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May 24, 2013 Our ref: 13-106

Apex Developments Inc.
900 Morrison Drive
Ottawa, ON
K2H 8K7

Attention: Mr. Michael Assal

RE: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
HAWTHORNE BUSINESS PARK (LOT H)
OTTAWA, ONTARIO

Dear Sir:

Enclosed are three (3) copies of the geotechnical report for the proposed commercial development located at Lot H on Sappers Ridge in the Hawthorne Business Park in Ottawa, Ontario.

We trust that this report provides sufficient information for your current purposes. If you have any questions concerning the report, please call.

Yours truly,

HOULE CHEVRIER ENGINEERING LTD.

Serge Bourque, M.Sc.E., P.Eng.
Senior Geotechnical Engineer

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REPORT ON

# GEOTECHNICAL INVESTIGATION
# PROPOSED COMMERCIAL DEVELOPMENT
# HAWTHORNE BUSINESS PARK (LOT H)
# OTTAWA, ONTARIO

Submitted to:

Apex Developments Inc.
900 Morrison Drive
Ottawa, Ontario
K2H 8K7

DISTRIBUTION:

3 bound copies – Apex Developments Inc.
1 electronic copy – Apex Developments Inc.
1 bound copy – Houle Chevrier Engineering Ltd.

May 2013

Our ref: 13-106

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## 1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed commercial development located at Lot H on Sappers Ridge in the Hawthorne Business Park in Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of test pits and boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the proposed development, including construction considerations that could influence design decisions.

This investigation was carried out in accordance with our proposal dated April 10, 2013 and technical memorandum dated May 7, 2013.

## 2.0 PROJECT AND SITE DESCRIPTION

### 2.1 Project Description

Plans are being prepared to develop the vacant parcel of land located at Lot H on Sappers Ridge in the Hawthorne Business Park (see Key Plan, Figure 1). Based on the information provided to us, it is understood that the proposed development plans include three (3) slab-on-grade buildings with associated parking and driveway areas. In addition, a septic system will be constructed on the west side of the site, adjacent to Sappers Ridge.

### 2.2 Review of Site Conditions and Geology

The site of the proposed commercial development is located on Sappers Ridge (Lot H), which is off of Somme Street. The site grades are generally flat and covered with sand and gravel fill. Surface water ponding was noted at localized areas across the site. In addition, an existing concrete septic tank protruding above the ground surface was noted near test pit 13-104.

The CRA Phase II ESA report provided to us indicates that the soil conditions south of the site consist of about 2 to 2.5 metres of miscellaneous fill underlain by silty sand till, which in turn is underlain by inferred bedrock at 3 to 3.5 metres below surface grade. The results of a monitoring well (MW 08-8) installed north of the site show the fill is about 4.5 metres thick and that the groundwater table is 2.8 metres below surface grade.

A review of historical aerial photographs shows that the site was originally a flat marshy area with large areas of standing water (1976 photo), which was subsequently infilled between 1999 and 2011.

Surficial geology maps indicate that the site is likely underlain by nearshore marine sediments likely consisting of reworked glaciofluvial sand. Bedrock geology and drift thickness maps indicate that the bedrock consists of sandstone of the Nepean formation at depths of between 0 to 1 metres below ground surface.

## 3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on May 1, 2013 and May 10, 2013. During that time, a total of eight (8) test pits and six (6) boreholes were put down at the site.

The test pits were excavated using a track mounted excavator on May 1, 2013. The excavator used was a track mounted CAT 304 supplied and operated by KingEx Landscaping and Excavating of Kemptville, Ontario. The subsurface conditions in the test pits were identified by visual and tactile examination of the materials exposed on the sides and bottom of the test pits. The groundwater conditions in the open test pits were observed on completion of excavating.

Two (2) test pits were excavated at each of the three (3) proposed building areas, test pits 13-101 to 13-106, inclusive. Two (2) test pits were excavated in the area of the proposed septic field, test pit 13-201 and 13-202.

As a result of the thick fill layer encountered within the test pits, six (6) boreholes were advanced at the site on May 10, 2013 using a track mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-Rouge, Quebec. The boreholes were put down adjacent to the test pit locations within the building areas and are numbered 13-1 to 13-6, inclusive.

Standard penetration tests were carried out in some of the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. The dynamic cone penetrometer was advanced to refusal at all boreholes to inferred bedrock, with exception of borehole 13-6. Bedrock was cored at borehole 13-3.

The field work was supervised throughout by a member of our engineering staff who directed the test pit and drilling operations, and logged the samples. 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.

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.

Descriptions of the subsurface conditions logged in the test pits and boreholes are provided in Appendix A following the text of this report.

The approximate locations of the test holes are shown on the Site Plan, Figure 2.

## 4.0 SUBSURFACE CONDITIONS

### 4.1 General

The soil and groundwater conditions logged in the test pits and boreholes are outlined on the Record of Test Pit and Borehole logs in Appendix A. The test pit and borehole logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at other than the test hole locations may vary from the conditions encountered in the test pits. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

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 judgment and Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

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

### 4.2 Fill

The thickness of the fill layer was confirmed at one (1) of the eight (8) test pit locations (test pit 13-201) and at four (4) of the six (6) borehole locations. Where confirmed, the thickness of the fill layer ranges from about 3.1 to 4.0 metres. The fill is composed of varying amounts of clay, silt, sand and gravel mixed with debris generally consisting of concrete, brick, asphalt and organics. Refer to the attached test hole logs for more detailed descriptions of the fill at specific locations.

Standard penetration tests carried out in the fill gave N values of 3 to 24 blows per 0.3 metres of penetration. Therefore, the relative density of the fill is highly variable and ranges from very loose to compact.

### 4.3 Former Topsoil

The former topsoil/peat layer was encountered below the fill layer at two (2) borehole locations (BH 13-1 and 13-3). The thickness of this layer is 0.6 and 0.5 metres at boreholes 13-1 and 13-3, respectively.

### 4.4 Silty Clay

Silty clay was noted below the fill and/or former topsoil/peat layers at three (3) of the six (6) borehole locations and at one (1) of the eight (8) test pit locations. The thickness of the silty clay was measured at to be about 0.2 and 0.8 metres were it was fully penetrated at boreholes 13-6 and 13-1, respectively.

The results of a moisture content and Atterberg limits test undertaken on a representative sample of the silty clay at borehole 13-1 (Sample 3) shows that the material has a natural moisture content of about 41%, liquid limit of 32%, plastic limit of 22% and a plasticity index of 10. Detailed laboratory test results are appended (Appendix B).

### 4.5 Sand

A layer of sand ranging in composition from silty sand to sand with trace to some silt was encountered below the fill, silty clay and/or former topsoil layer at three (3) of the six (6) borehole locations. It is assumed that this sand layer would be present at the remaining boreholes, where dynamic cone penetration tests were advanced in lieu of standard penetration tests with samples.

Standard penetration tests carried out in the sand layer gave N values of 1 to 7 blows per 0.3 metres of penetration. Therefore, the relative density of this layer is very loose to loose. These lower N values are likely attributed to disturbance from groundwater flow into the hollow stem augers.

Dynamic cone penetration tests advanced through the sand layer generally gave values of 10 to 30 blows per 0.3 metres of penetration, indicating a compact relative density. In our opinion, these values are more representative of the actual compactness of the soil

Three (3) laboratory moisture content and grain size analysis tests were undertaken on samples of the sand layer. These results show that the natural moisture content ranges from 14 to 24%.

The results of the grain size distribution curve show that the materials are primarily composed of sand with 4 to 14% clay and silt sized particles. Detailed laboratory results are appended (Appendix B).

### 4.6 Bedrock

Sandstone bedrock was encountered and cored in borehole 13-3 using rotary diamond drilling techniques. The total core recovery (TCR) was 79% for rock core sample 4 and 100% for rock core sample 5. The solid core recovery (SCR) was 78% and 100% for rock core samples 4 and 5, respectively. The rock quality designation (RQD) was 78% and 100% for rock cores 4 and 5, respectively, indicating that the bedrock quality is good to excellent.

### 4.7 Inferred Bedrock

Refusal of the split tube sampler or dynamic cone penetration test occurred at 6.9 to 9.8 metres below ground surface (elevation 82.7 to 86.1 metres, geodetic datum). It should be noted that practical refusal can sometimes occur within cobbles and boulders and may not necessarily be representative of the upper surface of the bedrock.

### 4.8 Groundwater Conditions

The groundwater seepage/inflow was noted on the test pit side walls at seven (7) of the eight (8) test pit locations at depths ranging from 0.9 to 1.8 metres (averaging 1.4 metres) below existing surface grade (elevation 90.7 to 92.0 metres, geodetic datum). It should be noted that groundwater levels will fluctuate seasonally and may be higher during wet periods of the year, such as the early spring or fall, or following periods of heavy precipitation.

# 5.0 GEOTECHNICAL DESIGN GUIDELINES

## 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.

The test pits were loosely backfilled with the excavated material and tamped with the bucket of the excavator. As such, the test pit locations represent areas of disturbance and should be monitored during proof rolling if encountered. If necessary some subexcavation and re-compaction of the backfill material may be required at these locations, to prevent excessive future settlement.

## 5.2 Proposed Building Foundations

### 5.2.1 General

It is our opinion that the site is not currently suitable for the three (3) proposed structures founded on conventional spread footings with a slab-on-grade due to the 3 to 4 metre thick layer of miscellaneous loose fill soils encountered across the site. Foundations and slabs on grade constructed above the existing fill materials would likely experience total and differential settlements that would result in severe cracking and distortion.

In our opinion the most suitable foundation option for this site is a deep foundation consisting of concrete filled steel pipe piles or steel H piles driven to bedrock with a structural slab. We have considered the option of removing the unsuitable soils and building the site up using imported OPSS Granular B Type II fill, but this alternative poses significant challenges, which include:

- Extensive dewatering, which will require a permit to take water; and
- Sensitive silt/sand subgrade soils which could liquefy during excavation and backfilling operations.

Other ground improvement options such as rapid impact compaction to densify the overburden soils or rammed aggregate piers (i.e. Geopiers) are not feasible alternatives for this site due to the high variability of the fill material (i.e. zones with concrete, asphalt, brick and organic debris), high groundwater, organic layers encountered and sensitive silt/sand subgrade soils.

The recommendations outlined in this report will focus on a driven pile deep foundation option only. If alternative proprietary foundation options are considered, a geotechnical engineer should review the final design to ensure that the geotechnical report has been interpreted correctly.

**5.2.2 Pile Foundation**

As noted above, it is our opinion that a pile foundation is the most suitable foundation option for the proposed buildings. We provide the following design recommendations:

- We recommend driven steel H or concrete filled pipe piles finalized on bedrock for the deep foundations. The pile cross section should be confirmed once foundation structural loads are known. When the pile design is complete, Houle Chevrier Engineering would be willing to provide additional recommendations.

- The contractor should be required to submit the pile design and pile driving criteria for review prior to pile driving at this site. An allowance should be made in the specifications for retapping all of the piles at least once after a minimum period of two days to confirm the permanence of the pile set.

- As a design example, the ultimate unfactored geotechnical resistance at ultimate limits states (ULS) in axial compression on a HP310X110 steel pile could be taken as 2,600 kilonewtons. This assumes that 350 megapascal steel is being used. A resistance factor of 0.4 should be applied to ULS resistance. If dynamic pile testing is undertaken the resistance factor may be increased from 0.4 to 0.5. The ULS resistance will govern the design since the stresses required to induce SLS criteria for piles terminated on bedrock will exceed those at ULS. Therefore, the SLS resistance has not been presented in this report.

- As a second example, the factored ULS structural resistance of a 245 millimetre diameter steel pipe pile with a wall thickness of 12 millimetres driven closed ended and filled with concrete may be taken as 1,900 kilonewtons.

- The refusal criteria will be highly dependent on the contractor's pile driving equipment. Typically, for the drop hammer type piling rigs available in Eastern Ontario, a refusal criteria of 10 blows for the last 25 millimetres of penetration would be sufficient to achieve the above loads, assuming that a hammer with a rated energy of about1,650 ft-lbs/in$^2$ or 350 J/cm$^2$. The actual hammer energy required to finalize the piles may vary depending on soil / bedrock conditions at each location and efficiency of the pile driving system.

- Driving criteria should be established using a Wave Equation Analysis, once the hammer details are established.

- A minimum nominal corrosion of 1/16" (1.6 mm) should be applied to the pile cross sectional area of the piles.

- According to the Canadian Foundation Engineering Manual an appropriate resistance factor of 0.4 may be applied to the ultimate geotechnical resistance in axial compression. For dynamically tested piles, the resistance factor for axial compression may be increased from 0.4 to 0.5.

- In order to increase the resistance factor from 0.4 to 0.5, dynamic pile testing should be undertaken on a minimum of 10 percent of the driven piles. We recommend that 24 hour restrike testing be carried out on PDA tested piles to evaluate pile relaxation. Piles that relax should be driven and tested again 24 hours later.

- Houle Chevrier Engineering provides dynamic pile testing services using our pile driving analyzer (PDA) and would be pleased to provide this service upon request.

- Full time inspection of pile driving by qualified geotechnical personnel is recommended.

### 5.2.3 Excavation

The excavation for the pile foundation caps will be carried out mostly through miscellaneous fill material. The sides of the excavation 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 carried out with side slopes of 1 horizontal to 1 vertical, or flatter.

In general, the groundwater inflow from the overburden deposits could be controlled by pumping from sumps within the excavation. Significant groundwater inflow would be expected if excavation depths extend below the groundwater (i.e. 0.9 to 1.8 metres below surface grade). If groundwater pumping exceeds 50,000 litres per day (or 8 gallons per minute on a 24 hours basis) a Permit to Take Water will be required in advance of the construction. Based on our experience, it takes at least 3 months to obtain a permit from the time of application.

No unusual problems are anticipated in excavating the fill above the groundwater level. In contrast, excavation of the fill soils below the groundwater level could cause sloughing of the soil into the excavation. The excavation side slopes could be made stable by using flatter side slopes (say at 3 horizontal to 1 vertical), by placing a 0.3 to 0.5 metre thick drainage layer of sand and gravel meeting OPSS Granular B Type II on the soil below the groundwater level, or a combination of these.

### 5.2.4 Frost Protection Requirements for Foundations

All exterior pile caps and grade beams 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.

### 5.2.5 Seismic Design of Proposed Structures

#### 5.2.5.1 Seismic Site Classification

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.

#### 5.2.5.2 Potential for Liquefaction

Dynamic cone penetration tests advanced through the sand layer generally gave values of 10 to 30 blows per 0.3 metres of penetration, indicating a compact relative density. Therefore, in our opinion, there is no potential for liquefaction of the overburden soils at this site.

### 5.2.6 Pile Cap and Grade Beam Backfill and Drainage

Generally, the fill material is frost susceptible and should not be used as backfill against pile caps and grade beams. To avoid frost adhesion and possible heaving, the pile caps and grade beams should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting OPSS Granular B Type I or II 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. 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 building, the existing fill materials should be removed to a depth of at least 1.5 metres and replaced with suitable compacted granular materials such as OPSS Granular B Type II. 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 to reduce the effects of differential frost heaving. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter. Notwithstanding the removal of some of the existing fill material, it should be expected that ongoing settlement of the existing fill materials will occur and this could result in settlement and cracking of concrete sidewalks and curbs, etc. As an alternative to concrete sidewalks, interlocking paving stones could be considered.

Perimeter foundation drainage is not considered necessary for structural slab structures at this site, provided that the floor slab levels are above the finished exterior ground surface level and above the groundwater table.

#### 5.2.7 Structural Slab

The grade within the proposed structural slab area could be raised, where necessary, with a material which meets OPSS specifications for Granular B Type I or II. The grade raise fill material below the proposed structural slab will require minimal compaction since it will ultimately be supported by the piles. In order to provide a stable working platform, the material should be compacted in maximum 300 millimetre thick lifts and compacted to at least 90 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

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

If any areas of the buildings are to remain unheated during the winter period, thermal protection of the materials beneath the structural slab may be required. Where engineered fill is used beneath the structural slab, the required depth of frost protection may be reduced by the thickness of the engineered fill. Further details on the insulation requirements could be provided, if necessary. Additional frost protection may also be required in heated or unheated buildings near large overhead doors which are frequently opened for loading/unloading trucks, etc. Further details could be provided, if necessary.

### 5.3 Access Roadways and Parking Areas for the Proposed Development

#### 5.3.1 Subgrade Preparation

In preparation for the construction of the access roadways and parking areas, any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. This need not include the removal of the all the fill material or former topsoil layer provided some minor post construction settlement of the flexible (asphaltic concrete) pavement can be accommodated. Furthermore, allowance should be made to pad the asphaltic concrete as necessary. It should be expected that some differential settlement of the pavement structure will occur over time.

Any subexcavated areas could be filled with compacted earth borrow that is frost compatible with the surrounding fill material. Similarly, should it be necessary to raise the parking lot grades at this site, the grade raise fill for the parking areas could consist of material which meets OPSS specifications for Granular B Type I, Select Subgrade Material, or suitable earth borrow. Any Granular B Type I, Select Subgrade Material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. It is noted, however, that some of the fill material contains higher amounts of silt 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. Depending on the weather conditions, it may be necessary to allow the material to dry prior to compaction.

The subgrade surfaces should be proof rolled with an 8 tonne or larger steel drum roller and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable earth borrow that is frost compatible with the surrounding soils.

### 5.3.2 Pavement Design

#### 5.3.2.1 Asphaltic Concrete Surfaced Areas

It is suggested that 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
- Approved subgrade

For any asphaltic concrete surfaced areas which will be used by frequent truck traffic or fire trucks, the asphaltic concrete surfacing thickness should be increased to 80 millimetres and the thickness of the subbase layer increased to 450 millimetres.

The asphaltic concrete should consist of 50 millimetres of Superpave 12.5 or HL3. For any access roadways that will be used by frequent truck traffic or fire trucks, the asphaltic concrete surfacing thickness should be increased to 80 millimetres (40 millimetres of Superpave 12.5 or HL3 over 40 millimetres of Superpave 19.0 or HL8). The superpave asphaltic concrete mixes should be designed for Traffic Level A or B. Performance grade PG 58-34 asphaltic concrete should be specified for either Superpave or Marshall mixes.
The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

The use of a woven geotextile such as OPSS Class II between the subbase and subgrade may be considered if construction takes place during the wet spring or fall months.

#### 5.3.2.2 Gravel Surfaced Areas

It is suggested that gravel surfaced areas to be used by light vehicles (cars, etc.) be constructed using the following minimum granular thicknesses:

- 150 millimetres of OPSS Granular A base, over
- 375 millimetres of OPSS Granular B Type II subbase

For any gravel surfaced areas which will be used by truck traffic or fire trucks, the thickness of the subbase layer should be increased to 450 millimetres.

The granular thicknesses given above assume that the parking areas are constructed on sand, sand and gravel, or glacial till and that the subgrade surfaces are prepared as described in this report. If the subgrade surface is composed of sandy silt or silty sand the thickness of the subbase should be increased.

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

The use of a woven geotextile such as OPSS Class II between the subbase and subgrade may be considered if construction takes place during the wet spring or fall months.

### 5.3.3 Granular Material Placement

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

### 5.3.4 Transition Treatments

In areas where the new pavement structure will abut existing pavement structures, 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.

### 5.3.5 Drainage

The subgrade surface should be shaped and crowned to promote drainage of the granular base and subbase materials.

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. If surface drainage is used, swales or ditches are suggested around the paved areas. The granular base and subbase materials should extend horizontally to the ditches/swales. Where possible, the bottom of the swales/ditches should be at least about 0.6 metres below the bottom of the Granular B Type II. If catch basins are used, filter wrapped, perforated subdrains should be installed at the catch basins within the parking areas. The catch basins should be provided with 3 metre (minimum) long perforated stub drains which extend in at least two directions from the catch basins at the pavement subgrade level

The need for additional subdrains within the granular material should be assessed by us as part of the design.

### 5.3.6 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.

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 and/or incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material.

### 5.4 Septic System

#### 5.4.1 Excavation

It is understood that a septic system is being proposed at the west end of the site, adjacent to Sappers Ridge. Based on the test pit information collected at TP 13-201 and 13-202 excavation of the septic system will be carried out through miscellaneous fill soils.

During construction, the sides of the excavation 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 carried out with side slopes of 1 horizontal to 1 vertical, or flatter.

In general, the groundwater inflow from the overburden deposits could be controlled by pumping from sumps within the excavation. Significant groundwater inflow would be expected if excavation depths extend below the groundwater (i.e. 0.9 to 1.8 metres below surface grade). If groundwater pumping exceeds 50,000 litres per day (or 8 gallons per minute on a 24 hours basis) a Permit to Take Water will be required in advance of the construction. Based on our experience, it takes at least 3 months to obtain a permit from the time of application.

#### 5.4.2 Infiltration Characteristics

The proposed septic area is covered with about 3 to 4 metres of miscellaneous fill. The composition of the fill is highly variable throughout the site; therefore the percolation rate and the compressibility of the fill cannot be accurately determined. As such, it is recommended that the fill be removed from the beneath the proposed septic system area, and the grade raised to the underside of the septic system level using suitable, compacted materials such as clean sand or earth borrow. The native silt/sand soils below the fill are located within the groundwater table, thus would not be relevant for the septic system design.

We recommend that the septic system be designed using imported fill. The infiltration rate of the proposed imported soil could be determined by Houle Chevrier Engineering once a material source is selected.

### 5.5 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, 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. The magnitude of the vibrations will be much less than that required to cause damage to the nearby structures.

### 5.6 Winter Construction

In the event that construction is required during freezing temperatures, the soil subgrade below the footings and slabs should be protected immediately from freezing using straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means.

### 5.7 Design Review and Construction Observation

The details for the proposed development 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 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 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. Full time pile driving inspection should be undertaken by a qualified geotechnical technician to ensure that piles are finalized as per the geotechnical engineers driving criteria.

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.

LICENSED PROFESSIONAL ENGINEER
S. BOURQUE
100194197
24 May 2013
PROVINCE OF ONTARIO

Serge Bourque, M.Sc.E., P.Eng.
Senior Geotechnical Engineer

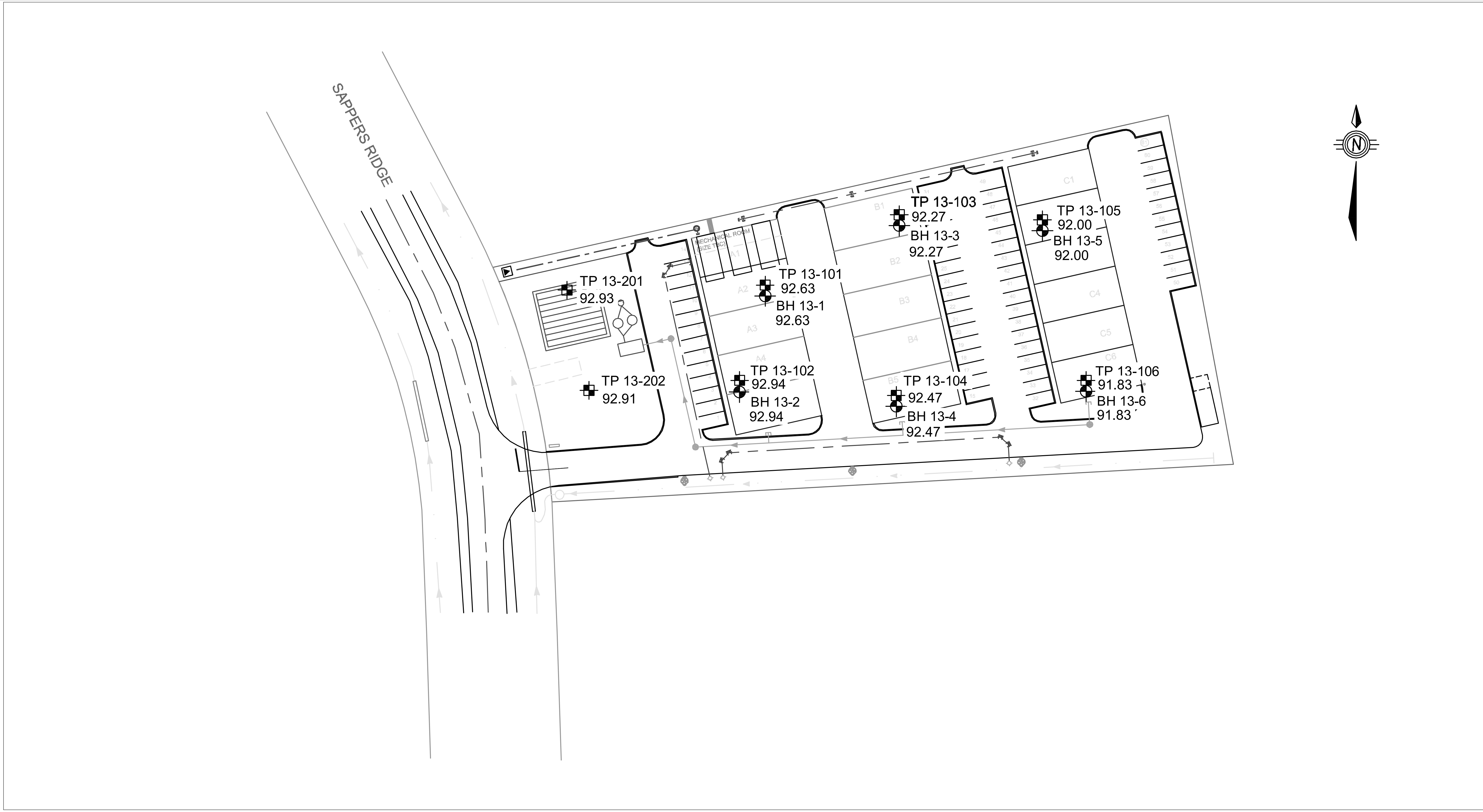
| KEY PLAN | FIGURE 1 |
|---|---|


N.T.S

Houle Chevrier Engineering

Date: May 2013

Project: 13-106




LEGEND

| | |
|---|---|
| TP 13-101 92.63 | APPROXIMATE TEST PIT LOCATION AND GROUND SURFACE ELEVATION (METRES, GEODETIC DATUM) IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD. |
| BH 13-1 92.63 | APPROXIMATE BOREHOLE LOCATION AND GROUND SURFACE ELEVATION (METRES, GEODETIC DATUM) IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD. |

| | | | |
|---|---|---|---|
| Client: APEX DEVELOPMENTS INC. | | Location: LOT H HAWTHORNE BUSINESS PARK OTTAWA, ON | Revision: 0 |
| Drawn by: B.T. | Approved by: S.B. | Project No.: 13-106 | Scale: 1 : 750 |
| Houle Chevrier Engineering | | Title: SITE PLAN | |
| | | Date: May 2013 | FIGURE 2 |

APPENDIX A

LIST OF ABBREVIATIONS AND TERMINOLOGY
RECORD OF TEST PIT AND BOREHOLE LOGS

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

Dynamic Penetration Resistance

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

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

## SOIL TESTS

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

## SOIL DESCRIPTIONS

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

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

## LIST OF COMMON SYMBOLS

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

# RECORD OF TEST PIT 13-101

PROJECT: 13-106
LOCATION: Lot H - Hawthorne Business Park
DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1
DATUM: Geodetic
TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.63 | | | | | |
| | Brown silty sand, some clay, gravel and brick (FILL MATERIAL) | | | 1 | | | | Backfilled with excavated materials |
| | Brown silty sand with ashpalt and large pieces of concrete (FILL MATERIAL) | | 91.87<br>0.76 | | | | | |
| 1 | | | | | | | | |
| 2 | Grey sand and gravel, some clay with ashpalt and brick (FILL MATERIAL) | | 90.65<br>1.98 | | | | | |
| 3 | | | | 2 | | | | |
| | End of test pit | | 88.97<br>3.66 | | | | | Groundwater seepage observed at 1.5 metres below ground surface at time of excavation |
| 4 | | | | | | | | |
| 5 | | | | | | | | |

DEPTH SCALE
1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L
CHECKED:

# RECORD OF TEST PIT 13-102

PROJECT: 13-106

LOCATION: Lot H - Hawthorne Business Park

DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1

DATUM: Geodetic

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | DESCRIPTION | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp — W — Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | 92.94 | | | | | |
| | Brown SILTY SAND and GRAVEL (FILL MATERIAL) | | 1 | | | | Backfilled with excavated materials |
| | Dark grey black SAND and GRAVEL with ashpalt pieces (FILL MATERIAL) | 92.43<br>0.51 | 2 | | | | |
| 1 | Grey sand and gravel with ashpalt pieces, clay, wood and peat (FILL MATERIAL) | 91.87<br>1.07 | | | | | ▽ |
| 2 | | | | | | | |
| 3 | | | 3 | | | | |
| | End of test pit | 89.28<br>3.66 | | | | | Groundwater seepage observed at 1.0 metre below ground surface at time of excavation |
| 4 | | | | | | | |
| 5 | | | | | | | |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF TEST PIT 13-103

PROJECT: 13-106

LOCATION: Lot H - Hawthorne Business Park

DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1

DATUM: Geodetic

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80 | WATER CONTENT (PERCENT) Wp ├──⊖ W──┤ Wl 20 40 60 80 | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.27 | | | | | |
| | Brown SAND and GRAVEL, cobbles with ashpalt and concrete pieces (FILL MATERIAL) | | | | | | | Backfilled with excavated materials |
| 1 | | | | 1 | | | | |
| 2 | | | 90.14<br>2.13 | | | | | |
| | Dark silty clay, some sand and ashphalt (FILL MATERIAL) | | | | | | | |
| 3 | | | | 2 | | | | |
| | End of test pit | | 88.61<br>3.66 | | | | | No groundwater observed at time of excavation |
| 4 | | | | | | | | |
| 5 | | | | | | | | |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF TEST PIT 13-104

PROJECT: 13-106

LOCATION: Lot H - Hawthorne Business Park

DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1

DATUM: Geodetic

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.47 | | | | | |
| | Brown SILTY SAND and GRAVEL (FILL MATERIAL) | | | 1 | | | | Backfilled with excavated materials |
| | Asphalt, brick and gravel (FILL MATERIAL) | | 91.86 0.61 | 2 | | | | |
| | Grey silty sand and gravel, asphalt and brick with debris (FILL MATERIAL) | | 90.95 1.52 | 3 | | | | ∇ |
| | Grey silty clay, sand and gravel with asphalt and wood (FILL MATERIAL) | | 90.34 2.13 | 4 | | | | |
| | End of test pit | | 88.81 3.66 | | | | | Groundwater seepage observed at 1.8 metres below ground surface at time of excavation |
| 5 | | | | | | | | |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF TEST PIT 13-105

PROJECT: 13-106

LOCATION: Lot H - Hawthorne Business Park

DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1

DATUM: Geodetic

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.00 | | | | | |
| | Grey SANDY SILT to SILTY SAND, trace to some gravel (FILL MATERIAL) | | 91.80<br>0.20 | 1 | | | | |
| | Brown grey, silty sand and gravel, cobbles and boulders, asphalt, brick and wood (FILL MATERIAL) | | | | | | | Backfilled with excavated materials |
| 1 | | | | 2 | | | | |
| 2 | | | 89.56<br>2.44 | | | | | |
| | End of test pit | | | | | | | Groundwater seepage observed at 1.3 metres below ground surface at time of excavation |
| 3 | | | | | | | | |
| 4 | | | | | | | | |
| 5 | | | | | | | | |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF TEST PIT 13-106

PROJECT: 13-106

SHEET 1 OF 1

LOCATION: Lot H - Hawthorne Business Park

DATUM: Geodetic

DATE OF EXCAVATION: May 1, 2013

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. / DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 91.83 | | | | | |
| | Brown silty sand, trace to some gravel, brick, concrete and wood (FILL MATERIAL) | | | 1 | | | | Backfilled with excavated materials |
| 2 | Grey clay, silty sand, wood and asphalt (FILL MATERIAL) | | 89.85 / 1.98 | 2 | | | | |
| | End of test pit | | 88.17 / 3.66 | | | | | Groundwater seepage observed at 1.2 metres below ground surface at time of excavation |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

TESTPIT RECORD 2012 13-106 TEST PITS.GPJ HCE DATA TEMPLATE.GDT 5/24/13

# RECORD OF TEST PIT 13-201

PROJECT: 13-106

LOCATION: Lot H - Hawthorne Business Park

DATE OF EXCAVATION: May 1, 2013

SHEET 1 OF 1

DATUM: Geodetic

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | DESCRIPTION | ELEV. DEPTH (m) | SAMPLE NUMBER | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|
| 0 | Ground Surface | 92.93 | | |
| | Clayey TOPSOIL FILL | 92.78 0.15 | | |
| | Brown silty sand and gravel, concrete fragments, asphalt, metal, wire and styrofoam (FILL MATERIAL) | 91.23 1.70 | 1 | Backfilled with excavated materials |
| 1 | | | | |
| | Dark grey organic clay, trace asphalt (FILL MATERIAL) | 90.78 2.15 | 2 | |
| 2 | | | | |
| | Brown silty sand and gravel, cobbles and boulders, trace clay (FILL MATERIAL) | 89.88 3.05 | 3 | |
| 3 | Very stiff, dark grey SILTY CLAY, trace sand, trace asphalt | 89.58 3.35 | | |
| | Very stiff, dark grey brown SILTY CLAY, trace sand | 89.27 3.66 | | |
| | End of test pit | | | Groundwater seepage observed at 2.15 metres below ground surface at time of excavation |
| 4 | | | | |
| 5 | | | | |

SHEAR STRENGTH, Cu (kPa) — Natural. V - + Remoulded. V - ⊕ — 20 40 60 80

WATER CONTENT (PERCENT) — Wp —○ W — Wl — 20 40 60 80

ADDITIONAL LAB. TESTING

STRATA PLOT

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

TESTPIT RECORD 2012 13-106 TEST PITS.GPJ HCE DATA TEMPLATE.GDT 5/24/13

PROJECT: 13-106

# RECORD OF TEST PIT 13-202

SHEET 1 OF 1

LOCATION: Lot H - Hawthorne Business Park

DATUM: Geodetic

DATE OF EXCAVATION: May 1, 2013

TYPE OF EXCAVATOR:

| DEPTH SCALE METRES | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80) | WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.91 | | | | | |
| | Brown silty sand and gravel and cobbles (FILL MATERIAL) | FILL | | 1 | | | | Backfilled with excavated materials |
| | Grey sand and gravel with asphalt (FILL MATERIAL) | FILL | 92.30<br>0.61 | | | | | ∇ |
| 1 | | | | | | | | |
| | Grey sand and gravel, metal and plastic (FILL MATERIAL) | FILL | 91.24<br>1.67 | | | | | |
| 2 | | | | 2 | | | | |
| | Stiff, brown grey silty clay, asphalt, wood and organic material (FILL MATERIAL) | FILL | 90.63<br>2.28 | 3 | | | | |
| 3 | End of test pit | | 89.86<br>3.05 | | | | | Groundwater seepage observed at 0.9 metres below ground surface at time of excavation |
| 4 | | | | | | | | |
| 5 | | | | | | | | |

DEPTH SCALE

1 to 25

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF BOREHOLE 13-1

PROJECT: 13-106

LOCATION: See Site Plan Figure 2

BORING DATE: May 10, 2013

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

Boring method: Power Auger, 200 mm Diameter Hollow Stem

| DEPTH SCALE METRES | SOIL PROFILE – DESCRIPTION | ELEV. / DEPTH (m) | SAMPLE NUMBER | TYPE | BLOWS/0.3m | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|
| 0 | Ground Surface | 92.63 | | | | |
| | Refer to TP 13-101 for fill description | | | | | Backfilled with soil cuttings |
| 3 | Dark grey silty clay, gravel and asphalt (FILL MATERIAL) | 89.58 / 3.05 | 1 | 50 DO | 7 | |
| 4 | Dark brown black PEAT (Former Topsoil) | 88.67 / 3.96 | 2 | 50 DO | 5 | |
| | Stiff, grey brown SILTY CLAY | 88.06 / 4.57 | 3 | 50 DO | 6 | |
| | | | 4 | 50 DO | 6 | |
| 6 | Grey fine silty, SAND trace silt | 86.70 / 5.93 | 5 | 50 DO | 1 | |
| | | | 6 | 50 DO | | |
| | | | 7 | 50 DO | 3 | |
| 8 | End of borehole<br>Refusal on inferred bedrock | 84.40 / 8.23 | | | | Please refer to test pit 13-101 for groundwater conditions |

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



DEPTH SCALE 1 to 50

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF BOREHOLE 13-2

PROJECT: 13-106

LOCATION: See Site Plan Figure 2

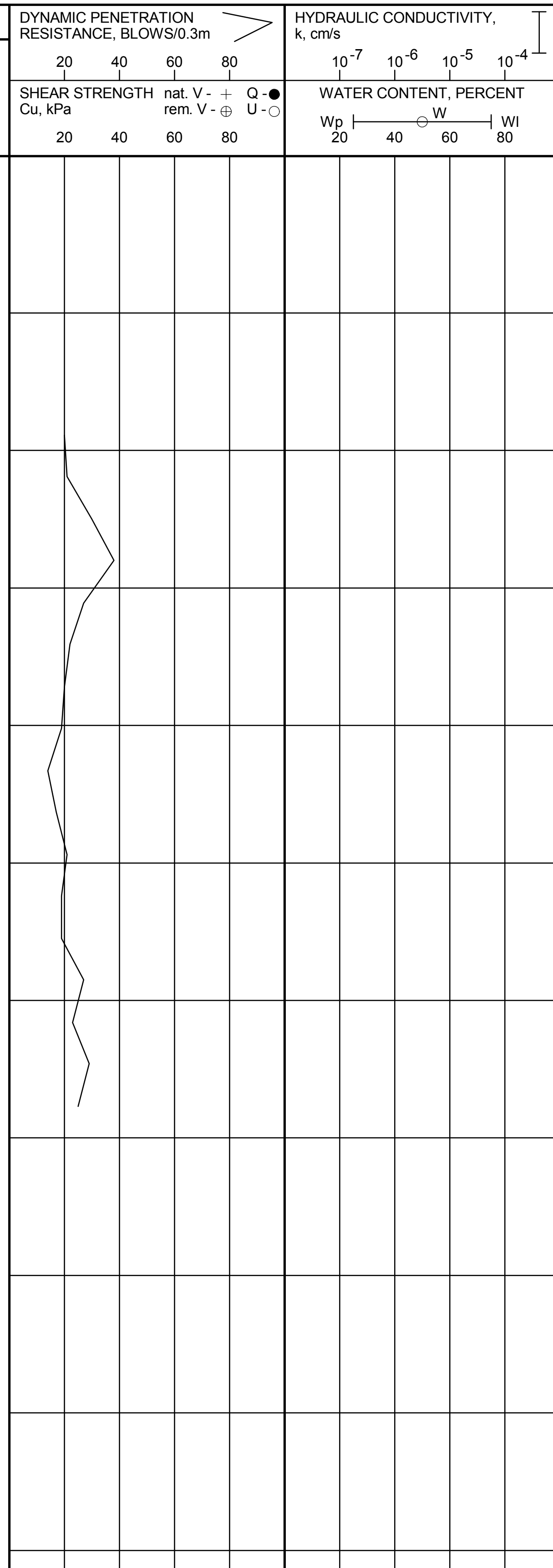
BORING DATE: May 10, 2013

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

| DEPTH SCALE METRES | BORING METHOD | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa | HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | Power Auger 200 mm Diameter Hollow Stem | Ground Surface | | 92.94 | | | | | | | |
| | | Refer to TP 13-102 for fill description | | | | | | | | | Backfilled with soil cuttings |
| 1.52 | Cone Penetration Test 38 mm Diameter Cone | | | 91.42<br>1.52 | | | | | | | |
| 6.89 | | End of borehole<br>Refusal on inferred bedrock | | 86.05<br>6.89 | | | | | | | Please refer to test pit 13-102 for groundwater conditions |




DEPTH SCALE

1 to 50

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF BOREHOLE 13-3

PROJECT: 13-106
LOCATION: See Site Plan Figure 2
BORING DATE: May 10, 2013

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

| DEPTH SCALE METRES | BORING METHOD | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa | HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | Power Auger 200 mm Diameter Hollow Stem | Ground Surface | | 92.27 | | | | | | | |
| | | Refer to TP 13-103 for fill description | | | | | | | | | Backfilled with soil cuttings |
| 3 | | Brown silty sand, gravel, clay and wood (FILL MATERIAL) | | 89.22 3.05 | 1 | 50 DO | 3 | | | | |
| | | Stiff, grey brown SILT some clay, trace wood (Former Topsoil) | | 88.61 3.66 | | | | | | | |
| 4 | | Brown grey SILTY SAND to SANDY SILT, trace clay | | 88.16 4.11 | 2 | 50 DO | 11 | | | | |
| | | Grey SAND trace to some silt | | 87.70 4.57 | 3 | 50 DO | 3 | | | | |
| 5 | Cone Penetration Test 38 mm Diameter Cone | | | 87.09 5.18 | | | | | | | |
| 8 | NQ Diamond Coring | Top of bedrock | | 84.30 7.97 | | | | | | | |
| | | SANDSTONE BEDROCK | | | 4 | RC | TCR = 79%, SCR = 78%, RQD = 78% | | | | |
| | | | | | 5 | RC | TCR = 100%, SCR = 100%, RQD = 100% | | | | |
| 10 | | End of borehole | | 81.91 10.36 | | | | | | | Please refer to test pit 13-103 for groundwater conditions |

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: $10^{-5}$ $10^{-4}$ $10^{-3}$ $10^{-2}$
WATER CONTENT, PERCENT: Wp W Wl 20 40 60 80

DEPTH SCALE
1 to 60

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L
CHECKED:

PROJECT: 13-106

# RECORD OF BOREHOLE 13-4

SHEET 1 OF 1

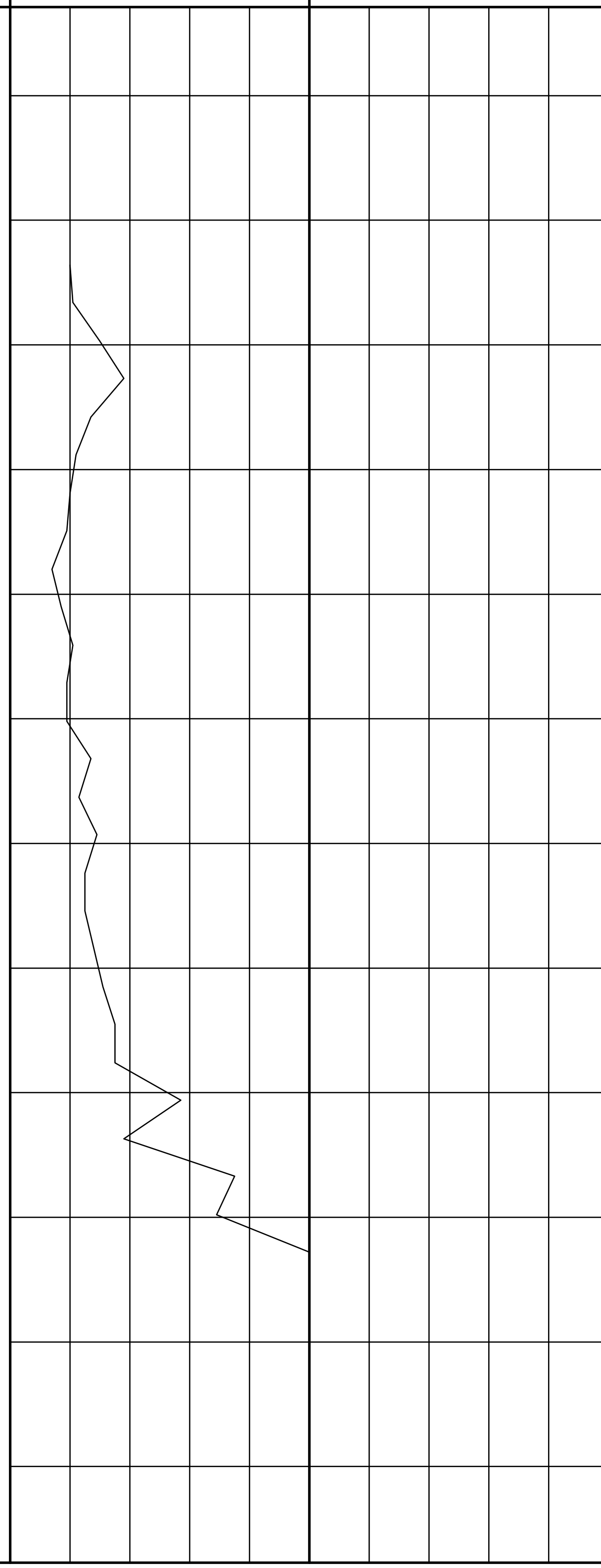
LOCATION: See Site Plan Figure 2

DATUM: Geodetic

BORING DATE: May 10, 2013

SPT HAMMER: 63.5 kg; drop 0.76 metres

| 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 Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | Power Auger 200 mm Diameter Hollow Stem | Ground Surface | | 92.47 | | | | | | | |
| | | Refer to TP 13-104 for fill description | | | | | | | | | Backfilled with soil cuttings |
| 1–9 | Cone Penetration Test 38 mm Diameter Cone | | | | | | | | | | |
| 9.76 | | End of borehole<br>Refusal on inferred bedrock | | 82.71<br>9.76 | | | | | | | Please refer to test pit 13-104 for groundwater conditions |
| 10, 11, 12 | | | | | | | | | | | |



DEPTH SCALE

1 to 60

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

CHECKED:

# RECORD OF BOREHOLE 13-5

PROJECT: 13-106
LOCATION: See Site Plan Figure 2
BORING DATE: May 10, 2013

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

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

| DEPTH SCALE METRES | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | SAMPLE NUMBER | TYPE | BLOWS/0.3m | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|
| 0 | Ground Surface | | 92.00 | | | | |
| | Brown sand and gravel and grey silty clay with ashphalt and brick (FILL MATERIAL) | | | 1 | 50 DO | 22 | Backfilled with soil cuttings |
| 2 | | | | 2 | 50 DO | 17 | |
| 3 | Stiff, brown SILTY CLAY with organic material | | 88.65<br>3.35 | 3 | 50 DO | 12 | |
| | | | 88.34<br>3.66 | | | | |
| 8 | End of borehole | | 83.67<br>8.33 | | | | Please refer to test pit 13-105 for groundwater conditions |

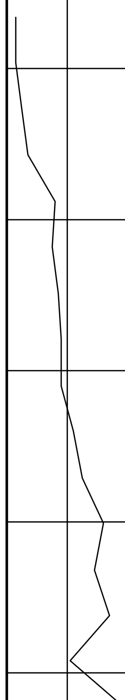
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

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



DEPTH SCALE
1 to 50

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L
CHECKED:

# RECORD OF BOREHOLE 13-6

PROJECT: 13-106

LOCATION: See Site Plan Figure 2

BORING DATE: May 10, 2013

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

| 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— Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | Power Auger 200 mm Diameter Hollow Stem | Ground Surface | | 91.83 | | | | | | | |
| | | Brown silty sand and gravel with asphalt and brick (FILL MATERIAL) | | | | | | | | | Backfilled with soil cuttings |
| 1 | | | | | 1 | 50 DO | 24 | | | | |
| 2 | | Brown grey silty sand with some gravel and clay (FILL MATERIAL) | | 89.55 2.28 | 2 | 50 DO | 3 | | | | |
| 3 | | | | | 3 | 50 DO | 5 | | | | |
| | | Grey brown SILTY CLAY | | 88.17 | | | | | | | |
| 4 | | Brown grey SAND trace to some silt | | 88.02 3.81 | 4 | 50 DO | 6 | | | | |
| 5 | | | | | 5 | 50 DO | 4 | | | | |
| | | Brown grey SILTY SAND | | 86.50 5.33 | 6 | 50 DO | 7 | | | | |
| 6 | | | | | 7 | 50 DO | 6 | | | | |
| 7 | | | | | 8 | 50 DO | 49 | | | | |
| 8 | | End of borehole Refusal on inferred bedrock | | 83.98 7.85 | 9 | 50 DO | 50 | | | | Please refer to test pit 13-106 for groundwater conditions |
| 9 | | | | | | | | | | | |
| 10 | | | | | | | | | | | |

DEPTH SCALE

1 to 50

**Houle Chevrier Engineering Ltd.**

LOGGED: M.L

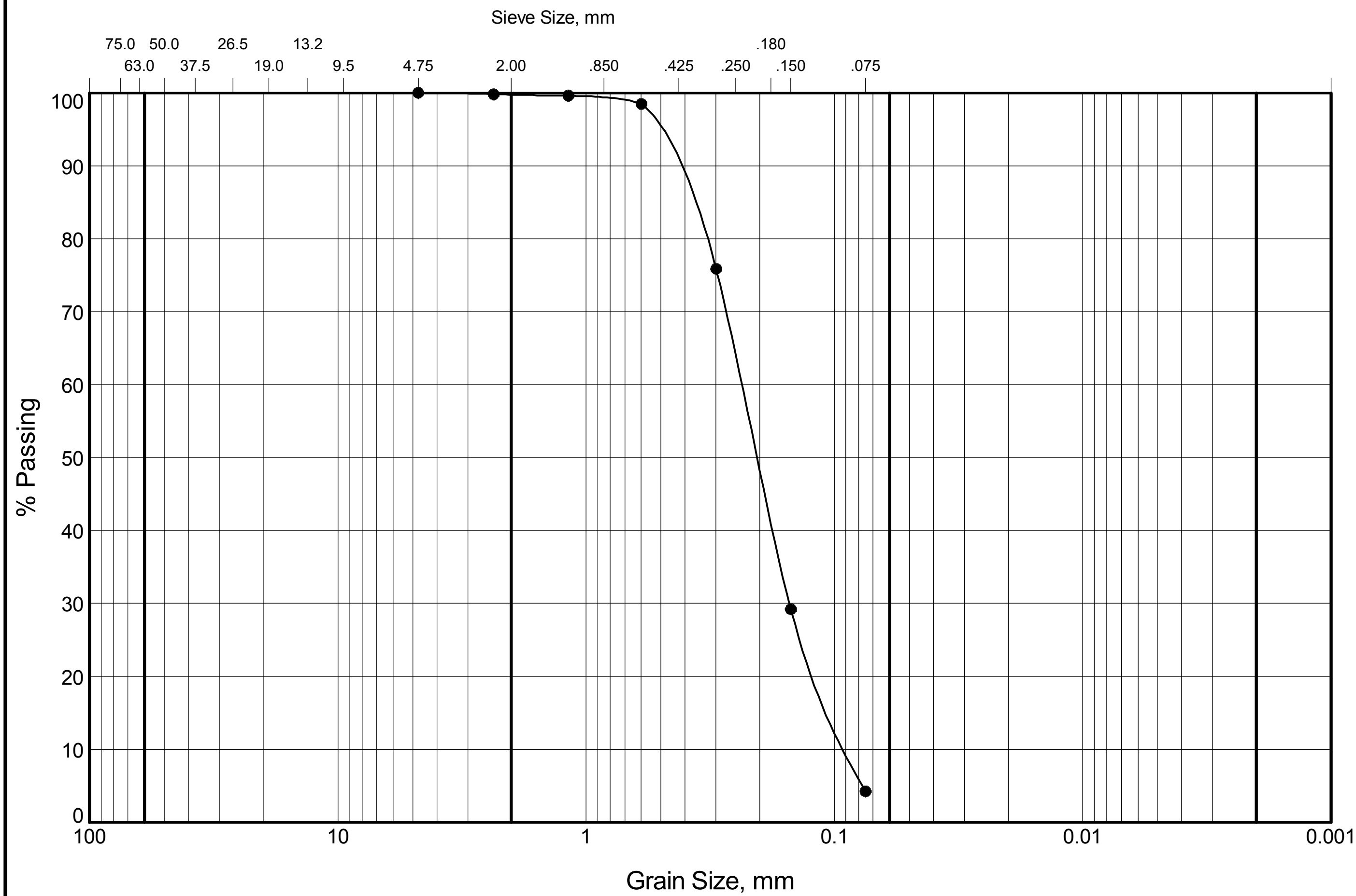
CHECKED:

APPENDIX B

LABORATORY TEST RESULTS

# GRAIN SIZE DISTRIBUTION

**FIGURE #1**




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

| Borehole | Sample | Depth (m) | Legend |
|---|---|---|---|
| 13-3 | 3 | 4.6 - 5.2 | ● |

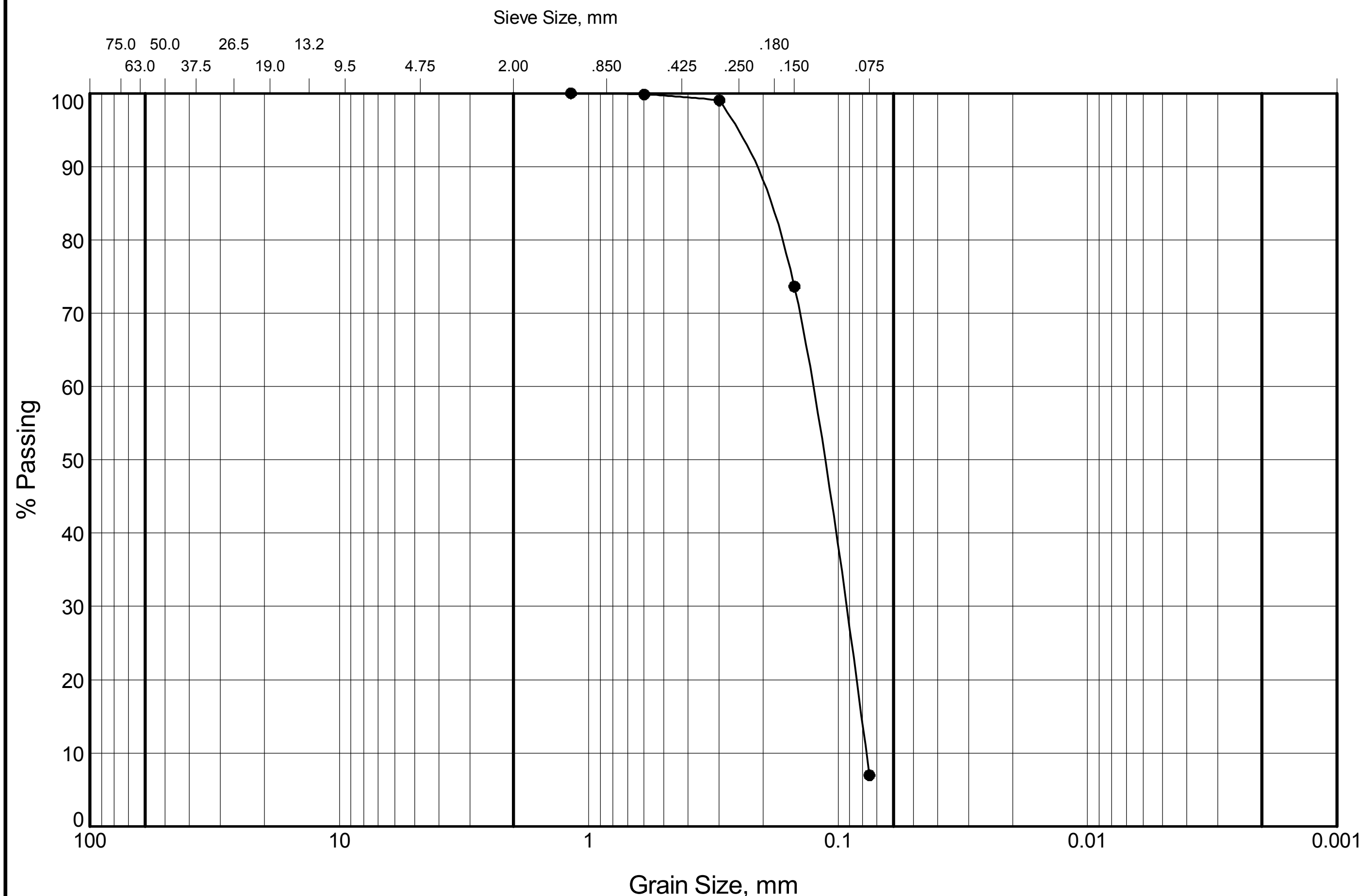
Houle Chevrier Engineering

Date:

Project: 13-106

# GRAIN SIZE DISTRIBUTION

**FIGURE #2**




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

| Borehole | Sample | Depth (m) | Legend |
|---|---|---|---|
| 13-6 | 4 | 3.8 - 4.4 | ● |



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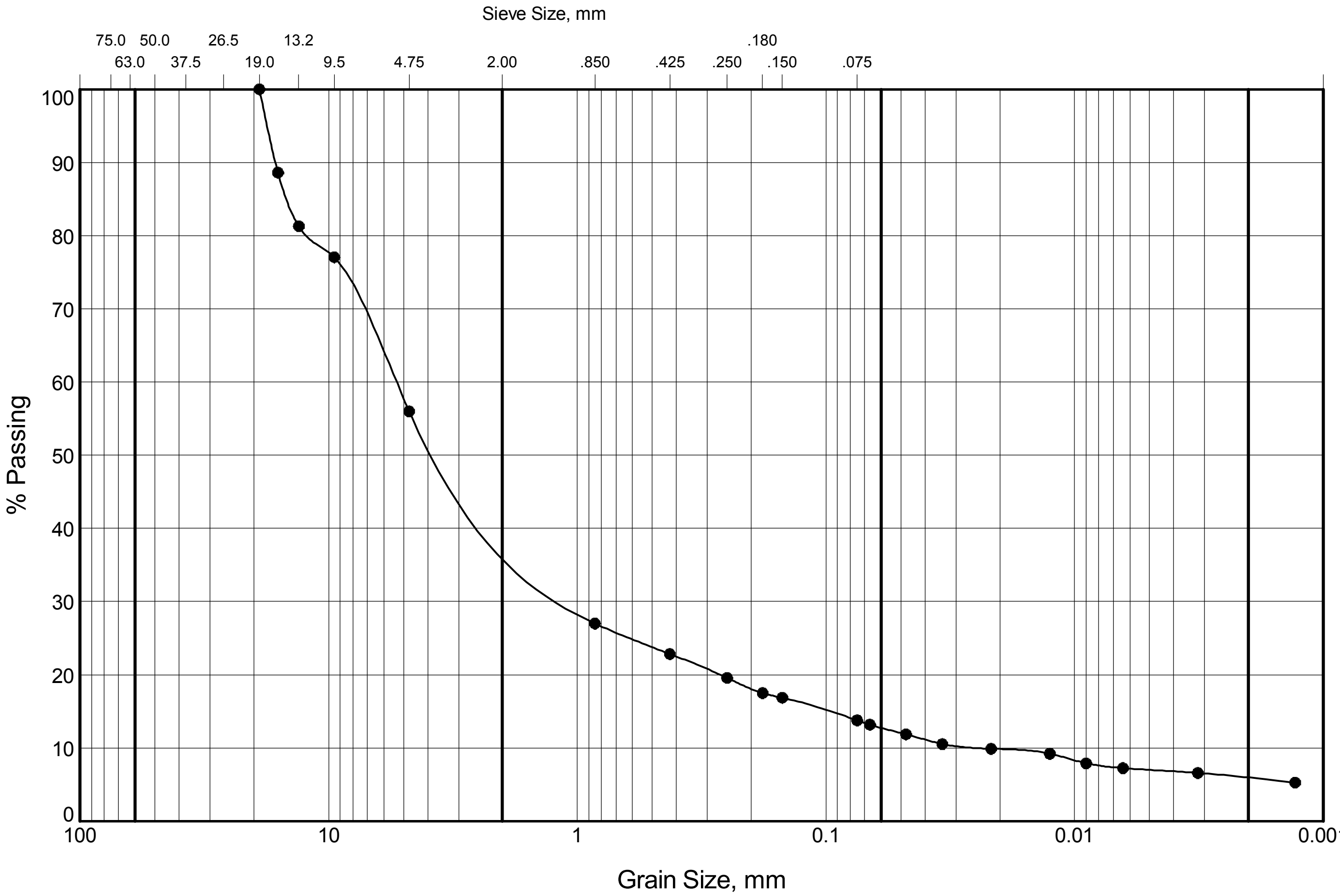
# GRAIN SIZE DISTRIBUTION

**FIGURE #3**




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

| Borehole | Sample | Depth (m) | Legend | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|
| 13-6 | 2 | 2.3 - 2.9 | ● | 58.6 | 28.7 | 6.9 | 5.9 |

SOILS GRAIN SIZE WITH PERCENT GRAVSILTCL 13-106 LOGS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 5/24/13

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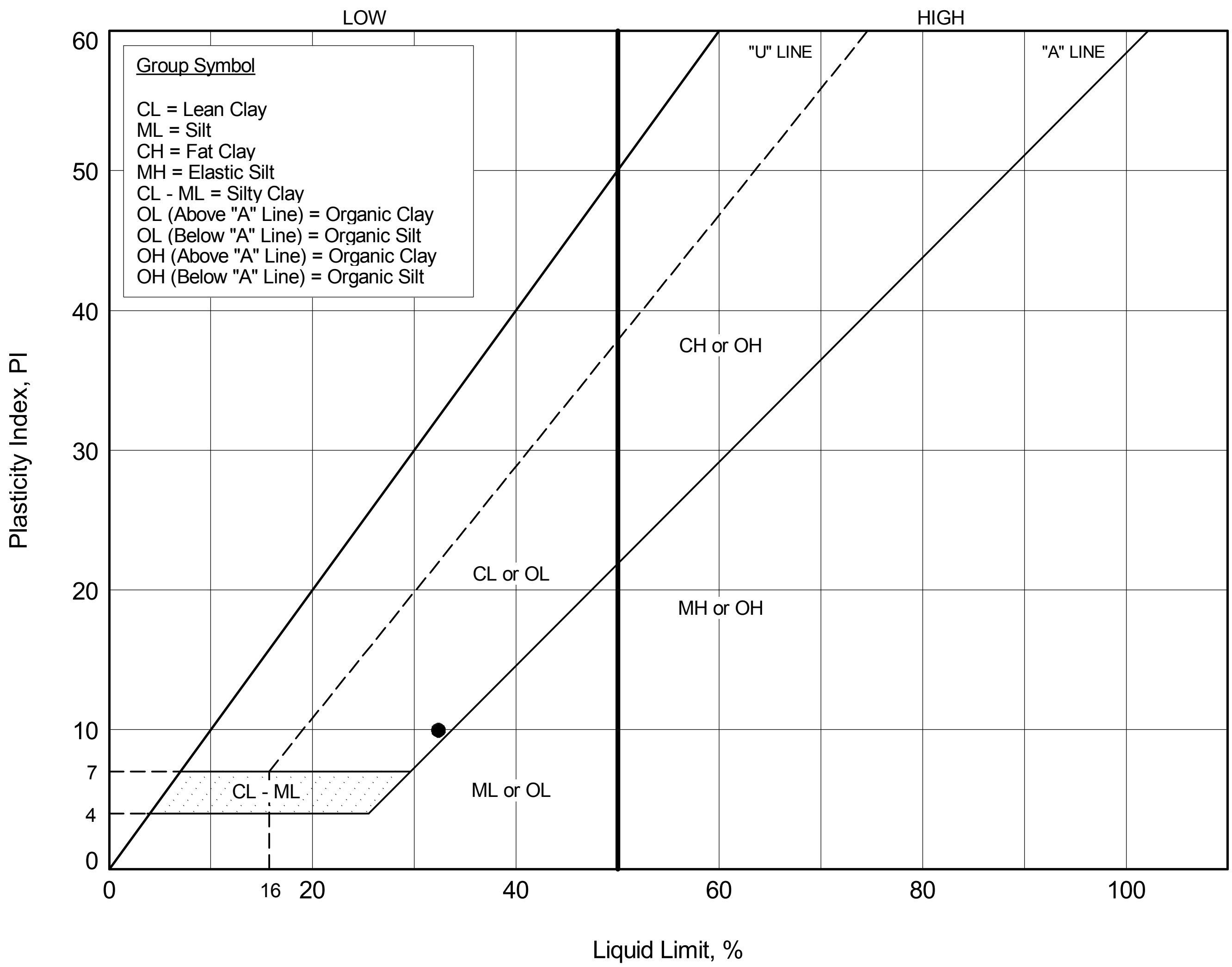
Date:

Project: 13-106

## PLASTICITY CHART

## FIGURE #4




| Borehole | Sample | Depth (m) | Moisture Content, % | Legend |
|---|---|---|---|---|
| 13-1 | 3 | 4.6 - 5.7 | 41.1 | ● |