Geotechnical Investigation
And Slope Stability Assessment
3455 Milton Road
Navan, Ontario
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Navan, Ontario

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1.0 INTRODUCTION

Plans are being prepared to develop a vacant parcel of land located on the east side of Milton Road in the village of Navan, Ontario. The site is bordered on the south by a former rail line and on the north by a vacant parcel of land. An existing residential subdivision is located to the east of the subject site. The parcel of land consists of approximately 12.57 hectares (31.06 acres) of land (see Key Plan, Figure 1). An east-west aligned slope divides the southern portion of the site from the northern portion. The northern portion of the site is heavily treed.

The proposed development plans include a residential subdivision consisting of twenty one (21) village residential lots serviced with on-site septic disposal systems and water supply wells. The proposed lots will be serviced by an internal roadway system. A proposed storm water management pond will be located in the south west corner of the site. The proposed layout of the development is shown on the Test Hole Location Plan, Figure 2.

This report presents the results of a geotechnical investigation and slope stability assessment carried out for the site. The geotechnical investigation was carried out in two stages between September 2013 and August 2014. The purpose of the investigation was to identify the subsurface conditions at the site by means of a limited number of test pits and boreholes and, based on the results of the factual information obtained, to provide engineering guidelines and recommendations on the geotechnical design aspects of this project, along with construction considerations that could influence design decisions.

The analyses provided herein were carried out in accordance with our proposal dated May 2, 2014.

2.0 SITE OVERVIEW

2.1 Site Description and Review of Geology Maps

The subject site is located on a vacant parcel of land located at 3455 Milton Road in the Village of Navan, Ontario. The site is bounded by a former railway on the south, a vacant parcel of land on the north, Milton Road on the west and an existing residential subdivision on the east.

An east-west aligned slope divides the southern portion of the site from the northern portion. The northern portion of the site is heavily treed. A channel is located within a north-south aligned internal ravine that extends from Meteor Avenue and outlets at the east-west aligned slope.

Published geology maps of the area indicate that the subsurface conditions are expected to consist of marine deposits of silty clay. The underlying bedrock is mapped as shale bedrock of the Billings formation at depths of between 15 and 25 metres.
2.2 Description of Slopes

A site reconnaissance was carried out on May 30, 2014 by a member of our engineering staff. At that time, the geometry of the slopes throughout the site was measured at a total of seven (7) locations using level surveying techniques. The cross sections were positioned at the site by Houle Chevrier Engineering Ltd. personnel at key locations based on slope geometry and height. Sections ‘A-A’ to ‘D-D’, inclusive, are located along the east-west aligned slope and Sections ‘E-E’ to ‘G-G’, inclusive were positioned along the ravine side slopes. The locations of the seven (7) cross sections considered are provided on the Cross Section Location Plan, Figure 3. Cross sections of the slopes are provided on Figures B1 to B7, inclusive, in Appendix B.

The geometries of the cross sections considered are summarized in the following table:

Table 2.1 – Geometries of Cross Sections

<table>
<thead>
<tr>
<th>Cross Section</th>
<th>Slope Height (metres)</th>
<th>Overall inclination from horizontal (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-A</td>
<td>3.5</td>
<td>24</td>
</tr>
<tr>
<td>B-B</td>
<td>7.0</td>
<td>16</td>
</tr>
<tr>
<td>C-C</td>
<td>4.5</td>
<td>16</td>
</tr>
<tr>
<td>D-D</td>
<td>6.5</td>
<td>18</td>
</tr>
<tr>
<td>E-E</td>
<td>5</td>
<td>19</td>
</tr>
<tr>
<td>F-F</td>
<td>5.5</td>
<td>21</td>
</tr>
<tr>
<td>G-G</td>
<td>3.0</td>
<td>18</td>
</tr>
</tbody>
</table>

In general, the east-west aligned slopes (i.e., Sections ‘A-A’ to ‘D-D’, inclusive) are vegetated with shrubs and small to large trees. No signs of slope instability were observed at the time of the site visit.

A channel is located within the internal ravine that extends from Meteor Avenue and outlets at the east-west aligned slope (i.e. Sections ‘E-E’ to ‘G-G’, inclusive). Active soil erosion was
observed along the bottom (toe) of the adjacent slopes. No signs of overall slope instability (i.e., rotational failures, tension cracks, etc.) were observed at the time of the site visit.

3.0 SUBSURFACE INVESTIGATION

The geotechnical investigation was carried out in two stages between September 2013 and August 2014.

On September 26, 2013, eighteen (18) test pits, numbered 13-01 to 13-18, inclusive, were advanced at the site using a mini excavator supplied and operated by the property owner. The test pits were advanced to depths ranging between 2.1 and 2.6 metres below ground surface. 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 short-term groundwater conditions in the open test pits were observed on completion of excavating.

On July 31 and August 1, 2014, six (6) boreholes numbered 14-1 to 14-6, inclusive, were advanced at the site using a track mounted drill rig supplied and operated by Aardvark Drilling Inc. The boreholes were advanced to depths ranging between 5.9 and 20.0 metres below ground surface. A Dynamic Cone Penetration Test was advanced in one of the boreholes from a depth of about 16.5 metres to 20.0 metres below ground surface. Standard penetration tests were carried out in the boreholes at regular depth intervals and samples of the soils encountered were recovered using 50 millimetre diameter split barrel sampling equipment. In situ vane shear testing was carried out, where possible, within the silty clay to measure the undrained shear strength of these deposits. Standpipe piezometers were installed in three (3) of the boreholes upon completion of drilling to measure the groundwater level.

One sample of the groundwater recovered from borehole 14-1 was submitted to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The field work was supervised throughout by members of our engineering staff who directed the drilling and excavating 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 distribution and plasticity index.

Descriptions of the subsurface conditions logged in the boreholes and test pits are provided on the Record of Borehole sheets and Test Pit sheets in Appendix A. The results of the laboratory classification tests on the soil samples are provided on Figures C1 to C3, inclusive, in Appendix C and the Record of Borehole sheets. The results of the chemical analysis of the groundwater sample relating to corrosion are provided in Appendix D.
The test pit locations were determined by Novatech Engineering Consultants Ltd. The borehole locations were determined relative to existing site features by Houle Chevrier Engineering Ltd. Personnel.

The ground surface elevation at the location of boreholes 14-3, 14-5 and 14-6 was determined using a Trimble R8 global positional system. The elevation is referenced to geodetic datum and is considered to be accurate within the tolerance of the instrument. The ground surface elevation could not be determined at the location of boreholes 14-1, 14-2 and 14-4 due to the presence of relatively heavy tree cover.

The approximate locations of the boreholes and test pits are shown on the Test Hole Location Plan, Figure 2.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil and groundwater conditions logged in the boreholes and test pits are given on the Record of Borehole sheets and Test Pit sheets in Appendix A. The 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 frequency and recovery of samples, the method of sampling and the uniformity of the subsurface conditions. Subsurface conditions at locations other than the test locations may vary from the conditions encountered in the test holes.

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

The following presents an overview of the subsurface conditions encountered in the test pits and boreholes advanced during the September 2013 and August 2014 investigations.

4.2 Topsoil

A surficial layer of topsoil was encountered at all of the borehole and test pit locations. The topsoil is generally composed of dark brown sandy silt with trace to some organic material and has a thickness ranging between about 30 and 250 millimetres.

Moisture content testing carried out on samples of the topsoil indicates moisture contents of 22 and 33 percent.
4.3 Sand

Deposits of reddish grey brown and brown to grey brown sand were encountered in boreholes 14-1 to 14-4, inclusive and in test pits 13-01 to 13-04, inclusive, 13-10, and 13-12 to 13-18, inclusive, at depths ranging from about 0.15 to 0.25 metres below ground surface. No sand deposits were encountered in the boreholes and test pits advanced south of the east-west aligned slope. The sand deposits contain trace to some silt and have a thickness ranging between about 0.4 to 1.6 metres.

A layer of grey silty sand was encountered in test pit 13-01 below the sand layer at a depth of about 1.0 metre below ground surface and extends to a depth of about 1.8 metres.

Three (3) grain size distribution tests were undertaken on samples of the sand from test pits 13-10, 13-15 and 13-17. The test results are provided on Figure C1 in Appendix C. Moisture content testing carried out on samples of the sand indicates moisture contents of 13 to 21 percent.

4.4 Silty Clay

Native deposits of silty clay were encountered in all boreholes and test pits below the topsoil, sand and silty sand at depths ranging between about 30 millimetres and 1.8 metres below ground surface.

The upper part of the silty clay is weathered and reddish grey brown. Where fully penetrated the weathered crust has a thickness of between about 0.4 and 3.1 metres and extends to depths ranging from 1.5 to 4.6 metres below ground surface. No weathered crust was encountered in test pit 13-16.

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 50 to 82 kilopascals, which reflect a stiff consistency. Standard penetration tests carried out in this layer gave N values ranging from 1 to 7 blows per 0.3 metres of penetration, which reflect a very stiff to stiff consistency.

Grey silty clay was encountered below the weathered silty clay crust and sand deposits at all borehole locations and some test pit locations at depths ranging from about 1.5 to 4.6 metres below ground surface.

The results of in situ vane shear strength testing carried out in the grey silty clay deposits indicate undrained shear strength values ranging from about 23 to 80 kilopascals, which reflect a soft to stiff consistency. Standard penetration tests carried out in this layer gave N values ranging from 1 blow to “manual pressure” (MP) per 0.3 metres of penetration, which reflect a firm to stiff consistency.
Five (5) grain size distribution tests were undertaken on samples of the silty clay from boreholes 14-3, 14-4 and test pits 13-05, 13-08 and 13-17. The test results are provided on Figure C2 in Appendix C.

Two (2) Atterberg limit tests were undertaken on samples of the silty clay recovered from boreholes 14-3 and 14-3 at depths of about 1.5 and 2.3 metres below ground surface. The results show that the samples of silty clay have a liquid limit of 50 and 54 percent and a plastic limit of 21 percent; as indicated on the plasticity chart on Figure C3 in Appendix C. Moisture content testing carried out on samples of the silty clay indicates moisture contents of 31 to 85 percent.

4.5 Groundwater Conditions

The groundwater levels measured in the standpipe piezometers installed in boreholes 14-1, 14-4 and 14-5 ranged from about 0.8 to 2.4 metres below ground surface on August 12, 2014.

Groundwater seepage was observed in test pits 13-02, 13-06, 13-08, 13-13, 13-15, 13-17 and 13-18 at depths of about 0.9 to 2.5 metres below ground surface. It should be noted that the groundwater inflow was only observed during the relatively short period of time that the test pits were left open following excavation.

The groundwater levels are expected to vary seasonally and may be higher during wet periods of the year such as the early spring or following periods of precipitation, particularly within the upper sand deposits.

4.6 Groundwater Chemistry Relating to Corrosion

The results of chemical testing on a sample of groundwater recovered from borehole 14-1 are provided in Appendix D and summarized below.

- Conductivity 551 µS/cm (microsiemen per centimetre)
- Hardness 272 gm/L (milligram per litre)
- pH 7.5
- Chloride 9 mg/L (milligram per litre)
- Sulphate 109 mg/L (milligram per litre)

5.0 GEOTECHNICAL GUIDELINES

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the boreholes advanced as part of this investigation, the available test pit information, and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is...
intended for 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 and have not been investigated or addressed.

5.2 Single Family Houses

5.2.1 Excavation

The excavations for the foundations should be taken through any surficial fill, topsoil, or otherwise deleterious material to expose undisturbed native deposits of sand or silty clay. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the shallow native overburden deposits can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical extending upwards from the base of the excavation.

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 14-1, 14-4 and 14-5 ranged from about 0.8 to 2.4 metres below ground surface on August 12, 2014. 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 is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

Groundwater seepage was observed in some test pits at depths ranging between about 0.9 and 2.5 metres below ground surface. It should be noted that the groundwater inflow was only observed during the relatively short period of time that the test pits were left open following excavation and therefore may not represent stabilized groundwater levels.
5.2.2 Placement of Engineered Fill

In areas where the proposed founding level is above the level of the native soil, or where subexcavation of disturbed material is required below proposed founding level, imported granular material (engineered fill) should be used. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. In areas where groundwater inflow is encountered, pumping should be carried out from sumps in the excavation during placement of the engineered fill. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.2 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations for the residential dwellings should be sized to accommodate this fill placement. Since the source of recycled material cannot be determined, it is suggested that for environmental reasons any granular materials used below founding level be composed of virgin material only. The engineered fill should be placed in accordance with the site grade raise restrictions, where applicable.

5.2.3 Grade Raise Restrictions

The site is underlain by deposits of sensitive grey silty clay, which have, in general, have a firm to stiff consistency. However, in boreholes 14-1 and 14-5, soft silty clay was encountered and in other investigations in this general area, deposits of soft, sensitive silty clay are known to exist in this vicinity. The placement of fill material across the site must be controlled so that the stress imposed by the fill material does not result in excessive consolidation of the grey silty clay deposits. The settlement response of the silty clay deposits due to the increase in stress caused by fill material is influenced by variables such as the existing effective overburden pressure, the past preconsolidation pressure for the silty clay, the compressibility characteristics of the silty clay, and the presence or absence of drainage paths, etc. It is well established that the settlement response of silty clay deposits can be significant when the stress increase is near or above the preconsolidation pressure.

For design purposes, the grade raise across the site should be restricted to about 1 metre, assuming that conventional earth fill (i.e. sand, silty sand) is used around the proposed houses. If somewhat lighter silty clay was used for grade raise purposes around the houses, a maximum grade raise of 1.2 metres could be used. For design purposes, this grade raise restriction assumes that the maximum groundwater lowering due to the development of this site is 1 metre below the highest measured groundwater level.

5.2.4 Foundation Bearing Capacity

In general, the native sand and weathered silty clay crust are considered suitable to support residential structures founded on conventional strip or pad footings foundations. All organic material, topsoil, and loose or water softened soils should be removed from within the proposed footing areas.
The bearing pressures for strip 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 grey silty clay beneath the weathered crust are:

- The thickness of the soil beneath the base of the foundation and the surface of the sensitive silty clay;
- The size, type (i.e. pad or strip) and loading of the foundation;
- The amount of surcharge (fill, etc.) in the vicinity of the foundation; and
- The amount of post-development groundwater lowering at the site.

From a spread footing design perspective, it is preferable to maximize the vertical separation between the underside of the footings and the surface of the softer, grey silty clay to distribute the foundation loads onto the softer, grey silty clay at depth. This can be achieved by founding the structures as high as practical within the soil profile and minimizing the amount of fill (surcharge) on the site. The vertical separation distance between the underside of footings and the surface of the softer, sensitive grey silty clay should not be less than about 1.0 metre (without approval of the geotechnical engineer).

The proposed finished grades and floor slab elevations of the buildings and the amount of grade raise fill around the house and in the garage were not available at the time of this report. For preliminary planning and design purposes, the following could be considered for the design of the residential building foundations:

### Table 5.1 – Foundation Bearing Capacity

<table>
<thead>
<tr>
<th>Type of Footing</th>
<th>Maximum Size of Footing (metres)</th>
<th>Net Geotechnical Reaction at Serviceability Limit State (SLS) (kilopascals)</th>
<th>Factored Net Geotechnical Resistance at Ultimate Limit State (ULS) (kilopascals)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior Strip</td>
<td>1.0</td>
<td>90</td>
<td>150</td>
</tr>
<tr>
<td>Interior Pad</td>
<td>1.2 square</td>
<td>90</td>
<td>150</td>
</tr>
</tbody>
</table>

Note:
1. The total and differential settlement of the foundation at SLS should be less than 20 and 25 millimetres, respectively
For the purpose of this assessment we have considered a long term groundwater lowering at the site equal to 1 metre below the highest measured groundwater level. Provided that any loose or disturbed soil is removed from the bearing surfaces, the post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres.

Due to the variable depth to the top of the softer, grey silty clay layer it is proposed that the actual allowable bearing pressure and grade raise restriction for each residential house be determined on a lot by lot basis at the time of construction by advancing two (2) hand auger probe holes from founding elevation to determine the actual depth below the footing elevation to the surface of the firm silty clay.

5.2.5 Frost Protection of Foundations
All exterior footings and those in any unheated parts of the structures should be provided with at least 1.5 metres of earth cover for frost protection purposes. If 1.5 metres of earth cover is not practicable (for example for walkout type basements) a combination of earth cover and polystyrene insulation could be considered. Further details regarding the insulation of foundations could be provided upon request.

5.2.6 Basement Foundation Wall Backfill and Drainage
In accordance with the Ontario Building Code, the following alternatives could be considered for drainage of the basement foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel such as that meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I or II. OR

- Damp proof the exterior of the foundation walls and install approved proprietary drainage material on the exterior of the foundation walls and backfill the walls with native material or imported soil.

A perforated drain should be installed around the basement area at the level of the bottom of the footings. The drain should outlet to a sump from which the water is pumped or should drain by gravity to a suitable drainage outlet.

5.2.7 Garage Foundation and Pier Backfill
To avoid adfreeze and possible jacking (heaving) of the foundation walls, between the unheated garage foundation walls and the wall backfill, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I or II. The backfill within the garage should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory
compaction equipment. Alternatively, 19 millimetre clear crushed stone could be used as backfill within garages. Compaction of the clear stone is not essential.

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as sand meeting OPSS Granular B Type I or II requirements. Other measures to prevent frost jacking of these foundation elements could be provided, if required.

5.2.8 Seismic Site Class
In accordance with the 2012 National Building Code of Canada and the Ontario Building Code, seismic Site Class E should be used for design purposes. In our opinion, there is no potential for liquefaction of the soils below founding level.

5.2.9 Effects of Trees on the Foundations
The results of this investigation indicate that sensitive silty clay exists. This material 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 clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations. 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 proposed buildings, services and other ground supported structures should be considered in the landscaping design.

5.3 Storm Water Management Pond
It is our understanding that a storm water management pond will be located in the south west corner of the site. The dimensions of the pond, the base elevation and the side slopes were not available at the time of this report.

5.3.1 Excavation
Based on the results of the investigation, the excavation for the pond will be carried out through topsoil and native deposits of silty clay. All topsoil should be removed from within the entire footprint area of the excavation.

Based on our previous experience, groundwater inflow from the native silty clay deposits into the excavation should be relatively small and controlled by pumping from filtered sumps within the excavation.
5.3.2 Side Slopes
As indicated previously, the maximum depth of the proposed pond was not known at the time of this report. Based on the results of the borehole and test pit investigation, we suggest that the ponds be constructed with 2.5 horizontal to 1 vertical, or flatter, side slopes.

The excavated side slopes could be protected against erosion using topsoil and suitable vegetation.

5.3.3 Perimeter Berms
Prior to the construction of perimeter berms, all topsoil and any loose, wet or otherwise disturbed material should be removed from within the entire footprint area of the berm. The surface of the exposed silty clay should be scarified to a depth of at least 200 millimetres using the teeth on the bucket of the excavator dragged along the ground surface in a direction parallel to the axis of the berm. This should ensure that the interface between the base of the berm and the surface of the underlying soil layer is relatively impermeable. Alternatively, the base of the berm could be keyed into the underlying soil.

The fill material for the proposed berm should consist of suitable, relatively low permeability material such as suitable silty clay earth borrow material. The placement of the silty clay material for construction of the berm should be carried out in maximum 200 millimetre thick lifts and each lift should be compacted to at least 95 percent of the standard Proctor dry density value. Our experience with compaction of weathered silty clay has shown that the use of large diameter (i.e., 1.5 metre) smooth drum compaction equipment is best suited for this purpose.

Compaction of the weathered silty clay material is best achieved when the material has a moisture content that is 0 to 3 percent above the optimum for compaction, as determined by Standard Proctor testing. Some drying and/or wetting of the silty clay material may be required during the berm construction to maintain the moisture content of the silty clay within the range of optimum for compaction purposes. The exterior surfaces of the berm should be covered with topsoil and seed or mulch for erosion purposes.

5.3.4 Erosion Control at the Pond Inlet and Outlet
The inlet and outlet of the pond should be protected from erosion using rip rap. The size and gradation of the rip rap will be a function of the expected water flow rate. The thickness of the rip rap should be determined based on the maximum size of the rip rap.

A nonwoven geotextile (OPSS 1860, Class II) separator should be placed between the rip rap and the subgrade surface. All seams in the geotextile should be provided with at least 0.5 metres of overlap.
5.3.5 Long Term Groundwater Inflow

The groundwater level in the well screen installed in borehole 14-5 was about 0.8 metres below ground surface (elevation 70.0 metres, geodetic datum) on August 12, 2014. Depending on the base elevation of the pond, groundwater inflow from the overburden into the pond should be anticipated.

5.4 Access Roadways

5.4.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil and any soft, wet or deleterious materials should be removed from the proposed roadways. Any subexcavated areas could be filled with compacted earth borrow or well shattered and graded rock fill material. Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material, earth borrow or well shattered and graded rock fill material may be used. In low, wet areas, well shattered and graded rock fill material is preferred. The 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. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both. Prior to placing granular material for the roadway, the exposed subgrade should be heavily proof rolled and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable earth borrow or rock fill approved by the geotechnical engineer.

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

5.4.2 Pavement Design

For the roadways within this residential development, the following minimum pavement structures are suggested:

**Local Roads**

- 80 millimetres of hot mix asphaltic concrete
- 150 millimetres of OPSS Granular A base over
- 375 millimetres of OPSS Granular B, Type II subbase

The above pavement structure assumes that any 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.

5.4.3 Granular Material Placement
The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.4.4 Asphaltic Concrete Types
For all pavements, the asphaltic concrete should consist of a 40 millimetre surface layer of Superpave 12.5 (Traffic Level A) or OPSS HL3 over one 40 millimetre lift of Superpave 19.0 (Traffic Level A) or OPSS HL8 asphaltic concrete. Performance grade PG 58-34 asphaltic concrete should be specified in accordance with City of Ottawa standards.

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

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

In order to provide drainage of the granular subbase, it is suggested that catch basins be provided with perforated stub drains extending about 3 metres out from the catch basins in two directions parallel to the roadway. These drains should be installed at the bottom of the subbase layer.

5.5 Corrosion of Buried Concrete and Steel
The measured sulphate concentration in the groundwater sample recovered from borehole 14-1 was 109 milligrams per litre. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use (formerly Type 10) cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) near the proposed building should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the resistivity and pH of the groundwater sample, the soil can be classified as slightly aggressive toward unprotected steel. It is noted that the corrosivity of the soil and groundwater could vary throughout the year due to the application of sodium chloride for de-icing.
5.6 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, etc.) will cause ground vibration on the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. It is suggested therefore that preconstruction surveys be carried out on any existing, nearby structures and water supply wells.

5.7 Slope Stability Analysis

5.7.1 General

The purpose of this slope stability assessment is to establish the ‘Erosion Hazard Limit’ for the site. This limit constitutes a safe setback for any proposed development at the site with respect to slope stability. The Erosion Hazard Limit was determined based on the Natural Hazard Policies set forth in Section 3.1 of the Provincial Policy Statements of the Planning Act of Ontario. Current regulations restrict development within the Erosion Hazard Limit.

The slope stability analyses were carried out at Sections ‘A-A’ to ‘G-G’, inclusive, using SLIDE, a state of the art, two dimensional limit equilibrium slope stability program. The results of the slope stability analyses are provided in Appendix B.

5.7.2 Soil Strength Parameters

The soil conditions used in the stability analyses were based, in part, on the results of the boreholes and test pits advanced across the site. The slope stability analyses were carried out using silty clay strength parameters based on site specific studies in the Ottawa area. To determine the existing factor of safety against overall rotational failure, the slope stability analyses were carried out using drained soil parameters, which reflect long term conditions.

The following table summarizes the soil parameters used in the analyses:

**Table 5.2 – Soil Parameters**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Effective Angle of Internal Friction, ( \varphi ) (degrees)</th>
<th>Effective Cohesion, ( c' ) (kilopascals)</th>
<th>Unit Weight, ( \gamma ) kN/m(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>32</td>
<td>0</td>
<td>19</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>30</td>
<td>10</td>
<td>17.5</td>
</tr>
</tbody>
</table>

The results of a stability analysis are highly dependent on the assumed groundwater conditions. No information is available on the long term groundwater levels throughout the year; however,
as a conservative approach, we have assumed full hydrostatic saturation with the groundwater level at ground surface and groundwater flow horizontally towards the slope.

The slope stability analyses were carried out using soil parameters, groundwater conditions and a slope profile that attempt to model the slopes in question but do not exactly represent the actual conditions. For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 is considered to represent a slope bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate a slope that is less likely to fail in the long term and provides a degree of confidence against failure ranging from marginal (1.3) to adequate (1.4 and greater) should conditions vary from the assumed conditions. A factor of safety of 1.5, or greater, is considered to indicate adequate long term stability.

5.7.3 Existing Conditions

The slope stability analyses indicated that the existing slopes, in their current configurations, have the following factors of safety against overall rotational failure:

Table 5.3 – Existing Factor of Safety

<table>
<thead>
<tr>
<th>Cross Section</th>
<th>Existing Factor of Safety</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-A</td>
<td>2.1</td>
<td>B1</td>
</tr>
<tr>
<td>B-B</td>
<td>1.8</td>
<td>B2</td>
</tr>
<tr>
<td>C-C</td>
<td>2.1</td>
<td>B3</td>
</tr>
<tr>
<td>D-D</td>
<td>1.7</td>
<td>B4</td>
</tr>
<tr>
<td>E-E</td>
<td>2.2</td>
<td>B5</td>
</tr>
<tr>
<td>F-F</td>
<td>1.7</td>
<td>B6</td>
</tr>
<tr>
<td>G-G</td>
<td>2.6</td>
<td>B7</td>
</tr>
</tbody>
</table>

Based on the results of the analyses, the east-west aligned slopes (i.e., Sections ‘A-A’ to ‘D-D’, inclusive), are considered stable under “worst case” conditions. The ravine side slopes (i.e., Sections ‘E-E’ to G-G’, inclusive) are considered stable under “worst case” conditions. The results of the stability analyses agree with our field observations on May 30, 2014.
5.7.4 Setback Requirements

For unstable slopes, the distance from the unstable slope to the safe setback line is called ‘Erosion Hazard Limit’. In accordance with the Ministry of Natural Resources (MNR) Technical Guide “Understanding Natural Hazards” dated 2001, the Erosion Hazard Limit consists of three components: (1) Stable Slope Allowance, (2) Toe Erosion Allowance, and (3) Erosion Access Allowance.

The Stable Slope Allowance, as described in the MNR procedures, is the area where a factor of safety of less than 1.5 against overall rotational failure is calculated. At Sections ‘A-A’ to ‘D-D’, inclusive, the slope stability analyses indicate that the existing east-west aligned slopes, in their current configurations, have a factor of safety against failure of greater than 1.5 (refer to Figures B1 to B4, inclusive, in Appendix B). Therefore, the Stable Slope Allowance described in the MNR procedures is not required. At Sections ‘E-E’ to ‘G-G’, inclusive, the slope stability analyses indicate that the existing creek slopes, in their current configurations, have a factor of safety against failure of greater than 1.5 (refer to Figures B5 to B7, inclusive, in Appendix B). Therefore, the Stable Slope Allowance described in the MNR procedures is not required.

As indicated above, active soil erosion was observed along the bottom (toe) of the ravine side slopes (i.e., Sections ‘E-E’ to ‘G-G’, inclusive). In accordance with the MNR documents, a minimum Toe Erosion Allowance of between 5.0 to 8.0 metres is required for clay soils and a minimum Erosion Allowance of between 5.0 to 15.0 metres is required for sandy soils. Given the potential for soil erosion, a Toe Erosion Allowance of 6.0 metres should be used at the location of Sections ‘E-E’ to ‘G-G’, inclusive (refer to Figures B5 to B7, inclusive, in Appendix B). The Toe Erosion Allowance is applied at the crest of the slope.

The MNR procedures also include the application of a 6 metre wide Erosion Access Allowance beyond the Toe Erosion Allowance to allow for access by equipment to repair a possible failed slope. However, based on the preliminary development plans, the Erosion Access Allowance is not required (i.e., for cases where rear lot lines of residential lots are not constructed right up to the Erosion Hazard Limit).

The east-west aligned slopes are relatively flat with no watercourse located at the toe. As such, no setback, from a slope stability perspective, is required from the east-west aligned slopes. The Erosion Hazard Limit (setback) for the ravine side slopes (i.e., Sections ‘E-E’ to ‘G-G’, inclusive) is located about 6.0 metres from the crest of the existing slopes.

It may be possible for the existing ravine to accept additional storm flows from the upstream development. However, for this case, we suggest that the channel and side slopes be protected from erosion (e.g., by installing suitably sized rip rap and a geotextile). It is noted that if the toe of the side slopes are protected from erosion, it may be possible to reduce or eliminate the 6 metre setback from the crest of the ravine side slopes.
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.

Lauren Ashe, M.A.Sc., E.I.T.

Craig Houle, M.Sc., P.Eng., Principal
APPENDIX A

Record of Borehole and Test Pit Sheets
### Record of Borehole 14-1

**Project:** 14-185  
**Location:** See Borehole Location Plan, Figure 2  
**Boring Date:** August 1, 2014

#### Soil Profile

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Type Description</th>
<th>Elevation (m)</th>
<th>Samples</th>
<th>Dynamic Penetration Resistance, Blows/0.3m</th>
<th>Hydraulic Conductivity, k cm/s</th>
<th>Additional Lab. Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Ground Surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.20</td>
<td>Dark brown sandy silt, some organic material (TOPSOIL)</td>
<td>1</td>
<td>50</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.27</td>
<td>Brown to grey brown, fine grained SAND, some silt</td>
<td>2</td>
<td>50</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.05</td>
<td>Very stiff, grey brown to reddish brown SILTY CLAY (WEATHERED CRUST)</td>
<td>3</td>
<td>50</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.05</td>
<td>Firm to soft, grey SILTY CLAY</td>
<td>4</td>
<td>50</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.10</td>
<td>End of Borehole</td>
<td>5</td>
<td>50</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Boring Method

- **Power Auger:** 200mm Diameter Diamond Drive Auger
- **Auger Cuttings & Above Ground Protector:** Bentonite
- **Bentonite:**
- **Auger Cuttings:** Bentonite
- **Filter Sand:** 50mm diameter, 1.52 m long slotted PVC pipe
- **Hydrometer:**
- **Water level in well screen at 0.85 metres below ground surface on August 12, 2014**

**Hydraulic Conductivity, k cm/s:** 80

**Additional Lab. Testing:**
- **Water Level:** 20
- **Dynamic Penetration Resistance, Blows/0.3m:** 40
- **Ground Surface:** 10
- **Soil Profile:**
  - **Cu, kPa:**
  - **V - rem. V:**
  - **U - V:**

**Boring Method:**

- **Depth Scale:** 1 to 40
- **SPT Hammer:** 63.5 kg; drop 0.76 m
- **Datum:**
  - **SPT:**
  - **Hole:**
  - **Chevrier:**
  - **2015:**

**Record of Borehole 14-1 Borehole Logs August 5 2014.GPJ**

**Houle Chevrier Engineering**

**Logged:** A.N.  
**Checked:**
# Record of Borehole 14-2

**PROJECT:** 14-185  
**LOCATION:** See Borehole Location Plan, Figure 2  
**BORING DATE:** August 1, 2014  
**SPT HAMMER:** 63.5 kg; drop 0.76 m  
**DATUM:**

<table>
<thead>
<tr>
<th>DEPTH SCALE METRES</th>
<th>BORING METHOD</th>
<th>SOIL PROFILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Power Auger</td>
<td>Ground Surface</td>
</tr>
<tr>
<td>1</td>
<td>Power Auger</td>
<td>Dark brown sandy silt, some organic material (TOPSOIL)</td>
</tr>
<tr>
<td>2</td>
<td>Power Auger</td>
<td>Brown to grey brown, fine grained SAND, some silt</td>
</tr>
<tr>
<td>3</td>
<td>Power Auger</td>
<td>Very stiff, grey brown SILTY CLAY (WEATHERED CRUST)</td>
</tr>
<tr>
<td>4</td>
<td>Power Auger</td>
<td>Firm, grey SILTY CLAY</td>
</tr>
<tr>
<td>5</td>
<td>Power Auger</td>
<td>End of Borehole</td>
</tr>
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</table>

## Soil Profile

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>TYPE</th>
<th>BLOW/SOIL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.10</td>
</tr>
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</table>

## Dynamic Penetration Resistance, Blows/0.3m

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>TYPE</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>40</td>
<td>60</td>
<td>80</td>
</tr>
</tbody>
</table>

## Shear Strength, Cu (kPa)

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>TYPE</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Hydraulic Conductivity, k, cm/s

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>TYPE</th>
<th>10^{-5}</th>
<th>10^{-4}</th>
<th>10^{-3}</th>
<th>10^{-2}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Water Content, Percent

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>TYPE</th>
<th>Wp</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Additional Lab Testing

- Soil Sample Analysis
- Geochemical Tests

## Boring Method

- Power Auger
- 20mm Diameter Hollow Stem Auger

## Borehole Log 14-185

**DATE:** August 5, 2014

**Houle Chevrier Engineering**
RECORD OF BOREHOLE 14-3

PROJECT: 14-185
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: July 31, 2014

DEPTH SCALE
1 to 110

SOIL PROFILE
DESCRIPTION

Ground Surface
Dark brown sandy silt, some organic material (TOPSOIL)
Brown, fine grained SAND, some silt, trace gravel
Very stiff, grey brown and reddish brown SILTY CLAY (WEATHERED CRUST)
Firm to stiff, grey SILTY CLAY
DCPT
DCPT refusal on inferred bedrock
End of Borehole

ELEV (m)

0.19
3.05
8.00
8.62
9.00
11.46
12.02
12.68
13.24
13.80
15.26
16.12
16.68
17.24
18.00
18.66
19.22
19.88
20.44
21.00
21.66
22.32

BLOW/0.3m

4
5
7
1
W.H.
W.H.
W.H.
W.H.

SHEAR STRENGTH Cu, kPa

0.15
3.05

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

45
90
45
90
90
45

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

Wp
W

Wp
Wp
W

Wp

W

W

W

80
20
60
80

90.50
88.62
75.21
71.65

WATER CONTENT, PERCENT

20

40
60
80

200mm Diameter Hollow Stem Auger
Dynamic Cone Penetration Test

DCPT

LOGGED: A.N.
CHECKED:
**PROJECT:** 14-185  
**LOCATION:** See Borehole Location Plan, Figure 2  
**BORING DATE:** August 1, 2014  
**DATUM:** 63.5 kg; drop 0.76 m

---

### SOIL PROFILE

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>DESCRIPTION</th>
<th>ELEV</th>
<th>DEPTH (m)</th>
<th>NUMBER</th>
<th>TYPE</th>
<th>BLOWS/0.3m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ground Surface</td>
<td>0.20</td>
<td>0</td>
<td>1</td>
<td>50 D.O.</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Dark brown sandy silt, some organic material (TOPSOIL)</td>
<td>1.52</td>
<td>1.05</td>
<td>3</td>
<td>50 D.O.</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Brown to reddish brown, fine grained SAND, trace to some silt</td>
<td></td>
<td>2.05</td>
<td>2</td>
<td>50 D.O.</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Very stiff, grey brown to reddish brown SILTY CLAY (WEATHERED CRUST)</td>
<td>3.57</td>
<td>3.57</td>
<td>4</td>
<td>50 D.O.</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Firm, grey SILTY CLAY</td>
<td>6.10</td>
<td>6.10</td>
<td>6</td>
<td>50 D.O.</td>
<td>W.H</td>
</tr>
<tr>
<td></td>
<td>End of Borehole</td>
<td></td>
<td>8.10</td>
<td>8</td>
<td></td>
<td>+</td>
</tr>
</tbody>
</table>

---

### HYDRAULIC CONDUCTIVITY, k, cm/s

<table>
<thead>
<tr>
<th>Shear Strength, Cu, kPa</th>
<th>Dynamic Penetration Resistance, BLOWS/0.3m</th>
<th>Hydraulic Conductivity, k, cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Strength, Cu, kPa</td>
<td>Dynamic Penetration Resistance, BLOWS/0.3m</td>
<td>Hydraulic Conductivity, k, cm/s</td>
</tr>
<tr>
<td>Cu</td>
<td>kPa</td>
<td>Cu</td>
</tr>
</tbody>
</table>

---

**Bentonite & Above Ground Protector**

50mm diameter, 1.52 m long slotted PVC pipe

**Filter Sand**

Water level in well screen at 2.43 metres below ground surface on August 12, 2014

---

**Figure 2** LOCATION: See Borehole Location Plan, Figure 2  
---

**Houle Chevrier Engineering**

---

**LOGGED:** A.N.  
**CHECKED:**
**RECORD OF BOREHOLE 14-5**

**PROJECT:** 14-185  
**LOCATION:** See Borehole Location Plan, Figure 2  
**BORING DATE:** July 31, 2014  
**SPT HAMMER:** 63.5 kg; drop 0.76 m

### SOIL PROFILE

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>ELEV.</th>
<th>STRAT.PLOT</th>
<th>DEPTH (m)</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Surface</td>
<td>70.84</td>
<td>0.08</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grey brown silty clay, some organic material (TOPSOIL)</td>
<td>69.01</td>
<td>1</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>Very stiff, grey brown SILTY CLAY (WEATHERED CRUST)</td>
<td>68.33</td>
<td>2</td>
<td>50</td>
<td>4</td>
</tr>
<tr>
<td>Firm to soft, grey SILTY CLAY</td>
<td></td>
<td>3</td>
<td>50</td>
<td>1</td>
</tr>
<tr>
<td>Power Auger</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200mm Diameter Hollow Stem Auger</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>End of Borehole</td>
<td>64.74</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### HYDRAULIC CONDUCTIVITY, k, cm/s

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>64.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
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<th>40</th>
<th>60</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>64.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### ADDITIONAL LAB. TESTING

- Water level in well screen at 0.83 metres below ground surface (70.01 metres) on August 12, 2014
- Filter Sand
- Bentonite
- Auger Cuttings & Above Ground Protector
- Bentonite
- Water level in well screen at 0.83 metres below ground surface (70.01 metres) on August 12, 2014
- Filter Sand
- Bentonite
- Auger Cuttings & Above Ground Protector
- Bentonite
- Water level in well screen at 0.83 metres below ground surface (70.01 metres) on August 12, 2014
- Filter Sand
- Bentonite
- Auger Cuttings & Above Ground Protector

**LOGGED:** A.N.  
**CHECKED:**
**SOIL PROFILE**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>ELEV.</th>
<th>STRATA PLOT</th>
<th>NUMBER</th>
<th>TYPE</th>
<th>DESCRIPTION</th>
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<td></td>
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</tr>
<tr>
<td>1</td>
<td>71.58</td>
<td></td>
<td>1</td>
<td>50</td>
<td>Grey brown, silty clay, trace sand, some organic material (TOPSOIL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Very stiff, grey brown SILTY CLAY (WEATHERED CRUST)</td>
</tr>
<tr>
<td>2</td>
<td>69.75</td>
<td></td>
<td>2</td>
<td>50</td>
<td>Firm, grey SILTY CLAY</td>
</tr>
<tr>
<td>3</td>
<td>66.83</td>
<td></td>
<td>3</td>
<td>50</td>
<td>Grey silty clay, some sand, some gravel (GLACIAL TILL)</td>
</tr>
<tr>
<td>4</td>
<td>66.64</td>
<td></td>
<td>4</td>
<td>50</td>
<td>Grey SILTY SAND, some gravel</td>
</tr>
<tr>
<td>5</td>
<td>65.64</td>
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<td>5</td>
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<td>End of Borehole</td>
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**SMP RESULTS**

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<th>ELEV.</th>
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<td>1</td>
<td>71.58</td>
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<tr>
<td>2</td>
<td>50</td>
<td></td>
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<td>69.75</td>
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<tr>
<td>6</td>
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**HYDRAULIC CONDUCTIVITY, \( k, \text{ cm/s} \)**

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**DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m**

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<th>60</th>
<th>80</th>
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</thead>
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<td>71.58</td>
<td>71.58</td>
<td>71.58</td>
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</table>

**SHEAR STRENGTH Cu, kPa**

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<th>60</th>
<th>80</th>
<th>10^{-5}</th>
<th>10^{-4}</th>
<th>10^{-3}</th>
<th>10^{-2}</th>
</tr>
</thead>
<tbody>
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<td>71.58</td>
<td>71.58</td>
<td>71.58</td>
<td>71.58</td>
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</tbody>
</table>

**WATER CONTENT, PERCENT**

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<th>80</th>
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<tbody>
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<td>71.58</td>
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<td>20</td>
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</tbody>
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**ADDITIONAL LAB TESTING**

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<th>10^{-4}</th>
<th>10^{-3}</th>
<th>10^{-2}</th>
</tr>
</thead>
<tbody>
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<td>71.58</td>
<td>71.58</td>
<td>71.58</td>
<td>71.58</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

**PROJECT:** 14-185

**LOCATION:** See Borehole Location Plan, Figure 2

**BORING DATE:** July 31, 2014

**SPT HAMMER:** 63.5 kg; drop 0.76 m

---

**Houle Chevrier Engineering**

**LOGGED:** A.N.

**CHECKED:**
APPENDIX B

Slope Stability Analyses
Figures B1 to B7
Loading Conditions: Static
Soil Properties: Drained

Material: Silt Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17.5 kN/m³
Cohesion: 10 kPa
Friction Angle: 30 degrees

Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Friction Angle: 32 degrees
Loading Conditions: Static
Soil Properties: Drained

SLOPE STABILITY ANALYSIS
SECTION B-B
3455 MILTON ROAD
EXISTING CONDITIONS

FIGURE B2

PROJECT: 14-185
DATE: March 2015
Loading Conditions: Static
Soil Properties: Drained

Elevation (metres, geodetic datum)

Metres

Material: Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17.5 kN/m³
Cohesion: 10 kPa
Friction Angle: 30 degrees

Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Friction Angle: 32 degrees

SLOPE STABILITY ANALYSIS
SECTION C-C
3455 MILTON ROAD
EXISTING CONDITIONS

PROJECT: 14-185
DATE: March 2015

FIGURE B3
Loading Conditions: Static
Soil Properties: Drained
Loading Conditions: Static
Soil Properties: Drained

Material: Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17.5 kN/m³
Cohesion: 10 kPa
Friction Angle: 30 degrees

Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Friction Angle: 32 degrees
Loading Conditions: Static
Soil Properties: Drained

Elevation (metres, geodetic datum)

Metres

Material: Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17.5 kN/m³
Cohesion: 10 kPa
Friction Angle: 30 degrees

Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Friction Angle: 32 degrees

SLOPE STABILITY ANALYSIS
SECTION F-F
3455 MILTON ROAD
EXISTING CONDITIONS

FIGURE B6

PROJECT: 14-185
DATE: March 2015
Loading Conditions: Static
Soil Properties: Drained

Elevation (metres, geodetic datum)

Metres

Material: Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17.5 kN/m³
Cohesion: 10 kPa
Friction Angle: 30 degrees

Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m³
Cohesion: 0 kPa
Friction Angle: 32 degrees

Erosion Hazard Limit (6 metres)
APPENDIX C

Laboratory Test Results
Grain Size Distribution and Plasticity Chart
Figures C1 to C3
Figure C1

GRAIN SIZE DISTRIBUTION

SAND

Legend

<table>
<thead>
<tr>
<th>Legend</th>
<th>Test pit</th>
<th>Sample</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>●</td>
<td>13-10</td>
<td>2</td>
<td>0.2 - 0.5</td>
</tr>
<tr>
<td>□</td>
<td>13-15</td>
<td>2</td>
<td>0.3 - 0.6</td>
</tr>
<tr>
<td>▲</td>
<td>13-17</td>
<td>2</td>
<td>0.2 - 0.5</td>
</tr>
<tr>
<td>★</td>
<td>13-17</td>
<td>4</td>
<td>2.0 - 2.3</td>
</tr>
</tbody>
</table>

Date: March 2015
Project: 14-185
GRAIN SIZE DISTRIBUTION
SILTY CLAY

Legend

<table>
<thead>
<tr>
<th>Test pit</th>
<th>Sample</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>● 14-3</td>
<td>6</td>
<td>4.6 - 5.2</td>
</tr>
<tr>
<td>□ 14-4</td>
<td>3</td>
<td>1.5 - 2.1</td>
</tr>
<tr>
<td>▲ 13-05</td>
<td>2</td>
<td>1.8 - 2.4</td>
</tr>
<tr>
<td>★ 13-08</td>
<td>3</td>
<td>1.5 - 2.4</td>
</tr>
<tr>
<td>○ 13-17</td>
<td>2</td>
<td>0.2 - 0.5</td>
</tr>
<tr>
<td>◆ 13-17</td>
<td>4</td>
<td>2.0 - 2.3</td>
</tr>
</tbody>
</table>

Date: March 2015
Project: 14-185
Group Symbol
CL = Lean Clay
ML = Silt
CH = Fat Clay
MH = Elastic Silt
CL - ML = Silty Clay
OL (Above "A" Line) = Organic Clay
OL (Below "A" Line) = Organic Silt
OH (Above "A" Line) = Organic Clay
OH (Below "A" Line) = Organic Silt

Legend

<table>
<thead>
<tr>
<th>Legend</th>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>LL %</th>
<th>PL %</th>
<th>PI %</th>
</tr>
</thead>
<tbody>
<tr>
<td>•</td>
<td>14-3</td>
<td>4</td>
<td>2.3 - 2.9</td>
<td>54.0</td>
<td>21.0</td>
<td>33.0</td>
</tr>
<tr>
<td>□</td>
<td>14-4</td>
<td>3</td>
<td>1.5 - 2.1</td>
<td>54.0</td>
<td>21.0</td>
<td>33.0</td>
</tr>
</tbody>
</table>
APPENDIX D

Chemical Testing of Groundwater Sample
Corrosion of Buried Concrete and Steel
Paracel Laboratories Order No. 1433120
geotechnical
environmental
hydrogeology
materials testing & inspection