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November 11, 2013

Our ref: 13-339

Starbank Developments 401 Corporation
329 Brooke Avenue
Toronto, Ontario
M5M 2L4

Attention: Mr. Dung Lam

RE: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
401 MARCH ROAD, OTTAWA, ONTARIO

Dear Mr. Lam,

Attached is our geotechnical report for the proposed commercial development at 401 March Road in, Ottawa, Ontario.

We trust that this report provides sufficient information for your current purposes. Please contact us if you have any questions concerning the report.

Yours truly,

HOULE CHEVRIER ENGINEERING LTD.

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

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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
401 MARCH ROAD
OTTAWA, ONTARIO

Submitted to:

Starbank Developments 401 Corporation
329 Brooke Avenue
Toronto, Ontario
M5M 2L4

November 2013

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed commercial development located at 401 March Road in the City of 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 project, including construction considerations, which could influence design decisions.

This investigation was performed in accordance with our proposal dated July 4, 2013.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct a commercial development on a vacant lot located at 401 March Road in the City of Ottawa, Ontario (Key Plan, Figure 1). Two site plan options were provided to us for the proposed development. Proposed design option 1 includes a gas bar, pumping island, underground fuel storage tank area and three (3) commercial buildings (Site Plan Option 1, Figure 2). Proposed design option 2 includes a gas bar, pumping island, underground fuel storage tank area, car wash and three (3) commercial buildings (Site Plan Option 2, Figure 3). At the time this report was prepared, details of the proposed structures were not available; however, it is understood that the buildings will be of slab on grade (i.e. basementless) construction. Exterior on-grade parking and access roadways are also included in the scope of the project.

The site of the proposed commercial development is covered with tall grass, shrubs and trees. The site is bordered by March Road to the northeast, Station Road to the northwest, and an existing rail line to the south.

2.2 Review of Geology Maps

Published geology maps of the area indicate that the subsurface conditions are expected to consist of deposits of sensitive silty clay. The thickness of the overburden is mapped as 15 to 25 metres. Bedrock geology maps indicate that the overburden is underlain by Precambrian bedrock.

3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on October 7 and 8, 2013. During that time, nine (9) boreholes were advanced at the site using a track mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-Rouge, Quebec. The locations of the boreholes were based on Site Plan Option 1. Details of the test holes are provided below:

- Four (4) boreholes, numbered 13-1, 13-3, 13-6 and 13-9, were advanced in the area of the proposed building footprints. Three (3) of these boreholes, numbered 13-1, 13-6 and 13-9, were advanced to between 4.5 and 6.1 metres below ground surface. One (1) borehole, numbered 13-3, was advanced to a depth of about 18.8 metres below ground surface to identify the Site Class for the seismic design of the structure. A dynamic cone was advanced in this test hole from a depth of about 15.9 to 18.8 metres below ground surface.

- Two (2) boreholes, numbered 13-5 and 13-8, were advanced to about 6.1 metres below ground surface in the areas of the proposed gas pumps and underground storage tanks.

- Three (3) boreholes, numbered 13-2, 13-4 and 13-7, were advanced to between about 1.8 and 4.4 metres below ground in the proposed parking and access roadway areas.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. In situ shear vane testing was carried out in boreholes 13-1, 13-3, 13-5 and 13-8 to measure the undrained shear strength of the silty clay. A well screen was sealed in the overburden soil at boreholes 13-1, 13-5 and 13-9 to measure the groundwater levels. Two soil samples from boreholes 13-5 and 13-9 were submitted to Paracel Laboratories Ltd. 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 who directed the drilling operations, logged the samples and carried out the in-situ testing. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, grain size and Atterberg limits.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the chemical analysis of the soil samples relating to corrosion are provided in Appendix B. The results of the laboratory classification tests on the soil samples are provided on Figures 4 and 5, and the Record of Borehole Sheets.

The test hole locations were determined relative to existing site features by Houle Chevrier Engineering Ltd. personnel. The test hole locations and elevations were measured using our Trimble R8 GPS survey instrument. The elevations are referenced to Geodetic datum. The approximate locations of the test holes are shown on the Site Plan, Figure 2.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater 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.

In summary, the soil conditions encountered across the site generally consist of a layer of fill material underlain by a weathered silty clay crust, which in turn is underlain by firm to stiff, grey silty clay. The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

4.2 Fill Material

A layer of fill was encountered from ground surface in all of the boreholes. The thickness of fill ranges from about 1.1 to 2.4 metres. The fill material is variable in nature and can generally be described as silty clay with varying amounts of sand, gravel, organic material, wood and pieces of asphaltic concrete.

Standard penetration tests carried out in the fill gave N values of 5 to 17 blows per 0.3 metres of penetration which indicates a very stiff consistency.

The moisture content of the fill material ranges from about 18 to 33, averaging 26 percent.

4.3 Former Topsoil

A 0.1 to 0.5 metre thick layer of dark brown silty clay/clayey silt with organic material (former topsoil) was encountered in boreholes 13-2, 13-7 and 13-8.

The moisture content of a sample of the former topsoil was obtained. The results show that the moisture content for the former topsoil is about 35 percent.

4.4 Weathered Silty Clay Crust

Deposits of silty clay were encountered below the fill and former topsoil at depths ranging from about 1.1 to 2.4 metres (elevation 81.3 to 84.5 metres, geodetic datum).

The upper part of the silty clay is weathered and grey brown. Where fully penetrated at boreholes 13-1, 13-3, 13-5, 13-6, 13-8 and 13-9, the weathered crust has a thickness of 2.0 to 3.5 metres and extends to depths ranging from 3.5 to 4.6 metres below ground surface (elevation 79.1 to 81.0 metres, geodetic datum).

The results of in situ vane shear strength testing carried out in the lower portion of the weathered silty clay crust indicate undrained shear strength values ranging from about 88 to 96 kilopascals, which reflect a stiff consistency. Standard penetration tests carried out in this layer gave N values ranging from 3 to 19 blows per 0.3 metres of penetration, which reflect a very stiff to stiff consistency.

One (1) grain size distribution test was carried out on a sample of the weathered silty clay from borehole 13-8. The test results are provided on Figure 4.

One (1) Atterberg limit test was undertaken on a sample of the weathered silty clay crust recovered from borehole 13-8 at a depth of about 3.0 metres. The results show that the sample of weathered silty clay has a liquid limit of 43 percent and a plastic limit of 17 percent; as indicated on the plasticity chart on Figure 5, this reflects a low plasticity.

The moisture content of the weathered silty clay crust ranges from about 29 to 45, averaging 36 percent.

4.5 Grey Silty Clay

Grey silty clay was encountered below the weathered silty clay crust in boreholes 13-1, 13-3, 13-5, 13-6, 13-8 and 13-9 at depths ranging from about 3.8 to 4.6 metres below ground surface (elevation 79.1 to 81.0 metres, geodetic datum). The thickness of the grey silty clay deposit in borehole 13-3 is about 10.8 metres.

The results of in situ vane shear strength testing carried out in the grey silty clay indicate undrained shear strength values ranging from about 42 to 88 kilopascales, which reflect a firm to stiff consistency. Standard penetration tests carried out in the grey silty clay gave N values of “static weight of hammer (WH)” to 5 blows per 0.3 metres of penetration.

One (1) grain size distribution test was carried out on a sample of the silty clay from borehole 13-8. The test result is provided on Figure 4.

One (1) Atterberg limit test was undertaken on a sample of the grey silty clay recovered from borehole 13-8 at a depth of about 4.5 metres. The results show that the silty clay has a liquid limit of 44 percent and a plastic limit of 18 percent; as indicated on the plasticity chart on Figure 5, this reflects a low plasticity.

The moisture content of the grey silty clay ranges from about 45 to 68, averaging 55 percent.

Boreholes 13-1, 13-4, 13-6, 13-8 and 13-9 were terminated within the silty clay layer.

4.6 Possible Glacial Till

A layer of possible glacial till was encountered in borehole 13-3 below the silty clay layer at a depth of about 15.2 metres. The glacial till consists of sandy silt with clay and gravel. Cobbles and boulders should be expected within the glacial till.

One standard penetration test carried out in the possible glacial till layer gave an N value of 17 blows per 0.3 metres of penetration, which reflects a compact relative density.

A dynamic cone was advanced from about 15.9 metres to 18.8 metres and gave N values of "static weight of hammer (WH)" to 22 blows per 0.3 metres of penetration. The dynamic cone penetration test was terminated at a depth of about 18.8 metres due to refusal on inferred bedrock.

The moisture content of the possible glacial till is about 30.5 percent.

4.7 Groundwater Levels

The groundwater levels measured in the standpipes installed in boreholes 13-1, 13-5 and 13-9 ranged from about 1.2 to 2.3 metres below existing surface grade (elevation 81.4 to 83.3 metres, geodetic datum) on October 16, 2013.

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

4.7 Soil Chemistry Relating to Corrosion

The results of chemical testing on samples of soil recovered from boreholes 13-5 and 13-9 at about 3.0 and 2.3 metres below ground surface, respectively, are provided in Appendix B and summarized in the following table:

Parameter	Borehole 13-05 (Sample No. 5)	Borehole 13-09 (Sample No. 4)
pH	7.50	7.32
Resistivity (Ohm.m)	35.4	30.9
Chloride (micrograms per gram)	10	48
Sulphate (micrograms per gram)	66	111

5.0 PROPOSED COMMERCIAL DEVELOPMENT

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 Proposed Buildings

5.2.1 Excavation

The excavation for the proposed buildings may be carried out through fill and weathered silty clay.

The excavation sides should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the act, soils at this site can be classified as Type 3. That is, open cut excavations within overburden deposits should be sloped at 1 horizontal to 1 vertical, or flatter from the base of the excavation to surface grade.

All foundation excavations should be undertaken with an excavator equipped with a smooth bucket to minimize disturbance of the sensitive subgrade soils. Based on our previous experiences at sites underlain by silty clay, it is possible that the upper 0.3 to 0.5 metres of the weathered silty clay may be affected by past frost action and may unavoidably “peel” during excavation. If this occurs, an allowance should be made to remove and replace any disturbed silty clay with compacted granular material within the building areas.

The groundwater levels measured at boreholes 13-1, 13-5 and 13-9 ranged from about 1.2 to 2.3 metres below existing surface grade (elevation 83.3 to 81.4 metres, geodetic datum) on October 16, 2013. Based on our previous experience, groundwater inflow from the silty clay deposits should be relatively small and controlled by pumping from filtered sumps within the excavations. Suitable detention and filtration will be required before discharging the water to a sewer or ditch.

It should be noted that groundwater levels would be expected to fluctuate seasonally.

5.2.2 Spread Footing Design

Based on the results of the subsurface investigation, the proposed gas bar, car wash and commercial buildings could be founded on spread or pad footings bearing on or within undisturbed silty clay. All organic material, fill material, topsoil, and loose or water softened soils should be removed within the areas.

The bearing pressures for spread or pad footing foundations at this site are based on the necessity to limit the stress increase on the softer grey silty clay layer below the weathered crust to an acceptable level so that foundation settlements will not be excessive. Four important parameters in calculating the stress increase on the silty clay are:

1) The underside of footing elevation (depth of excavation);
2) The size and type (i.e., pad or strip), and loading of the foundation;
3) The amount of surcharge (fill, etc.) in the vicinity of the foundation; and
4) The amount of post-development groundwater lowering at the site;

There are many possible combinations of founding depths, footing sizes and thickness of fill which might be suitable for this site. For the purpose of this analysis we have assumed that the finished ground floor in the proposed buildings will be at about elevation 84.0 metres. In addition, we have considered a long term groundwater lowering at the site equal to 1 metre below the current groundwater level. For preliminary planning and design purposes, the following is one example that could be considered for the design of the building foundations:

1. Found the exterior strip and pad footings at about 1.5 metres below the proposed exterior ground level for frost protection purposes.

2. Found the interior pad footings at about 0.5 metres below the proposed interior finished floor elevation.

3. For the purpose of the analysis, we have assumed that the fill material will consist of imported OPSS Granular B Type II.

The preliminary geotechnical details for this foundation scenario are presented in the following table.

Summary of Preliminary Foundation Bearing Pressures

Type of Footing	Minimum Elevation of Footing[2] (metres)	Maximum Size of Footing (metres)	Factored Net Geotechnical Reaction at Serviceability Limit State (SLS)[1] (kilopascals)	Factored Net Geotechnical Reaction at Ultimate Limit State (ULS) (kilopascals)
Exterior Strip	82.35	0.9	120	250
Interior Pad	83.50	2.0 square	120	250

Notes:
1. The total and differential settlement of the foundation at SLS should be less than 20 and 25 millimetres, respectively.
2. Preliminary bearing values assume a finished ground floor elevation in the proposed buildings of 84.0 metres, geodetic datum

The post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

There are many other possible combinations of finished floor elevations, foundation depths and footing sizes which might be suitable for this site. All other alternatives must be checked by the geotechnical engineer to ensure that overstressing of the softer silty clay soil does not occur, as this could result in excessive settlement and cracking/distress of the structures. The bearing pressures given in the above tables may have to be reduced if:

- The footing sizes are larger than that given above;
- The footings are founded deeper than anticipated;
- The sustained slab-on-grade load exceeds 2.0 kilopascals; OR
- The horizontal separation distance between footings is less than 2.0 metres.

5.2.3 Engineered Fill Below Footings

Fill material was encountered in all of the boreholes that were advanced at the site. Any fill material that is encountered below founding level should be removed. The grade could then be raised with engineered fill material composed of granular material meeting OPSS Granular B Type II. The engineered fill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density. To allow adequate spread of load, the engineered fill should extend at least 0.3 metres beyond the edge of the footings and down and out from that point at 1 horizontal to 1 vertical, or flatter.

5.2.4 Seismic Design of Proposed Structure

The site classification for seismic site response may be taken as Site Class D as per the 2010 National Building Code of Canada Table 4.1.8.4.A. In our opinion, there is no potential for liquefaction of the overburden deposits at this site.

5.2.5 Grade Raise Restrictions

Based on the undrained shear strength measurements within the grey silty clay deposit, this material generally has a firm to stiff consistency and has a limited capacity to support loads from footings, grade raise fill, and equipment. The proposed finished grades and floor slab elevation for the commercial buildings and gas bar were not available at the time of writing this report. However, for design purposes, we have assumed a finished floor slab elevation of 84.0 metres.

Any changes to the design grade raise and finished floor slab elevation of 84.0 metres should be reviewed by the geotechnical engineer to assess the potential for long term consolidation of the silty clay and settlement of the structure.

5.2.6 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) piers that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Details on foundation insulation could be provided, if required.

5.2.7 Foundation Backfill and Drainage

The native soil deposits at this site are highly frost susceptible and should not be used as backfill against foundations, piers, etc. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material meeting OPSS Granular B Type I or II requirements. Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed buildings, 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 native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the bottom of the excavation or 1.5 metres below finished grade, whichever is less, 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.

Perimeter foundation drainage is not considered necessary for slab on grade structures at this site provided that the floor slab level is above the finished exterior ground surface level at the building.

5.2.8 Slab-on-Grade Support (Heated Areas Only)

To prevent long term settlement of the floor slabs, all fill material, former topsoil, organic, loose, wet or deleterious material should be removed from below the slab on grade.

The grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II. The use of Granular B Type II is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slabs be composed of virgin material (100 percent crushed rock) or native pit run material only for environmental reasons.

All imported granular materials placed below the proposed floor slabs should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

Where any interior areas of the buildings will be unheated, thermal protection for the subgrade will be required where less than 1.5 metres of non-frost susceptible fill cover will exist below the floor slab. Further details on the insulation requirements could be provided, if necessary.

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.

5.3 Underground Fuel Storage Tanks

5.3.1 Slab Support

Based on the results of the investigation, the proposed tank slab could be founded on native silty clay (weathered crust) or engineered fill above the native silty clay. Any fill, topsoil, disturbed, soft or deleterious materials should be removed from below the tank footprint.

If necessary, the grade below the proposed tank slabs could be raised with imported granular material meeting OPSS Granular B Type II (engineered fill). To allow adequate spread of load, the engineered fill should extend at least 0.3 metres beyond the sides of the concrete slab and down and out from this point at 1 horizontal to 1 vertical, or flatter. The engineered fill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

A granular bedding layer consisting of at least 150 millimetres of granular material meeting OPSS requirements for Granular A placed directly below the slab.

5.3.2 Fuel Tank Backfill

The native soils at this site are potentially frost susceptible and should not be used as backfill against the fuel storage tanks. To avoid frost adhesion and possible heaving, the tanks should be backfilled with imported, free-draining, non-frost susceptible granular material such as those meeting OPSS Granular B Type I or II requirements or clear crushed stone. Any backfill material should meet the underground tank manufacturer's requirements.

Where the backfill will ultimately support areas of hard surfacing (sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Alternatively, if backfill is to be placed "in the wet", 19 millimetre clear crushed stone could be used, provided that a suitable nonwoven geotextile separator is placed between the clear crushed stone and any sandy backfill material and native soil. The clear crushed stone should be compacted by tamping with the bucket of the excavator.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed tanks, 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 native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.8 metres below ground surface to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 3 horizontal to 1 vertical, or flatter.

The lateral earth pressure on the tanks should be designed to resist the following earth pressures:

$$P_o = K_o \gamma_s z + K_o q + \text{compaction stress}$$

Where,

P_o = At rest earth pressure on the tank (kilopascals)
K_o = At rest earth pressure coefficient
γ_s = Unit weight of backfill material (kilonewtons per cubic metre)
z = Depth below ground surface (metres)
q = Uniform surcharge at ground surface to take into account traffic, equipment, or stockpiled soil (typically 10 kilopascals or more)

The following earth pressure parameters could be used to calculate the horizontal earth pressure on the proposed structure:

	Earth Pressure Parameters for free draining sand and gravel meeting OPSS Granular B Type I	Earth Pressure Parameters for OPSS Granular B Type II	Earth Pressure Parameters for Clear Crushed Stone
Material Bulk Unit Weight, γ (kN/m^3):	21.5	22.0	17.0
Material Buoyant Unit Weight, γ (kN/m^3):	11.7	12.2	7.2
Estimated Friction Angle (degrees)	35	40	30
"At Rest" Earth Pressure Coefficient, K_0, assuming horizontal backfill behind the structure	0.43	0.36	0.50

The proposed tanks should be designed to resist uplift due to hydrostatic pressures below the base of the structures. The groundwater levels, which were measured in the well screens in borehole 13-5 was at elevation 81.4 metres on October 16, 2013. Given that the groundwater levels could be higher during spring thaw conditions, we suggest that a design groundwater level of at least 82.5 metres be used to assess buoyant conditions.

5.4 Proposed Services

5.4.1 Excavation for the Services

The excavation for the sewer and watermain services will be carried out mostly through fill materials and weathered silty clay.

The excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 Soil. The excavation for rigid service pipes should be in accordance with OPSD 802.031 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 should be in accordance with OPSD 802.010 and OPSD 802.031 for flexible and rigid pipes, respectively. 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, topsoil, 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 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.

It is noted that the silty clay, deposits at this site are sensitive to disturbance and construction traffic. Disturbance to the silty clay subgrade can occur during excavation due to flow of soil between the teeth on a standard bucket. To reduce disturbance, the excavating equipment could be equipped with a bucket with a flat blade.

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 (access roadway, parking lot, 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 or II.

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.

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 has 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.

The soils at this site are highly frost susceptible and are prone to significant ice lensing. In order to carry out the work during freezing temperatures and maintain adequate performance of the trench backfill as a roadway subgrade, the service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The 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.

5.4.4 Seepage Barriers

To prevent the granular bedding in the services trench from acting as a “French Drain” and thereby promoting groundwater lowering below that which was assumed in the analysis, seepage barriers should be installed along the service trenches just inside the property lines. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The locations of the seepage barriers could be provided at the final design stage.

5.5 Access Roadway and Parking Areas

5.5.1 Subgrade Preparation

In preparation for the construction of the access roadway and parking areas at this site any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. This need not include removal of the existing fill material provided that some minor post construction settlement of the pavement structure can be tolerated. Prior to placing granular fill for the parking areas and access roadway, the exposed subgrade should be proof rolled with a large (minimum 10 tonne) vibratory steel drum roller under dry conditions and inspected and approved by geotechnical personnel. Any soft areas that are evident from the proof rolling should be subexcavated and replaced with suitable earth borrow.

It is our experience that the upper part of the weathered silty clay (i.e., within 0.3 to 0.5 metres from original ground surface) may be impacted by past frost action. During removal of the topsoil and fill material, the upper part of the silty clay could unavoidably peel upwards and become disturbed. Where this occurs in the proposed parking and access roadway areas, the upper part of the silty clay should be re-compacted in place using suitable compaction equipment.

Should it be necessary to raise the roadway/parking area grades, the grade raise fill for the roadway/parking areas could consist of material which meets OPSS specifications for Granular B Type I or II, Select Subgrade Material, or suitable earth borrow. The grade raise fill should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. It is noted, however, that clayey and silty earth borrow materials are sensitive to changes in moisture content, precipitation and frost heaving. As such, unless the earth material placement is planned during the dry period of the year (June to September), precipitation and freezing conditions may restrict or delay adequate compaction of these materials. Based on our experience, clayey earth borrow materials should be compacted within 0 to 4 percent above the optimum moisture content, as defined by the standard Proctor test, to reduce the post construction settlement of the fill material. Depending on the weather conditions, it may be necessary to allow the material to dry prior to compaction.

The thickness of the grade raise fill below the access roadway/parking areas should comply with the requirements given in Section 5.2.5 Site Grade Raise Restrictions.

5.5.2 Flexible Pavement Structures for the Parking Lots and Access Roadways

It is suggested that parking areas to be used by light vehicles (cars, etc.) be constructed using the following minimum pavement structure:

50 millimetres of asphaltic concrete, over
150 millimetres of OPSS Granular A base, over
300 millimetres of OPSS Granular B Type II subbase

For the access roadways and the areas that are used by trucks the suggested minimum pavement structure is:

90 millimetres of asphaltic concrete, over
150 millimetres of OPSS Granular A base, over
450 millimetres of OPSS Granular B Type II subbase

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

Where the new pavement will abut existing pavement on March Road and Station Road, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter to match the depths of the granular material(s) exposed in the existing pavement.

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 subbbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.5.3 Asphaltic Concrete Type

The asphaltic concrete in the parking areas should consist of 50 millimetres of Superpave 12.5. For any access roadways, the asphaltic concrete surfacing thickness should be increased to 90 millimetres (40 millimetres of Superpave 12.5 over 50 millimetres of Superpave 19.0).

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

5.5.4 Granular Material Compaction

The granular base and subbase materials for the parking areas and access roadways should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.

5.5.5 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. The subgrade surfaces should be crowned and shaped to drain to the ditches and the catch basins to promote drainage of the pavement granular materials.

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. Where ditches are used, the bottom of the OPSS Granular B Type II should be at least 0.3 metres above the bottom of the ditch and the granular material should extend to the ditch slopes.

5.6 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the soil samples recovered from boreholes 13-5 and 13-9 at about 3.0 and 2.2 metres below ground surface are 66 and 111 micrograms per gram, respectively. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the soil for these samples is below the moderate range. For this exposure condition, any concrete that will be in contact with the native soil should be batched with General Use (formerly known as Type 10 cement).

The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the resistivity and pH of the soil samples recovered from boreholes 13-5 and 13-9, the soil samples can be classified as nonaggressive towards unprotected steel. It is noted that the corrosivity could vary throughout the year due to the application sodium chloride for de-icing.

5.7 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, 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. However, the magnitude of the vibrations is expected to be much less than that required to cause damage to the nearby structures or services.

5.8 Winter Construction

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In the event that construction is required during freezing temperatures, the soil below the footings and floor slabs should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

5.9 Effects of Trees

This site is underlain by deposits of sensitive silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the sensitive silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or above the silty clay. Therefore, no deciduous trees should be permitted closer to the buildings (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees. For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees.

The effects of existing and future trees on the proposed buildings, services and other ground supported structures should be considered in the landscaping design.

5.10 Design Review and Construction Observation

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

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the proposed building and access roadways/parking areas 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.

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

Yours truly,

HOULE CHEVRIER ENGINEERING LTD.

Lauren Ashe, B.Sc., E.I.T.

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

LICENSED PROFESSIONAL ENGINEER
A.F. CHEVRIER
7974504
11 Nov. 2013
PROVINCE OF ONTARIO

KEY PLAN

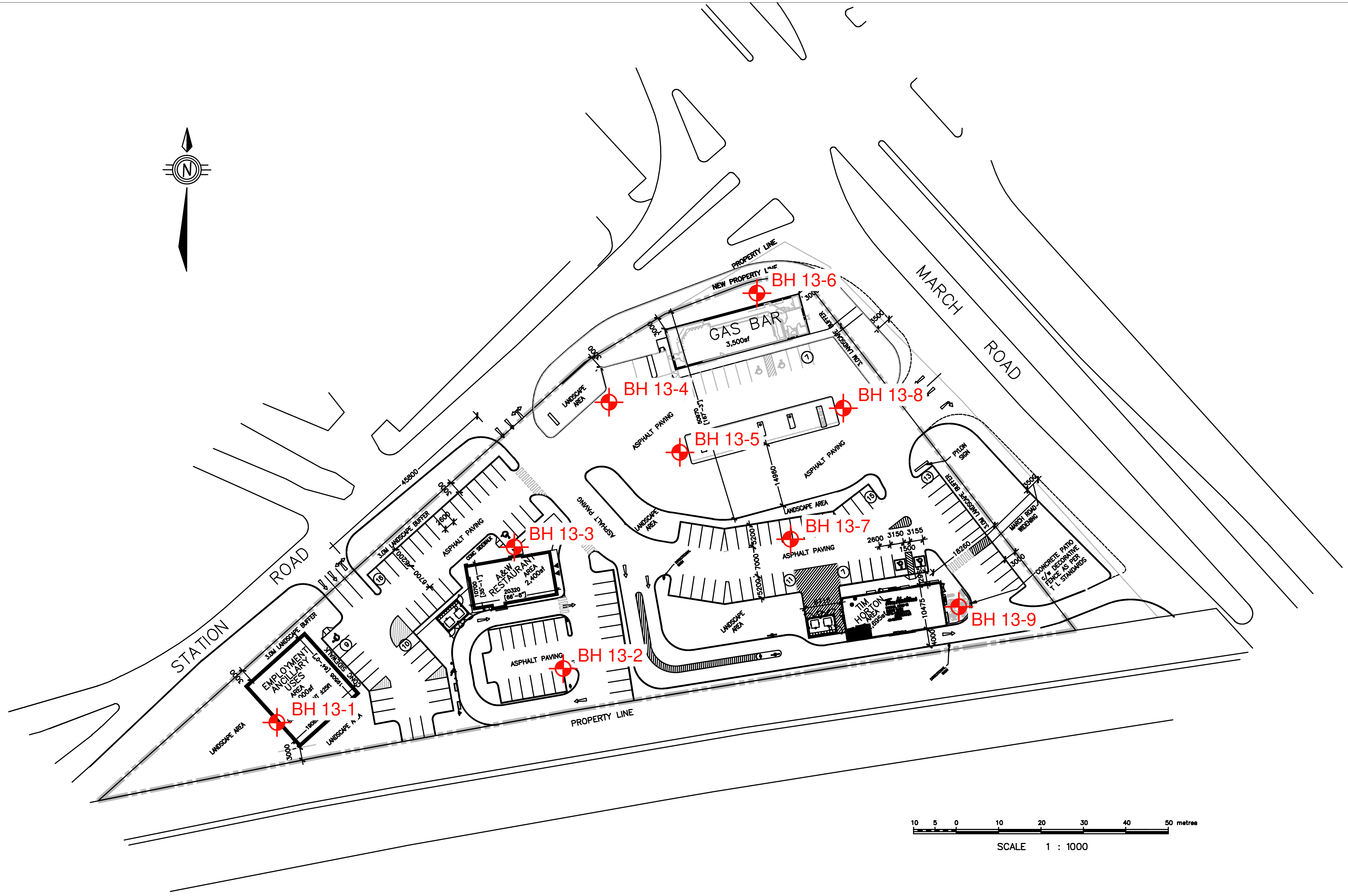
FIGURE 1

N.T.S

Houle Chevrier Engineering

Date November 2013

Project: 13-339



LEGEND

⊕ BH 13-1 APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.

Client STARBANK DEVELOPMENTS 2148 CORP

Location 401 MARCH ROAD
OTTAWA, ON

Revision 0

Drawn by D.J.R

Approved by A.F.C

Project No. 13-339

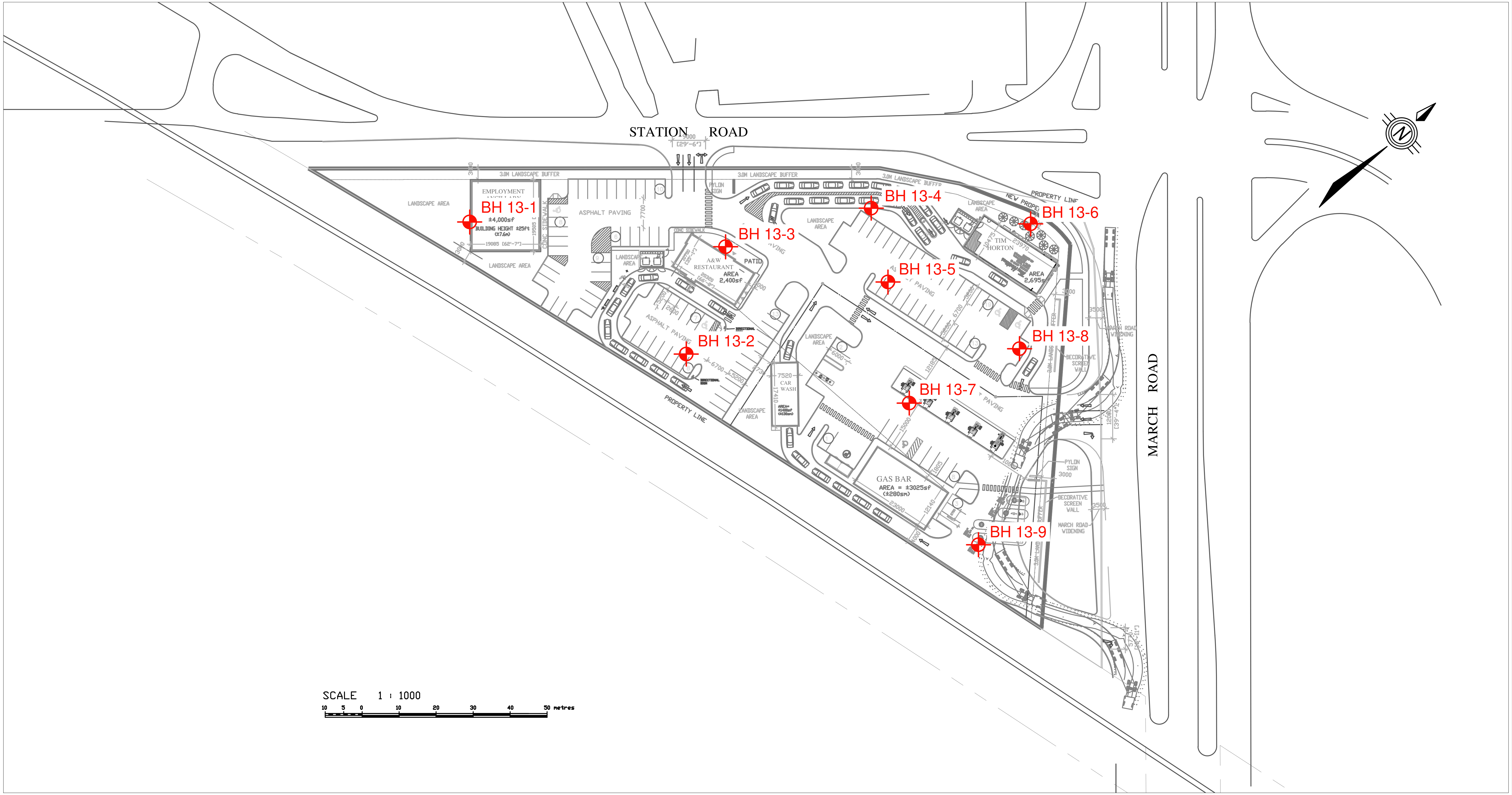
Scale 1 : 1000

Houle Chevrier Engineering

Title SITE PLAN OPTION 1

Date November 2013

FIGURE 2



LEGEND

BH 13-1 APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.

Client STARBANK DEVELOPMENTS 2148 CORP

Location 401 MARCH ROAD OTTAWA, ON

Revision 0

Drawn by D.J.R

Approved by A.F.C

Project No. 13-339

Scale 1 : 1000

Houle Chevrier Engineering

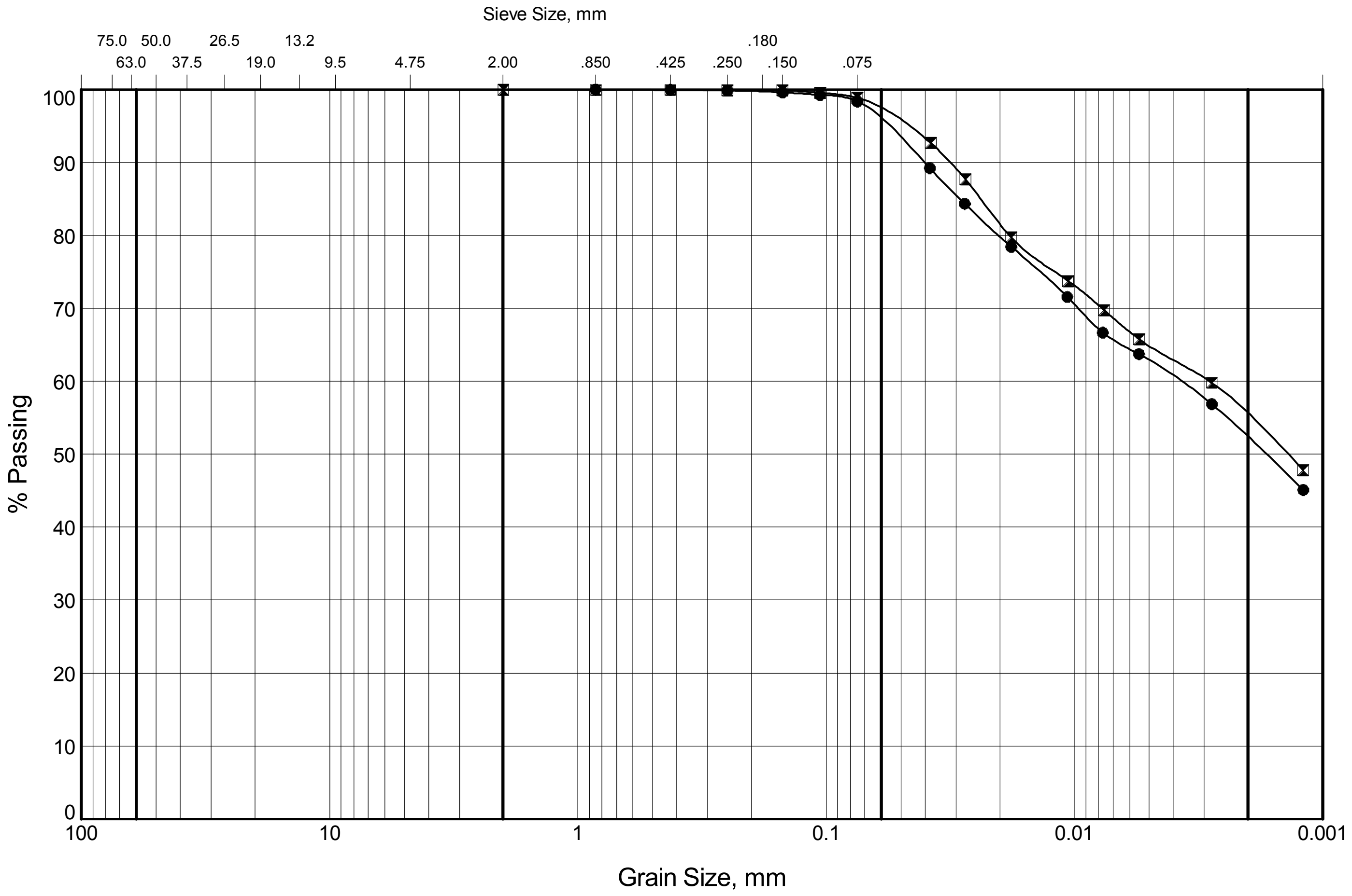
Title SITE PLAN OPTION 2

Date November 2013

FIGURE 3

GRAIN SIZE DISTRIBUTION

FIGURE 4



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

Borehole	**Sample**	**Depth (m)**	**Legend**
13-8	5	3.1 - 3.7	●
13-8	7	4.6 - 5.2	⧗

Houle Chevrier
Engineering

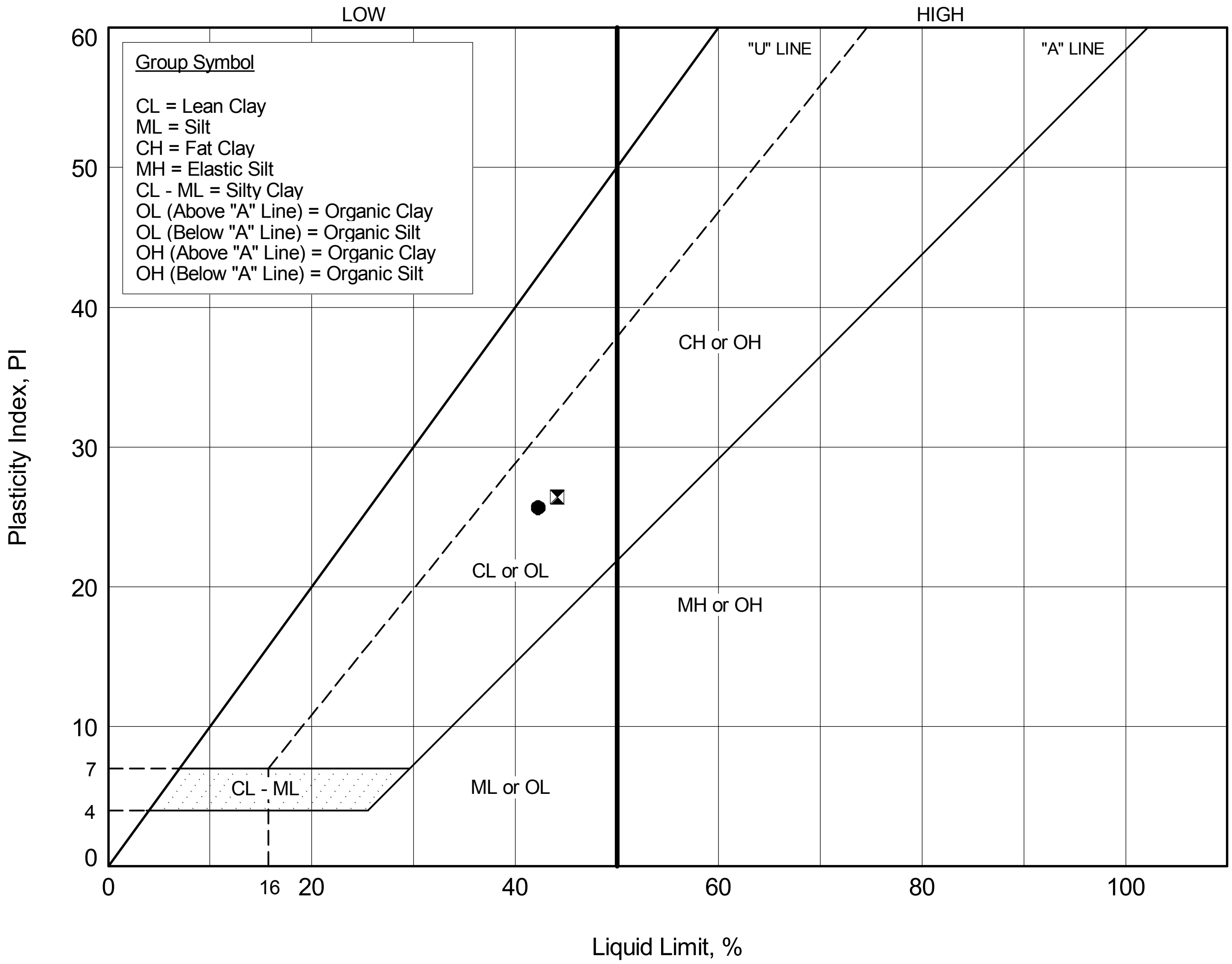
Date: November 2013

Project: 13-339

SOILS GRAIN SIZE GRAPH 13-339 BOREHOLE LOGS OCTOBER 9 2013.GPJ HOULE CHEVRIER FEB 9 2011.GDT 11/7/13

PLASTICITY CHART

FIGURE 5



Borehole	Sample	Depth (m)	Moisture Content, %	Legend
13-8	5	3.1 - 3.7	34.3	●
13-8	7	4.6 - 5.2	49.4	⧗

Houle Chevrier Engineering

Date: November 2013

Project: 13-339

APPENDIX A

ABBREVIATIONS AND SYMBOLS
RECORD OF BOREHOLE SHEETS

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

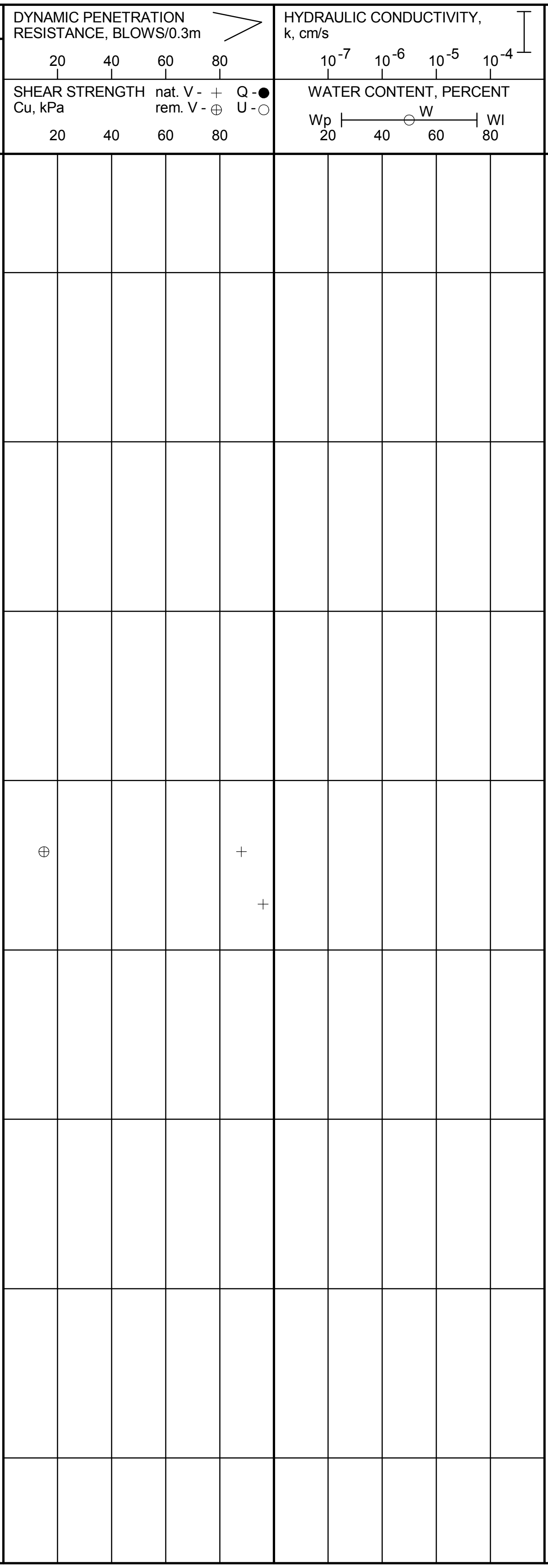
PROJECT: 13-339
LOCATION: See Site Plan, Figure 2
BORING DATE: October 7, 2013

RECORD OF BOREHOLE 13-1

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

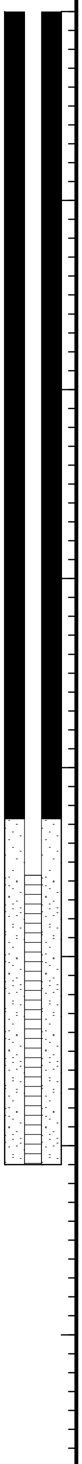
BORING METHOD: Power Auger, 200mm Diameter Hollow Stem Auger

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface	85.54			
	Very stiff, grey brown silty clay, some sand, trace gravel, trace organics, trace wood pieces (FILL MATERIAL)		1	50 D.O.	11
1	Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)	84.45 1.09	2	50 D.O.	19
2			3	50 D.O.	10
			4	50 D.O.	6
3			5	50 D.O.	3
4					
	Firm, grey SILTY CLAY	80.97 4.57	6	50 D.O.	WH
6	End of borehole	79.44 6.10			



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Above Ground Protector

Bentonite

Filter Sand

51mm diameter, 1.52m long slotted PVC pipe

Groundwater level at 2.27 metres below ground surface (elevation 83.27 metres geodetic datum) on October 16, 2013.

DEPTH SCALE
1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.
CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-2

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 7, 2013

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 W WI 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		84.32							
0	Power Auger 200mm Diameter Hollow Stem Auger	Very stiff, grey brown silty clay, some sand, trace gravel (FILL MATERIAL)			1	50 D.O.	5				Native Backfill
1					2	50 D.O.	6				
					3	50 D.O.	7				
2		Dark brown clayey silt, trace sand (Former TOPSOIL)		82.39 1.93							
		Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)		81.93 2.39	4	50 D.O.	6				
3					5	50 D.O.	7				Bentonite
4					6	50 D.O.	5				
		End of borehole		79.90 4.42							
5											
6											
7											
8											

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

BOREHOLE_RECORD WITH LAB WC 13-339 BOREHOLE LOGS OCTOBER 9 2013.GPJ HCE DATA TEMPLATE.GDT 10/29/13

PROJECT: 13-339

RECORD OF BOREHOLE 13-3

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

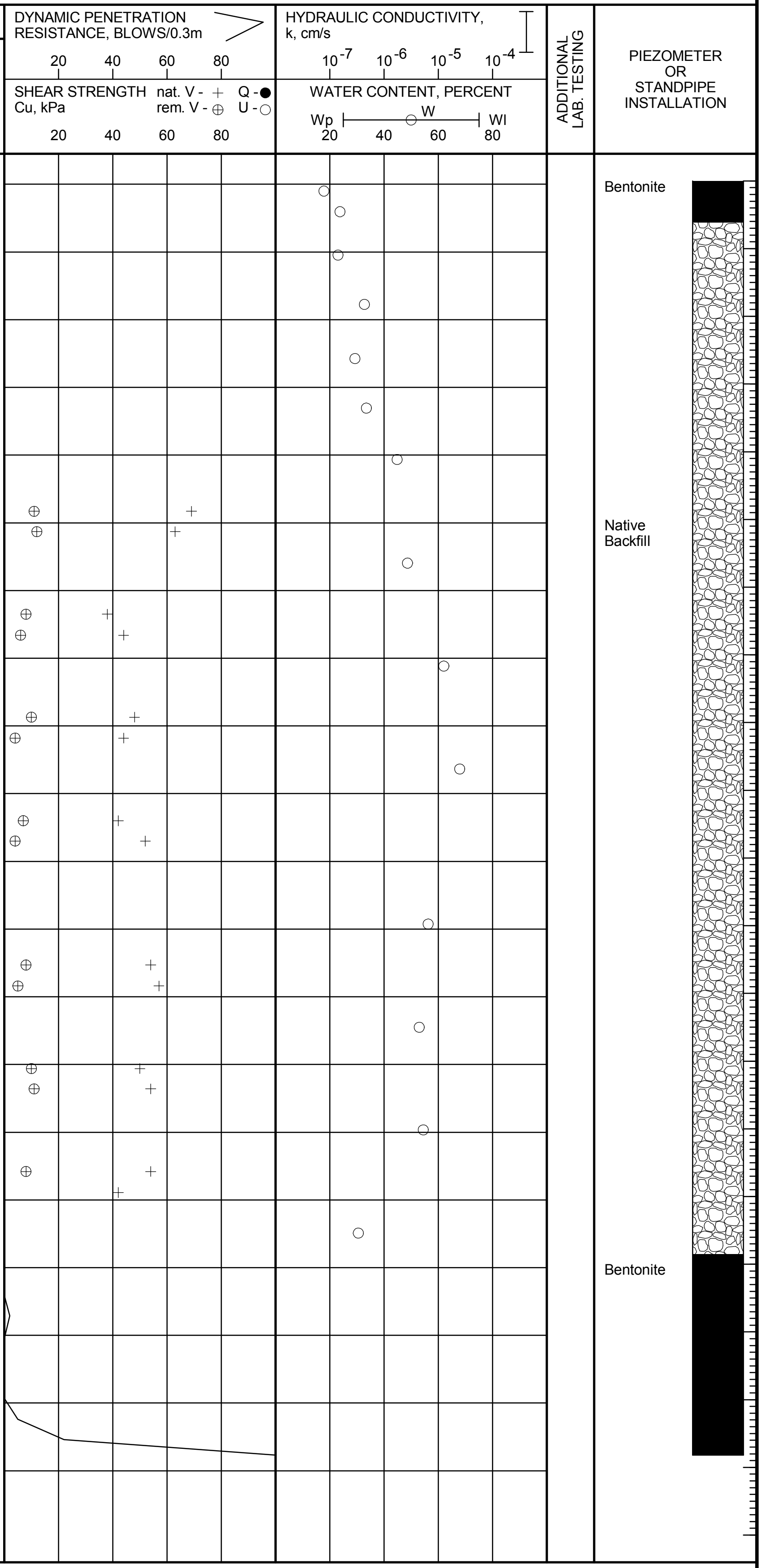
DATUM: Geodetic

BORING DATE: October 7, 2013

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
	Ground Surface	84.05
Power Auger 200mm Diameter Hollow Stem Auger	Very stiff, grey brown silty clay, trace sand, some gravel, trace organic material (FILL MATERIAL)	81.69 2.36
	Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)	79.63 4.42
	Stiff to firm, grey SILTY CLAY	68.81 15.24
	Compact, grey sandy silt, some clay and gravel (Possible GLACIAL TILL)	68.20 15.85
Dynamic Cone Penetration Test 50 mm Diameter	Dynamic Cone Penetration Test	65.23 18.82
	Borehole Terminated due to DCPT refusal on inferred bedrock	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	15
2	50 D.O.	15
3	50 D.O.	8
4	50 D.O.	11
5	50 D.O.	7
6	50 D.O.	3
7	50 D.O.	WH
8	50 D.O.	WH
9	50 D.O.	WH
10	50 D.O.	MP
11	50 D.O.	WH
12	50 D.O.	WH
13	50 D.O.	17



DEPTH SCALE

1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

BOREHOLE_RECORD WITH LAB WC 13-339 BOREHOLE LOGS OCTOBER 9 2013.GPJ HCE DATA TEMPLATE.GDT 10/29/13

PROJECT: 13-339
LOCATION: See Site Plan, Figure 2
BORING DATE: October 7, 2013

RECORD OF BOREHOLE 13-4

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

BORING METHOD: Power Auger, 200mm Diameter Hollow Stem Auger

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	83.28
Dark brown silty clay, some sand, some organic material, some gravel (FILL)	83.13 0.15
Very stiff, grey brown silty clay, some sand, trace gravel (FILL)	81.48 1.80
Very stiff to stiff, grey brown SILTY CLAY, trace sand (Weathered Crust)	79.62 3.66
End of borehole	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	11
2	50 D.O.	7
3	50 D.O.	10
4	50 D.O.	10
5	50 D.O.	6

PIEZOMETER OR STANDPIPE INSTALLATION: Native Backfill

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8

BOREHOLE_RECORD WITH LAB WC 13-339 BOREHOLE LOGS OCTOBER 9 2013.GPJ HCE DATA TEMPLATE.GDT 10/29/13

DEPTH SCALE
1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: A.N.
CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-5

SHEET 1 OF 1

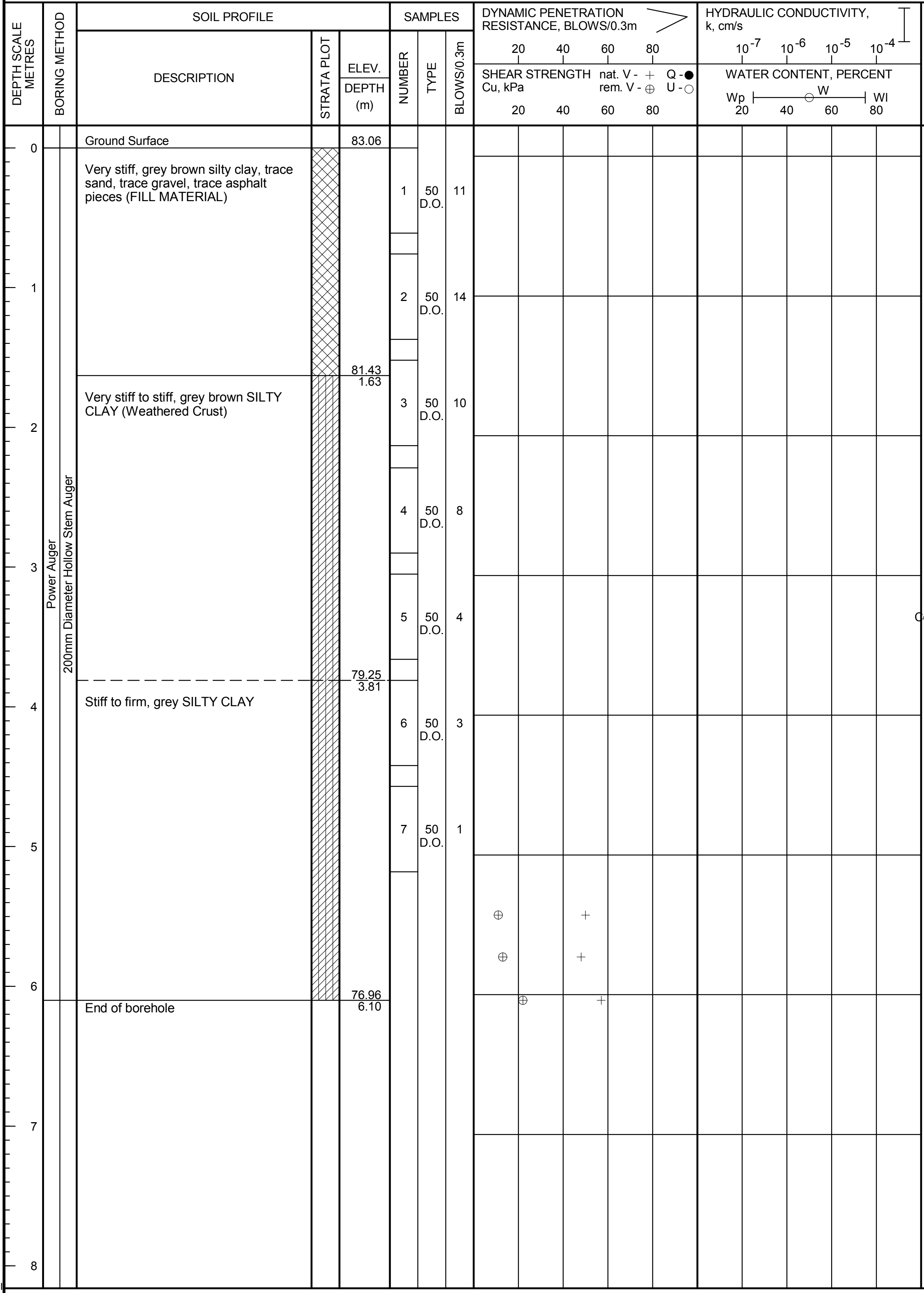
LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

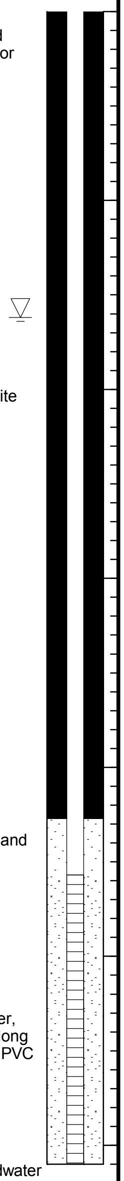
BORING DATE: October 8, 2013

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Power Auger 200mm Diameter Hollow Stem Auger	Ground Surface	83.06				
	Very stiff, grey brown silty clay, trace sand, trace gravel, trace asphalt pieces (FILL MATERIAL)		1	50 D.O.	11	
			2	50 D.O.	14	
	Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)	81.43 1.63	3	50 D.O.	10	
			4	50 D.O.	8	
			5	50 D.O.	4	Corrosion
	Stiff to firm, grey SILTY CLAY	79.25 3.81	6	50 D.O.	3	
			7	50 D.O.	1	
	End of borehole	76.96 6.10				



PIEZOMETER OR STANDPIPE INSTALLATION



Above Ground Protector

Bentonite

Filter Sand

51mm diameter, 1.52m long slotted PVC pipe

Groundwater level at 1.62 metres below ground surface (elevation 81.44 metres geodetic datum) on October 16, 2013.

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-6

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 8, 2013

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 200mm Diameter Hollow Stem Auger	Ground Surface		83.25							
		Very stiff, grey brown silty clay, trace to some sand, trace gravel, trace asphalt pieces (FILL MATERIAL)			1	50 D.O.	10				Native Backfill
1					2	50 D.O.	17				
2		Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)		81.47 1.78	3	50 D.O.	18				
					4	50 D.O.	9				
3					5	50 D.O.	3				
4		Stiff to firm, grey SILTY CLAY		79.40 3.85	6	50 D.O.	5				Bentonite
		End of borehole		78.79 4.46							
5											
6											
7											
8											

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-7

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 8, 2013

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

ADDITIONAL LAB. TESTING

BORING METHOD: Continues Split Spoon 50 mm Diameter

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	83.45				
	Very stiff, grey brown silty clay, trace sand, trace gravel, trace asphaltic concrete pieces with fine grained sand pockets (FILL MATERIAL)		1	50 D.O.	12	Native Backfill
1		82.23 1.22	2	50 D.O.	13	
	Dark brown silty clay, some organic material (Former TOPSOIL)	81.82 1.63	3	50 D.O.	11	
	Very stiff, grey brown SILTY CLAY (Weathered Crust)	81.62 1.83				
2	End of borehole					
3						
4						
5						
6						
7						
8						

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-8

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

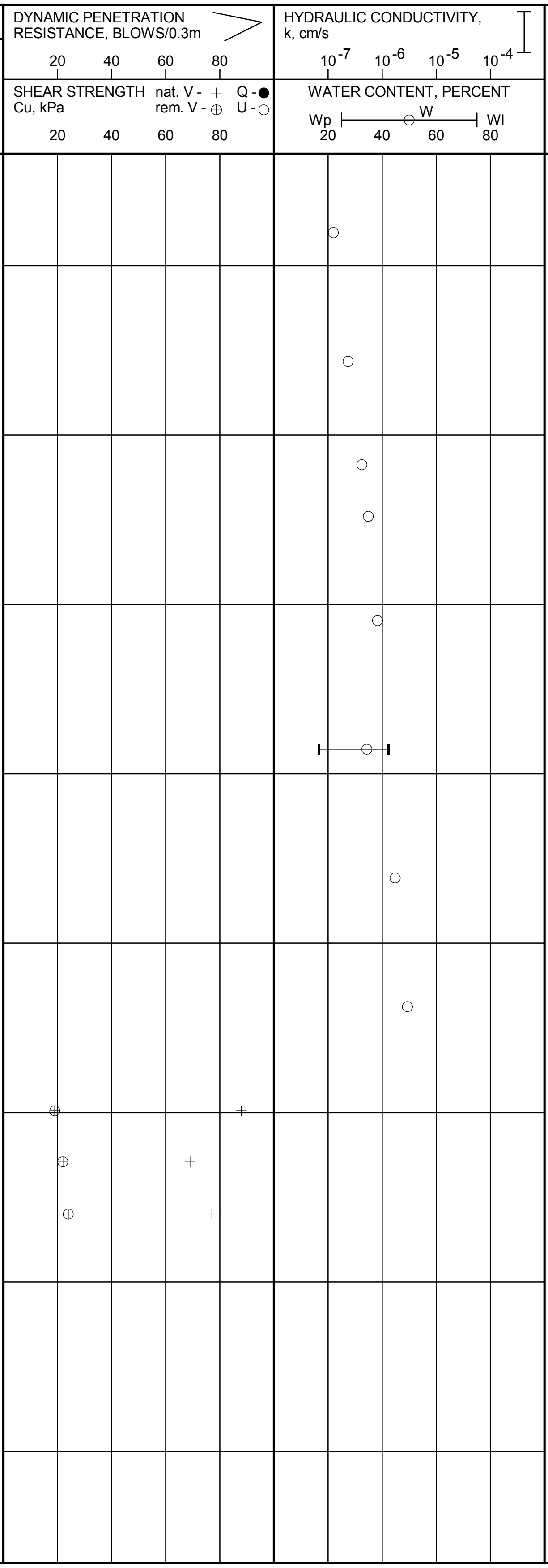
BORING DATE: October 8, 2013

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger
200mm Diameter Hollow Stem Auger

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	83.50
	Very stiff, grey brown silty clay, trace to some sand, trace gravel (FILL MATERIAL)	81.52
2	Dark brown silty clay, trace gravel, some organic material (Former TOPSOIL)	81.39 2.11
	Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)	79.38 4.12
4	Stiff, grey SILTY CLAY	77.40 6.10
6	End of borehole	

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 D.O.	13	
2	50 D.O.	7	
3	50 D.O.	7	
4	50 D.O.	14	
5	50 D.O.	9	M
6	50 D.O.	4	
7	50 D.O.	2	M



PIEZOMETER OR STANDPIPE INSTALLATION

Native Backfill

Bentonite

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

PROJECT: 13-339

RECORD OF BOREHOLE 13-9

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: October 8, 2013

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES

BORING METHOD: Power Auger; 200mm Diameter Hollow Stem Auger

SOIL PROFILE

DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	82.89
Very stiff, grey brown silty clay, trace sand, trace gravel, trace roots (FILL MATERIAL)	
Very stiff to stiff, grey brown SILTY CLAY (Weathered Crust)	81.34 1.55
Stiff to firm, grey SILTY CLAY	79.08 3.81
End of borehole	78.32 4.57

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 D.O.	9	
2	50 D.O.	8	
3	50 D.O.	13	
4	50 D.O.	10	Corrosion
5	50 D.O.	4	
6	50 D.O.	4	

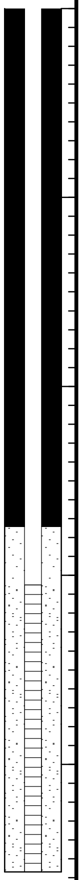
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

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

PIEZOMETER OR STANDPIPE INSTALLATION



Above Ground Protector

Bentonite

Filter Sand

51mm diameter, 1.52m long slotted PVC pipe

Groundwater level at 1.22 metres below ground surface (elevation 81.67 metres geodetic datum) on October 16, 2013.

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: L.A.

CHECKED:

APPENDIX B

CHEMICAL ANALYSIS OF SOIL SAMPLE RELATING TO CORROSION (PARACEL LABORATORIES LTD. ORDER No. 1342265)

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e: paracel@paracellabs.com
www.paracellabs.com

Certificate of Analysis

Houle Chevrier
180 Wescar Lane
Ottawa, ON K0A1L0
Attn: Lauren Ashe

Phone: (613) 836-1422
Fax: (613) 836-9731

Client PO:
Project: 13-339
Custody: 12255

Report Date: 23-Oct-2013
Order Date: 17-Oct-2013
Order #: 1342265

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	**Client ID**
1342265-01	BH13-5 SA5
1342265-02	BH13-9 SA4

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 23-Oct-2013
Order Date:17-Oct-2013

Client: **Houle Chevrier**
Client PO:
Project Description: 13-339

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	22-Oct-13	22-Oct-13
pH	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	22-Oct-13	22-Oct-13
Resistivity	EPA 120.1 - probe, water extraction	22-Oct-13	22-Oct-13
Solids, %	Gravimetric, calculation	21-Oct-13	21-Oct-13

Certificate of Analysis

Client: **Houle Chevrier**
Client PO:

Report Date: 23-Oct-2013
Order Date:17-Oct-2013

Project Description: 13-339

	Client ID:	BH13-5 SA5	BH13-9 SA4	-	-
	Sample Date:	16-Oct-13	16-Oct-13	-	-
	Sample ID:	1342265-01	1342265-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.7	75.3	-	-
General Inorganics					
pH	0.05 pH Units	7.50	7.32	-	-
Resistivity	0.10 Ohm.m	35.4	30.9	-	-
Anions					
Chloride	5 ug/g dry	10	48	-	-
Sulphate	5 ug/g dry	66	111	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis

Report Date: 23-Oct-2013
Client: **Houle Chevrier**
Order Date:17-Oct-2013
Client PO:
Project Description: 13-339

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	226	5	ug/g dry	217			3.9	20	
Sulphate	7.43	5	ug/g dry	7.07			5.0	20	
General Inorganics									
pH	7.50	0.05	pH Units	7.50			0.0	10	
Resistivity	54.8	0.10	Ohm.m	55.4			1.1	20	
Physical Characteristics									
% Solids	72.2	0.1	% by Wt.	70.7			2.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	31.4		mg/L	21.7	96.9	78-113			
Sulphate	10.8		mg/L	0.71	101	78-111			

Qualifier Notes:

Login Qualifiers :

Sample not received in Paracel verified container / media

Applies to samples: BH13-5 SA5, BH13-9 SA4

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.