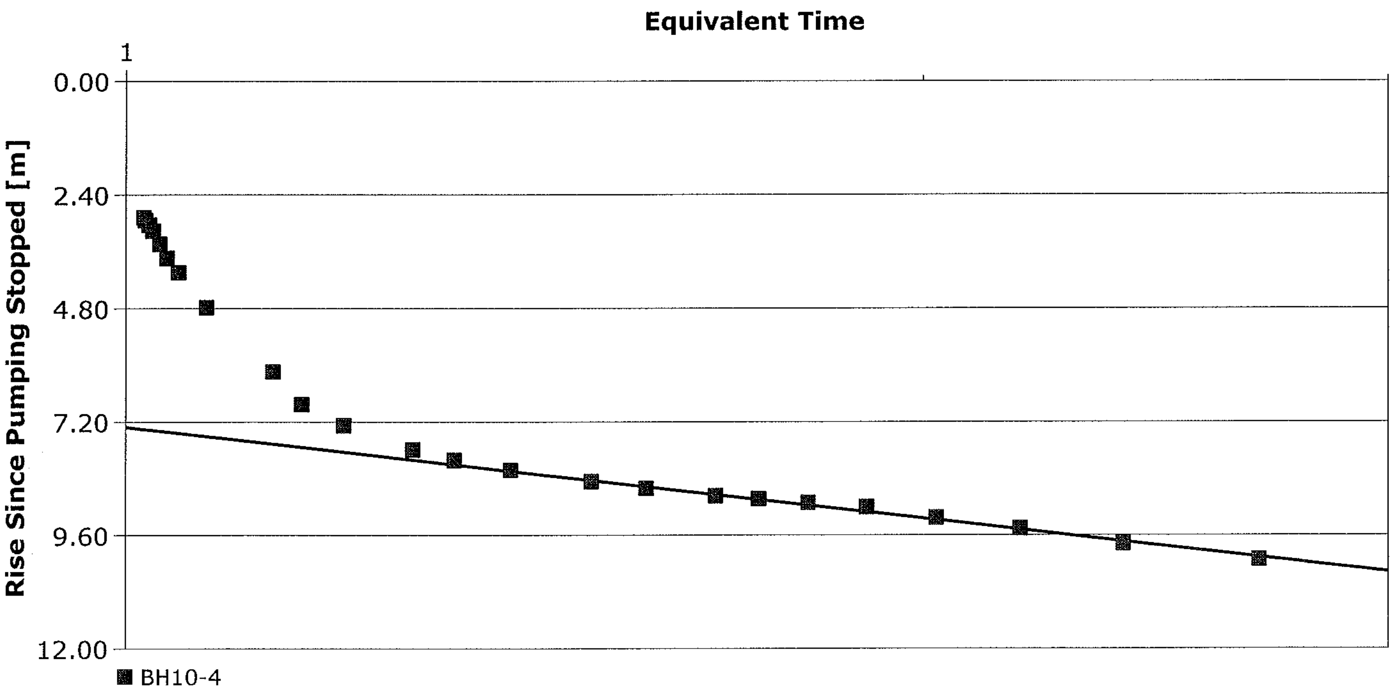
Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-4 Rising Head Test	Pumping Well: BH10-4
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by: JM	Theis Analysis Recovery Period	Analysis Date: 11/18/2010
Aquifer Thickness: 12.32 m	Discharge: variable, average rate 5.9889E-5 [m³/min]	



Calculation after Theis & Jacob

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-4	2.47×10^{-3}	2.00×10^{-4}	0.02	

Houle Chevrier Engineering

Pumping Test - Water Level Data

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road

Pumping Test: BH10-6 Rising Head Test

Pumping Well: BH10-6

Test Conducted by: BP

Test Date: 11/1/2010

Discharge: variable, average rate 0.000126 [m³/min]

Observation Well: BH10-6

Static Water Level [m]: 2.04

Radial Distance to PW [m]: -

	Time [min]	Water Level [m]	Drawdown [m]
1	0	2.04	0.00
2	5	4.12	2.08
3	5.5	3.90	1.86
4	6	3.74	1.70
5	6.5	3.72	1.68
6	7	3.71	1.67
7	8	3.68	1.64
8	10	3.63	1.59
9	30	3.30	1.26
10	47	2.90	0.86
11	75	2.19	0.15
12	107	2.11	0.07
13	134	2.09	0.05
14	180	2.08	0.04

Houle Chevrier Engineering

Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road

Pumping Test: BH10-6 Rising Head Test

Pumping Well: BH10-6

Test Conducted by: BP

Test Date: 11/1/2010

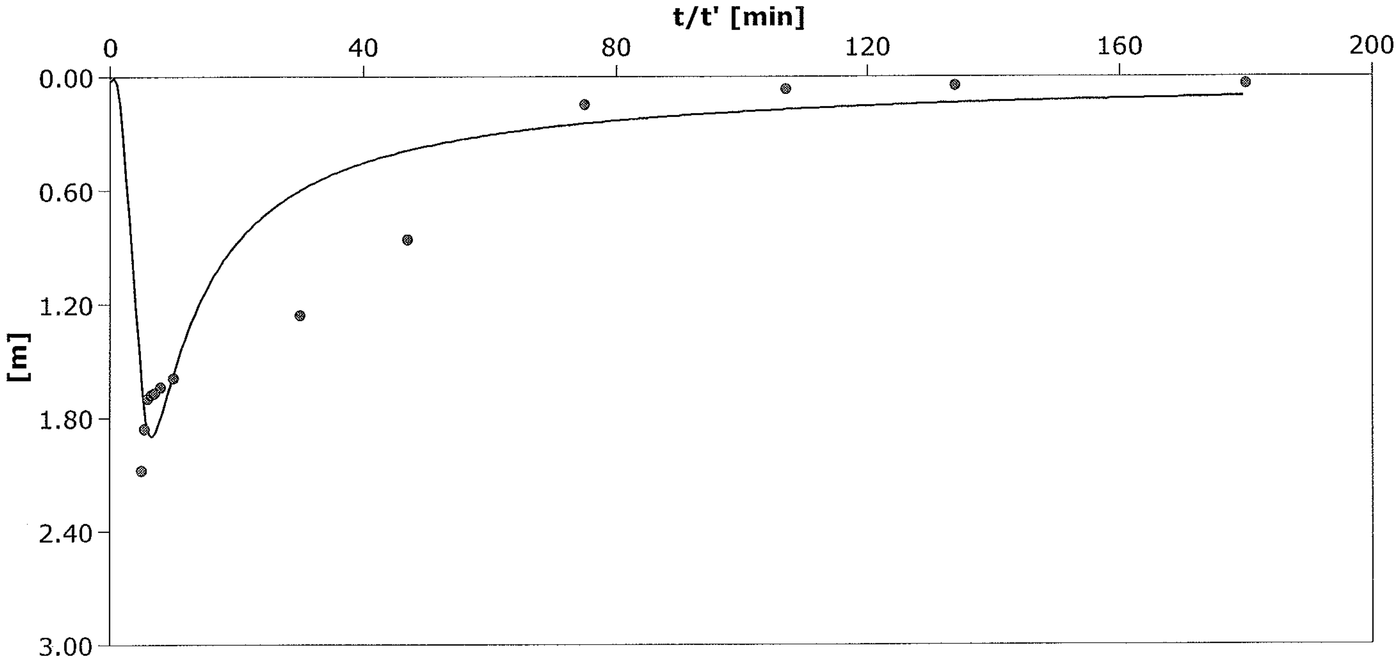
Analysis Performed by: JM

Theis Analysis

Analysis Date: 11/19/2010

Aquifer Thickness: 2.82 m

Discharge: variable, average rate 0.000126 [m³/min]

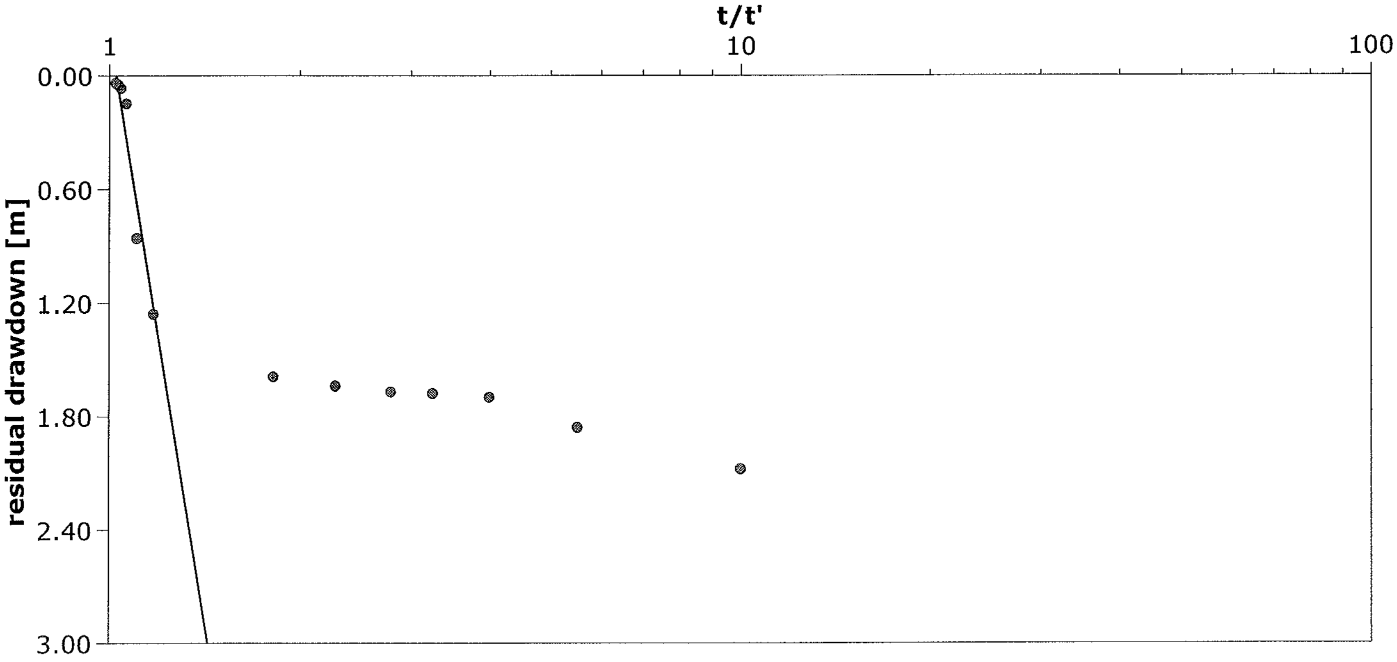


Calculation after Theis

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-6	1.54×10^{-1}	5.45×10^{-2}	0.02	

	Pumping Test Analysis Report
	Project: Office Building Geotechnical Investigation
	Number: 10-513
	Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-6 Rising Head Test	Pumping Well: BH10-6
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by:	New analysis 2	Analysis Date: 11/19/2010
Aquifer Thickness: 2.82 m	Discharge: variable, average rate 0.000126 [m³/min]	



Calculation after Theis & Jacob

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-6	1.60×10^{-3}	5.67×10^{-4}	0.02	

Houle Chevrier Engineering

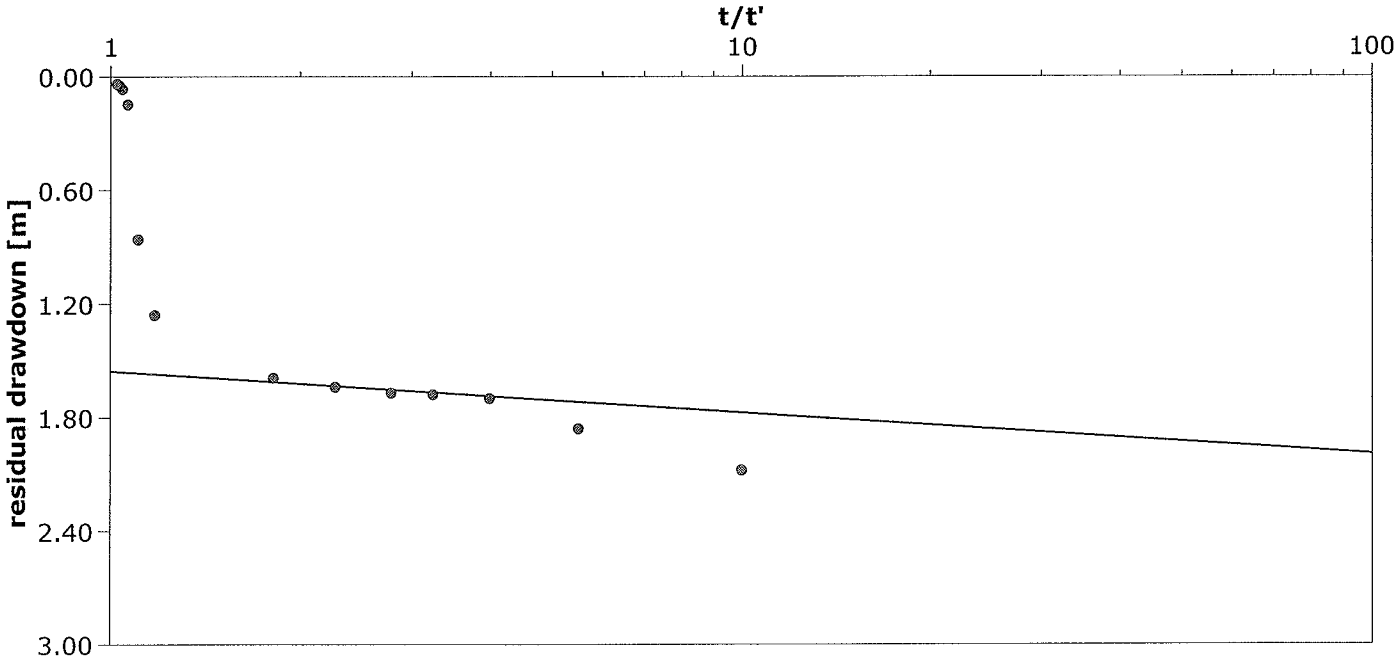
Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-6 Rising Head Test	Pumping Well: BH10-6
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by:	New analysis 2	Analysis Date: 11/19/2010
Aquifer Thickness: 2.82 m	Discharge: variable, average rate 0.000126 [m³/min]	



Calculation after Theis & Jacob

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-6	1.51 × 10^{-1}	5.37 × 10^{-2}	0.02	

Houle Chevrier Engineering

Pumping Test - Water Level Data Page 1 of 1

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road

Pumping Test: BH10-7 Rising Head Test

Pumping Well: BH10-7

Test Conducted by: BP

Test Date: 11/1/2010

Discharge: variable, average rate 6.7363E-5 [m³/min]

Observation Well: BH10-7

Static Water Level [m]: 0.24

Radial Distance to PW [m]: -

	Time [min]	Water Level [m]	Drawdown [m]
1	0	0.24	0.00
2	4	13.50	13.26
3	4.5	13.06	12.82
4	5	12.60	12.36
5	5.5	12.16	11.92
6	6	11.75	11.51
7	6.5	11.40	11.16
8	7	10.98	10.74
9	7.5	10.61	10.37
10	8	10.28	10.04
11	9	9.70	9.46
12	10	9.16	8.92
13	12	8.21	7.97
14	14	7.39	7.15
15	16	6.69	6.45
16	18	6.10	5.86
17	20	5.62	5.38
18	25	4.79	4.55
19	54	3.83	3.59
20	74	3.75	3.51
21	102	3.71	3.47
22	135	3.69	3.45
23	162	3.67	3.43
24	180	3.67	3.43

Houle Chevrier Engineering

Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road

Pumping Test: BH10-7 Rising Head Test

Pumping Well: BH10-7

Test Conducted by: BP

Test Date: 11/1/2010

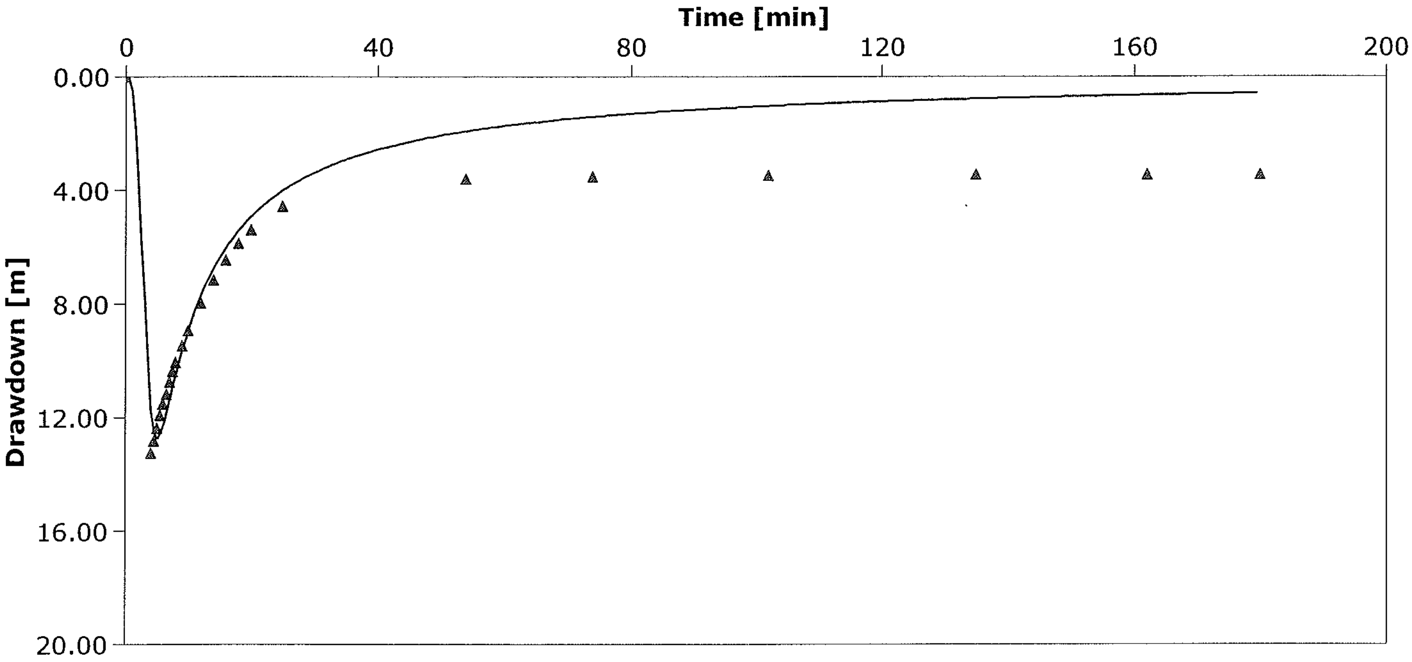
Analysis Performed by: JM

Theis Analysis

Analysis Date: 11/18/2010

Aquifer Thickness: 11.03 m

Discharge: variable, average rate 6.7363E-5 [m³/min]



Calculation after Theis

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Storage coefficient	Radial Distance to PW [m]	
BH10-7	1.75×10^{-2}	1.58×10^{-3}		0.02	

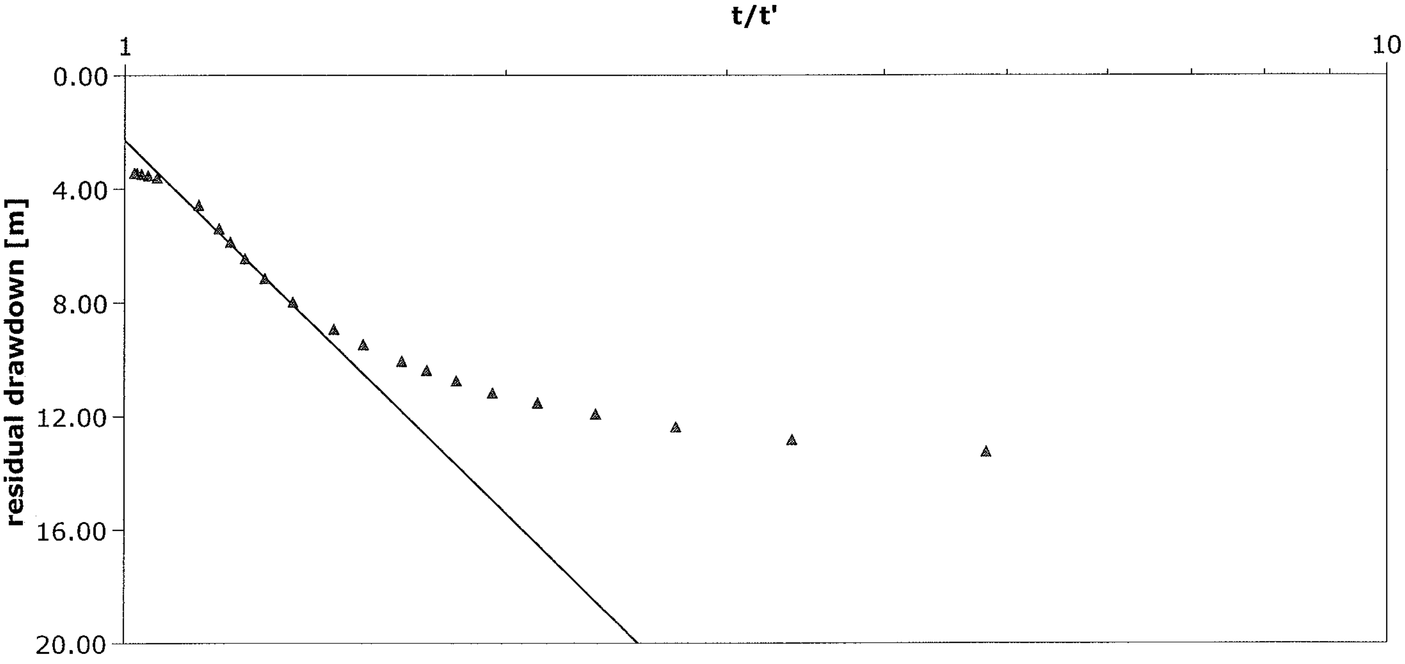
Pumping Test Analysis Report

Project: Office Building Geotechnical Investigation

Number: 10-513

Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-7 Rising Head Test	Pumping Well: BH10-7
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by:	New analysis 2	Analysis Date: 11/18/2010
Aquifer Thickness: 11.03 m	Discharge: variable, average rate 6.7363E-5 [m^3/min]	

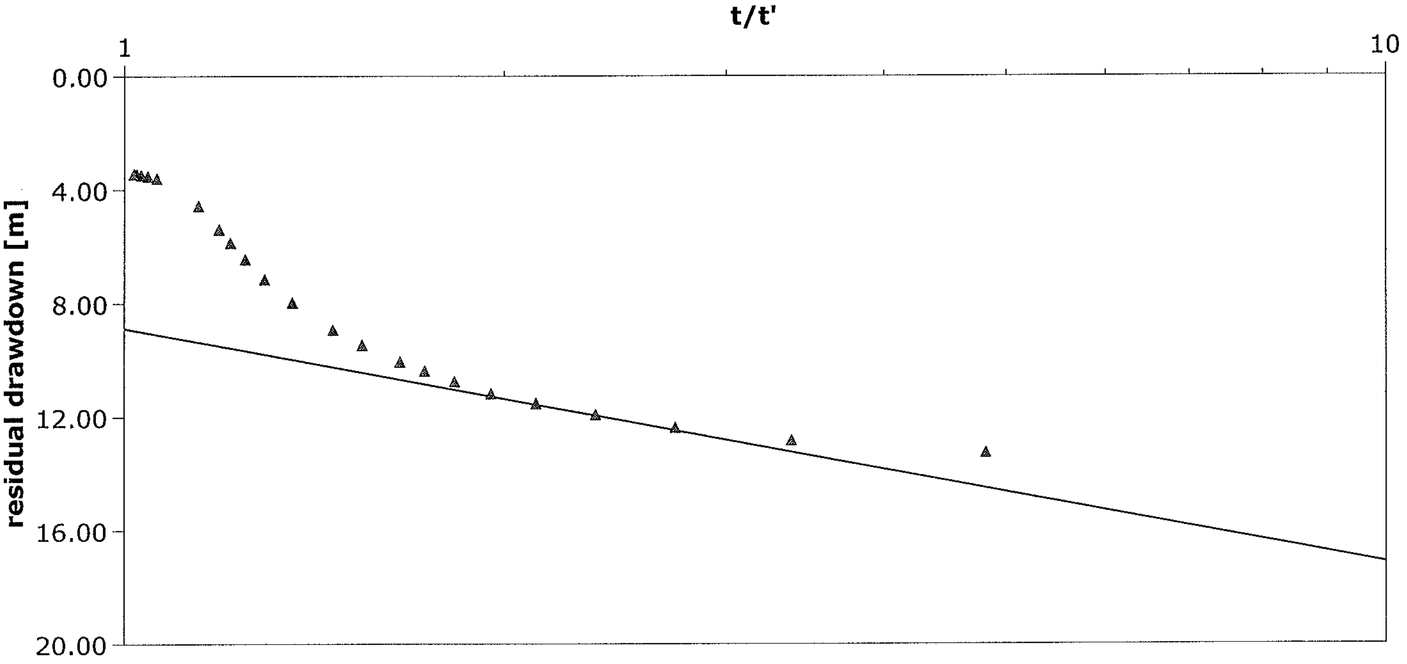


Calculation after Theis & Jacob

Observation Well	Transmissivity [m^2/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-7	4.07×10^{-4}	3.69×10^{-5}	0.02	

Houle Chevrier Engineering	**Pumping Test Analysis Report**
	Project: Office Building Geotechnical Investigation
	Number: 10-513
	Client: Canderel

Location: 485 Richmond Road	Pumping Test: BH10-7 Rising Head Test	Pumping Well: BH10-7
Test Conducted by: BP		Test Date: 11/1/2010
Analysis Performed by:	New analysis 2	Analysis Date: 11/18/2010
Aquifer Thickness: 11.03 m	Discharge: variable, average rate 6.7363E-5 [m³/min]	



Calculation after Theis & Jacob

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Radial Distance to PW [m]	
BH10-7	2.15×10^{-3}	1.95×10^{-4}	0.02	

APPENDIX D

CHEMICAL TEST RESULTS ON GROUNDWATER SAMPLE RELATING TO CORROSION

(EXOVA ACCUTEST. REPORT No. 1027581)

EXOVA ACCUTEST

REPORT OF ANALYSIS

Client: **Houle Chevrier Engineering**
180 Wescar Lane, R.R. #2
Carp, ON
K0A 1L0
Attention: **Mr. John Cholewa**

Report Number: 1027581
Date: 2010-11-16
Date Submitted: 2010-11-09

Project: 10-513

P.O. Number:
Matrix: Water

Chain of Custody Number: **125752**

LAB ID:			844011	844012	844013			**GUIDELINE**		
Sample Date:			2010-11-09	2010-11-09	2010-11-09			ODWSOG		
Sample ID:			BH10-4	BH10-6	BH10-7					
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	mg/L	1	516	1340	439			AO	250	mg/L
Conductivity	uS/cm	5	3080	5180	2620					
pH			8.16	7.69	8.22				6.5-8.5	
Sulphate	mg/L	1	569	116	412			AO	500	mg/L

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Ewan McRobbie
Inorganic Lab Supervisor

Methods references and/or additional QA/QC information available on request.