Preliminary Foundation Investigation Geotechnical Report

Proposed Culvert Replacements and Rail Bridge Upgrade
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City of Barrie

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# Preliminary Foundation Investigation Geotechnical Report

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was commissioned by the City of Barrie to undertake a Transportation Environmental Assessment for the Salem Growth Development Area, denoted as Assignment #3.

The Assessment included a geotechnical component consisting of two parts: a geotechnical desktop review of existing documents and information for the study area and a field investigation that included boreholes at the locations of an at-grade CN Rail Crossing intended for widening/lengthening and at the locations of three (3) existing culverts that may be upgraded/replaced.

The purpose of the geotechnical component was to determine geotechnical issues that may affect the alternative design concept for the major structures being considered for implementation.

This report provides:

- The results of the field investigation and laboratory testing program;
- Preliminary geotechnical parameters and recommendations for consideration in the preliminary design process for the planned scope of improvements/development;
- Limitations and constraints (both design and construction) associated with the geotechnical information submitted in the context of the planned scope of improvements/development; and,
- Comment regarding suitable pavement structure(s) for the planned scope of improvements/development.

This report does not address any environmental aspects of the project or site conditions such as the potential presence of environmental contamination, species at risk, surface water, or related topics.

Use of this report is subject to the Statement of General Conditions provided in Appendix A.
2.0  SITE DESCRIPTION

2.1  SITE LOCATIONS

The locations of the railway overpass and the three (3) culverts are illustrated on the Key Plan in Drawing No. 1 in Appendix B. The specific locations of the culverts and railway overpass are shown on Drawing Nos. 2, 3, 4 and 5 in Appendix B.

The railway overpass on Lockhart Road is located approximately 385 m west of the intersection with Rawson Avenue and crosses over the road on a north to south orientation.

A large diameter corrugated steel pipe (CSP) passes under Huronia Road at a location approximately 35 m south of the intersection with Lockhart Road.

A double-box concrete culvert passes under Lockhart Road at a location approximately 140 m west of the intersection with Rawson Avenue.

An open-footed concrete box culvert passes under Essa Road at a location approximately 65 m south of the intersection with Athabaska Road.

2.2  SITE DESCRIPTIONS AND EXISTING INFRASTRUCTURE

2.2.1  CSP Culvert under Huronia Road south of Lockhart Road - Location of BH1

The existing culvert is a corrugated steel pipe (CSP) with a diameter of approximately 1 m. The culvert is approximately 20 m long. There are gabion baskets on both sides of the culvert. The culvert has a cover of approximately 0.5 m including the pavement structure.

At this location, Huronia Road is a two lane paved road with gravel shoulders and a speed limit of 80 km/h. The road embankment is approximately 1.5 m in height, with side-slopes of approximately 1 horizontal (H): 1 vertical (V). At the location of the culvert, there are guard rails on both sides of the road.

The land beyond the road limits is covered by trees and rough vegetation.

Land use in the area consists of a golf course to the east, west and south, industrial lands to the northwest, and residential lands to the northeast.

There is no buried or overhead infrastructure in the immediate vicinity of the culvert. However, overhead wires are present along the south side of Lockhart Road and along the east side of Huronia Road, to the north of Lockhart Road.

A photograph of the CSP is included in Appendix E for reference.
2.2.2 Double Box Culvert under Lockhart Road – Location of BH2

The existing culvert is a concrete double box, with a length of approximately 20 m. The boxes exposed on the south side of Lockhart Road each have a height of approximately 1 m and a width of approximately 3.5 m. On the north side of Lockhart Road, the outlet appears to consist of a single box with a height of approximately 1 m and a width of approximately 5 m.

The culvert has a cover of approximately 0.5 m including the pavement structure.

At this location, Lockhart Road is a two lane paved road with gravel shoulders and a speed limit of 60 km/h. The lands on the north side of the road are approximately 3 m above the elevation of the travel surface whereas the lands on the south side of the road are approximately 1.5 m below the elevation of the travel surface. Guard rails exist on both sides of the road in the vicinity of the culvert.

The land on the south side of the road is covered by mature, dense trees and rough vegetation. The land on the north side of the road is covered by sparse trees and rough vegetation.

Land use in the area consists of a golf course to the south and industrial lands to the north.

Existing infrastructure in the area includes a buried water main under the north shoulder of the road, and overhead wires along the south side of the road.

A photograph of the culvert is included in Appendix E for reference.

2.2.3 Single Span Railway Bridge – Location of BH3

The existing railway bridge is a single span structure with concrete abutments and wing walls. The span of the bridge is approximately 10 m long and 5 m wide. The wing walls extend approximately 8 m from the edge of the abutment. A report prepared by Terraprobe in 2007 for the rehabilitation of the bridge and planned installation of a water main on Lockhart Road did not include any details regarding the bridge. However, based on the content of the report and visual observation of the bridge it is inferred that either the wingwalls were added after original construction or the wingwalls were rehabilitated at some point in time. It was also inferred, based on the content of the report, that the existing abutments are founded on spread footings placed on the underlying native sandy silt soils.

The clear height from the road surface to the underside of the bridge deck is approximately 4.3 m. The slope from the road grade to the top of the bridge deck (beyond the wing walls) is approximately 1.5:1 (Horizontal:Vertical). The slopes are covered with dense rough vegetation.

At this location Lockhart Road is a two lane paved road with gravel shoulders and a speed limit of 60 km/h. The road narrows to one lane under the rail bridge. The road is approximately level with the land to the north and approximately 1 m below the prevailing grade of the land to the
south. There is a small ditch with a steady flow along the north side of the road; the ditch does not extend under the bridge.

Land use in the area consists of a residence to the northwest, industrial land to the northeast, agricultural land to the southwest, and a golf course to the southeast. The driveway to the residence is located immediately west of the bridge.

Existing infrastructure in the area includes a buried water main along the north shoulder of the road, and overhead wires along the south side of the road and to the north of the residence.

A photograph of the rail bridge is included in Appendix E for reference.

### 2.2.4 Open Footed Box Culvert under Essa Road South of Athabaska Road - Location of BH4

The existing culvert is a concrete box, with a height of approximately 1 m and a width of approximately 3 m. The culvert is approximately 20 m in length. On the east side of Essa Road there are limestone blocks extending approximately 3 m on both sides of the culvert.

The culvert has a cover of approximately 0.5 m, including the pavement structure.

At this location, Essa Road is a three lane road, with one southbound lane, one northbound lane, and one northbound left hand turning lane. There are gravel shoulders on both sides of the road and the speed limit is 60 km/h. The road embankment is approximately 1.5 m above the adjacent prevailing ground surface.

The area on the west side of the road is covered by dense trees and rough vegetation and the area on the east side of the road is covered by landscaped grass and vegetation.

Land use in the area consists of industrial lands to the east and residential lands to the west.

Existing infrastructure in the area includes a buried Bell cable approximately 7.5 m to the west of Essa Road, a buried gas main approximately 7.8 m to the west of Essa Road, and overhead wires along the west side of the road.

A photograph of the culvert is included in Appendix E for reference.

### 3.0 REGIONAL GEOLOGY

The Geotechnical Overview in Appendix F provides the details of the review of documents and subsurface information available for the study area. In brief, the conditions can be described as follows:
• The overburden stratigraphy across the Study Area typically consists of a stratum of granular soil (typically silty sand, sandy silt, and silt) characterized as loose to compact (and locally dense to very dense) underlain by a stratum of either granular or cohesive glacial till (typically silty sand with gravel to silty clay) characterized as either compact to very dense or stiff to hard.
• Groundwater was recorded at relatively shallow depth in a number of the borehole records reviewed; the depths recorded were often less than 5 m below grade.
• Bedrock was not encountered in the borehole records reviewed; the boreholes were advanced to depths in the order of 11 m to 19 m below grade).
• Several boreholes advanced in the Study Area encountered the presence of organic silt or peat.

4.0 INVESTIGATION PROCEDURES

4.1 FIELD INVESTIGATION

The field investigation consisted of advancing one (1) borehole at the location of each of the culverts and one (1) borehole at the location of the railway overpass. The boreholes were designated BH1, BH2, BH3, and BH4. The locations are shown on the Key Plan on Drawing No.1 in Appendix B and on the respective borehole location plans in the same appendix.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of public utilities.

Temporary lane closure and traffic control was provided by Direct Traffic Management Inc.

The field drilling program was carried out on November 11th and November 14th, 2016. Boreholes BH1 and BH4 were advanced with solid stem augers, and Boreholes BH2 and BH3 were advanced with hollow-stem augers. All Boreholes were completed using a truck mounted drill rig owned and operated by Terex Drilling Solutions Ltd.

The subsurface stratigraphy encountered in the boreholes was recorded in the field by Stantec personnel. Split spoon samples were collected at regularly spaced intervals (typically every 760 mm) during the course of Standard Penetration Testing (ASTM D1586). All samples recovered were returned to Stantec’s Markham laboratory for detailed classification and testing.

Groundwater measurements were carried out in open holes immediately upon completion of drilling.

The boreholes were backfilled with a mixture of the auger cuttings and granular bentonite. Cold asphalt mix was used to reinstate the pavement surface.
4.2 LOCATION AND ELEVATION SURVEY

The borehole locations were established in the field with consideration for the presence and locations of existing infrastructure. The locations are shown on Drawing Nos. 2, 3, 4 and 5 in Appendix B. The coordinates of the boreholes are provided in the Table 4.1 below.

Table 4.1 Coordinates of Boreholes

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>Northing</th>
<th>Easting</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH1</td>
<td>4909187.10</td>
<td>606904.85</td>
</tr>
<tr>
<td>BH2</td>
<td>4909064.51</td>
<td>606331.53</td>
</tr>
<tr>
<td>BH3</td>
<td>4908989.57</td>
<td>606116.74</td>
</tr>
<tr>
<td>BH4</td>
<td>4908323.17</td>
<td>602118.13</td>
</tr>
</tbody>
</table>

The ground surface elevations at the borehole locations were not determined as a component of this preliminary investigation program. The sampling depths and conditions and stratigraphy reported on the boreholes are referenced to the ground surface.

4.3 LABORATORY TESTING

All samples were transported to our Markham geotechnical and construction materials testing laboratory. The samples were subjected to a visual examination by a Geotechnical Engineer.

Samples were selected for laboratory testing that included a combination of: gradation analysis, Atterberg Limits, and moisture content testing.

The scope of the laboratory testing program is summarized in Table 4.2 below.

Table 4.2: Geotechnical Laboratory Testing Program

<table>
<thead>
<tr>
<th>Laboratory Testing</th>
<th>Moisture Content</th>
<th>Gradation Analysis</th>
<th>Atterberg Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Tests</td>
<td>31</td>
<td>6</td>
<td>3</td>
</tr>
</tbody>
</table>

Unless specific instructions are received to the contrary, the samples remaining after testing will be placed in storage for a period of one month after issue of the final report. After the storage period, the samples will be discarded.
5.0 SUBSURFACE CONDITIONS

5.1 FRAME OF REFERENCE

The soils encountered in the boreholes and reported herein have been classified in accordance with the Unified Soil Classification System as defined in ASTM D2487, with modifications consistent with the methods of the Ontario Ministry of Transportation (MTO). The modifications specifically include the removal of the descriptions “lean” and “fat” with reference to clay soils and include a “Medium” category with respect to plasticity.

It should be noted that the internal diameter (I.D.) of the SPT sampler is 38 mm and hence the grain size test results and soil classifications may not reflect the entire gravel size fraction which extends to 75 mm diameter. The presence of cobbles (particles from 75 mm to 300 mm) and boulders (particles >300 mm) were inferred to be present in particular strata and are described separately from the gravel content.

5.2 OVERVIEW

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix C.

In general, the subsurface stratigraphy consisted of:

- Organic ground surface cover and topsoil or asphalt; underlain by
- Fill materials consisting of sand and gravel to silty sand with trace gravel; underlain by
- Stiff to hard silty clay at two (2) locations or loose to very dense silty sand.

Bedrock was not encountered to the termination depth of the boreholes.

Groundwater was recorded in all boreholes on completion of drilling. Free groundwater was recorded in the open boreholes at depths ranging from 1.5 m to 3 m below grade.

5.3 GROUND SURFACE COVER

5.3.1 Organics & Topsoil

Organic ground surface cover consisting of short rough grass (on the road shoulder) with underlying topsoil was present at the location of Boreholes BH2 and BH3.

The topsoil was approximately 100 mm and 150 mm thick at these locations.
5.3.2 Asphalt

Asphalt pavement was present at the location of Borehole BH1 (located in the southbound lane of Huronia Road).

The asphalt was approximately 75 mm thick at the borehole location.

5.3.3 Fill

Sand and gravel fill material (gravel road shoulder) was present at the location of Borehole BH4. The sand and gravel fill extended to a depth of 1.8 m below the ground surface.

Fill material was encountered underlying the organics with topsoil and asphalt in Boreholes BH1, BH2 and BH3. In Boreholes BH1 the fill material consisted of sand and gravel (similar to that encountered at the location of Borehole BH4) with occasional topsoil inclusions, and extended to a depth of approximately 3.0 m below the ground surface. In Boreholes BH2 and BH3 the fill material consisted of silty sand with trace gravel and occasional topsoil inclusions, and extended to depths of approximately 3.4 m and 2.6 m, respectively.

The N-values obtained from the SPTs conducted in the fill material ranged from 3 to 22.

Based on visual and textural examination, the fill was assessed as dry to wet. The results of the moisture content tests conducted on samples of the fill ranged from approximately 8.7% to 27.5%, generally remaining below 20%.

A gradation test was completed on one (1) sample of the soil. The test results are summarized in Table 5.1 below.

Table 5.1: Grain Size Distribution – Fill Material

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH3</td>
<td>SS2</td>
<td>1.1</td>
<td>Fill: silty sand</td>
<td>8</td>
<td>58</td>
<td>21</td>
<td>13</td>
</tr>
</tbody>
</table>

The test result is shown on the borehole record in Appendix C and is illustrated on Figure 1 in Appendix D.

5.3.4 Silty Clay

A stratum of native silty clay was encountered underlying the fill materials in Boreholes BH3 and BH4. The samples of the silty clay obtained from the boreholes typically contained trace gravel and trace to some sand. This stratum extended to a depth of approximately 5.6 m and 8.7 m in Boreholes BH3 and BH4, respectively.
The N-values obtained from the SPTs conducted in this native silty clay till stratum ranged from 14 to 46. The N-values indicate a stiff to hard consistency.

The soil was assessed as moist, based on visual and textural examination of the samples in the field. Laboratory test results on samples of the soil yielded moisture contents ranging from approximately 11.2% to 25.0%.

Gradation tests were completed on two (2) samples of the soil. The test results are summarized in Table 5.2 below.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH3</td>
<td>SS5</td>
<td>3.4</td>
<td>silty CLAY (CL-ML)</td>
<td>9</td>
<td>6</td>
<td>56</td>
<td>29</td>
</tr>
<tr>
<td>BH4</td>
<td>SS5</td>
<td>3.4</td>
<td>silty CLAY (CL-ML)</td>
<td>9</td>
<td>32</td>
<td>31</td>
<td>28</td>
</tr>
</tbody>
</table>

The test results are shown on the borehole records in Appendix C and are illustrated on Figure 2 in Appendix D.

Atterberg Limits tests were also conducted the samples referenced above. The results of the tests are shown in Table 5.3 below.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH3</td>
<td>SS5</td>
<td>3.4</td>
<td>silty CLAY (CL-ML)</td>
<td>28</td>
<td>20</td>
<td>3</td>
</tr>
<tr>
<td>BH3</td>
<td>SS5</td>
<td>3.4</td>
<td>silty CLAY (CL-ML)</td>
<td>23</td>
<td>13</td>
<td>10</td>
</tr>
</tbody>
</table>

The test results are shown on the borehole record in Appendix C and are illustrated on Figure 3 in Appendix D.

5.3.5 Silty Sand

A stratum of silty sand was encountered underlying the fill materials in Boreholes BH1 and BH2, and underlying the silty clay in Boreholes BH3 and BH4. The boreholes were terminated in this stratum at depths of 9.8 m, 9.4 m, 20.4 m, and 9.6 m, respectively.

The N-values obtained from the SPTs conducted in the silty sand in Borehole BH1 ranged from 6 to 37, remaining above 18 below a depth of approximately 6 m. The N-values obtained in this borehole indicate a loose consistency in the upper portion of the stratum, transitioning to a compact consistency in the lower portion.
The N-values obtained from the SPTs conducted in the silty sand in Boreholes BH2, BH3, and BH4 were generally above 50 to a depth of approximately 15 m (e.g. in Borehole BH3). This indicates a very dense consistency. Below 15 m in Borehole BH3, the N-values obtained from the SPTs ranged from 5 to 25, indicating a loose to compact consistency.

The silty sand was assessed as moist to wet, based on visual and textural examination of the samples in the field. Laboratory test results on samples of the clay yielded moisture contents ranging from approximately 6.4% to 24% generally remaining above 10%.

Gradation tests were completed on three (3) samples of the silty sand. The test results are summarized in Table 5.4 below.

Table 5.4: Grain Size Distribution – Silty Sand

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH1</td>
<td>SS5</td>
<td>3.4</td>
<td>silty SAND (SM)</td>
<td>5</td>
<td>51</td>
<td>28</td>
<td>16</td>
</tr>
<tr>
<td>BH2</td>
<td>SS7</td>
<td>6.2</td>
<td>silty SAND (SM)</td>
<td>0</td>
<td>54</td>
<td>43</td>
<td>3</td>
</tr>
<tr>
<td>BH3</td>
<td>SS16</td>
<td>20.1</td>
<td>silty SAND (SM)</td>
<td>0</td>
<td>76</td>
<td>24</td>
<td></td>
</tr>
</tbody>
</table>

The test results are shown on the borehole records in Appendix C and are illustrated on Figure 4 in Appendix D.

An Atterberg Limits test was conducted on one (1) of the samples referenced above. The results of the test are shown in Table 5.5 below.

Table 5.5: Atterberg Limits – Silty Sand

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH1</td>
<td>SS5</td>
<td>3.4</td>
<td>silty SAND (SM)</td>
<td>13</td>
<td>10</td>
<td>3</td>
</tr>
</tbody>
</table>

The test results are shown on the borehole record in Appendix C and are illustrated on Figure 5 from Appendix D.

5.3.6 Groundwater

Free groundwater was recorded at depths of approximately 2.4 m, 1.5 m, 2.3 m and 2.4 m below grade in Boreholes BH1, BH2, BH3 and BH4, respectively, on completion of drilling. Monitoring wells were not installed as a component of the current investigation.
6.0 DESIGN RECOMMENDATIONS

6.1 STRATIGRAPHIC SUMMARY

As discussed above, a component of the overall assessment included a geotechnical desktop review. A copy of the Geotechnical Overview letter is included in Appendix F of this report for reference.

The predominant subsurface stratigraphy encountered in the boreholes advanced for the current investigation consisted of the following:

- Fill materials consisting of sand and gravel to silty sand with trace gravel; underlain by
- Either stiff to hard silty clay at two (2) locations or loose to very dense silty sand.

The conditions encountered in the boreholes was general representative of the conditions reviewed in the reports and documents as a component of the desktop study. Although organic soils and peat were noted in the desktop study, these materials were not encountered in the boreholes advanced for the current investigation.

6.2 GEOTECHNICAL MODEL

At this preliminary stage of evaluation and design, the properties provided below in Table 6.1 are recommended for consideration.

Table 6.1: Geotechnical Model & Parameters

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Design Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty Sand with Gravel Fill</td>
<td>Bulk Unit Weight = 21 kN/m³</td>
</tr>
<tr>
<td>(Pavement Structure)</td>
<td>Effective Friction Angle, $\phi = 32^\circ$ (drained)</td>
</tr>
<tr>
<td></td>
<td>$E' = 40$ MPa</td>
</tr>
<tr>
<td>Silty Sand; Sand and Gravel</td>
<td>Bulk Unit Weight = 19.5 kN/m³</td>
</tr>
<tr>
<td>(Common Fill)</td>
<td>Effective Friction Angle, $\phi = 28^\circ$ (drained)</td>
</tr>
<tr>
<td></td>
<td>$E' = 7$ MPa</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>Bulk Unit Weight = 22 kN/m³</td>
</tr>
<tr>
<td>Stiff to hard</td>
<td>Undrained Shear Strength, $S_u = 50$ kPa</td>
</tr>
<tr>
<td></td>
<td>Effective Friction Angle, $\phi = 32^\circ$ (drained)</td>
</tr>
<tr>
<td></td>
<td>$E' = 25$ MPa</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>Total Unit Weight = 21 kN/m³</td>
</tr>
<tr>
<td>Loose to Compact</td>
<td>Effective Friction Angle, $\phi = 32^\circ$ (drained)</td>
</tr>
<tr>
<td></td>
<td>$E' = 10$ MPa</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>Total Unit Weight = 21 kN/m³</td>
</tr>
<tr>
<td>Dense to Very Dense</td>
<td>Effective Friction Angle, $\phi = 32^\circ$ (drained)</td>
</tr>
<tr>
<td></td>
<td>$E' = 40$ MPa</td>
</tr>
</tbody>
</table>
The parameters provided in the table have been developed based on the soil conditions encountered (SPT N-values and the laboratory test results) and consideration of literature references for similar soils. Project-specific field or laboratory testing has not been undertaken as a component of this investigation to confirm the values presented.

6.3 CULVERTS

6.3.1 Existing Conditions

Visual observations at the time of the geotechnical investigation did not identify any obvious signs of distress to the existing culverts. There was no evidence of unusual or problematic cracking of the paved road surface above the culverts. Similarly, there was no obvious indication of settlement of the culverts or distortion of the road surface that could be construed as arising from consolidation settlement of the road embankment, the existing culverts, or the underlying soils.

6.3.2 Foundation Recommendations

Details associated with the planned replacement/upgrades to the existing culverts were not available at the time of the current investigation. For purposes of this report, it has been presumed that the existing concrete box culverts may get extended and/or new larger concrete box culverts would be installed. In the case of the CSP, it has been assumed that this will be replaced with a concrete box culvert. In consideration of the visual observations summarized in the preceding section and the conditions encountered in the boreholes, the use of conventional pre-cast box culverts (either open frame or closed box) can be considered for the locations investigated.

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2006).

In all cases it is assumed that any/all existing fill materials will be removed from below the new culverts. Subject to the design elevations for the culverts, the culverts can be placed on the prepared surface of the native silty clay or silty sand. Engineered fill can be placed on the prepared surface of the native silty clay or native silty sand to develop the design founding level if and as required. Should this be considered, the engineered fill should consist of OPSS Granular A or Granular B, Type II uniformly compacted to achieve 100% of the material's Standard Proctor Maximum Dry Density (SPMDD).

6.3.3 Geotechnical Vertical Resistance & Reaction

Given the depth of fill encountered in the boreholes and with consideration for similar size culverts to the existing, new culverts at a similar elevation to the existing would require sub-excavating the existing fill materials and placing some engineered fill. The geotechnical bearing
resistances and reactions provided below in Table 6.2 can be considered for use in preliminary
design of conventional foundations for either open-frame or closed box culverts placed on
engineered fill, native silty clay, or native silty sand.

Table 6.2: Geotechnical Resistance and Reaction for Shallow Foundation

<table>
<thead>
<tr>
<th>Culvert Type and Location</th>
<th>Embedment Depth of Foundation and Founding Strata</th>
<th>Culvert Dimension W x L (m)</th>
<th>Factored Geotechnical Resistance at ULS (kPa)</th>
<th>Geotechnical Reaction at SLS (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Huronia Road</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Open Frame</td>
<td>1.5 m</td>
<td>0.6 x 20</td>
<td>250</td>
<td>150</td>
</tr>
<tr>
<td>Closed Box</td>
<td>1.5 m</td>
<td>1.0 x 20</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>Lockhart Road</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Open Frame</td>
<td>1.5 m</td>
<td>0.6 x 20</td>
<td>250</td>
<td>200</td>
</tr>
<tr>
<td>Closed Box</td>
<td>1.5 m -</td>
<td>3.5 x 20</td>
<td>400</td>
<td>150</td>
</tr>
<tr>
<td>Essa Road</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Open Frame</td>
<td>1.5 m</td>
<td>0.6 x 20</td>
<td>250</td>
<td>200</td>
</tr>
<tr>
<td>Closed Box</td>
<td>1.5 m -</td>
<td>3.0 x 20</td>
<td>400</td>
<td>150</td>
</tr>
</tbody>
</table>

In accordance with Section 6.6.2 of the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS).

The axial reaction at SLS corresponds to a total vertical deflection (settlement) not exceeding 25 mm.

6.3.4 Geotechnical Horizontal Resistance (Sliding)

The un-factored horizontal resistance of a spread footing (the culvert box) to sliding can be calculated using the following un-factored coefficient of friction:

- 0.30 Between new engineered fill consisting of OPSS Granular A or B (Type II) and precast concrete
- 0.25 Between silty sand and precast concrete
- 40 kPa Adhesion between precast concrete and the stiff to very stiff silty clay

In accordance with Table 6.1 of the CHBDC, a resistance factor against sliding of 0.8 should be applied to obtain the resistance at ULSf.
6.4 RAIL BRIDGE

6.4.1 Existing Conditions

Information specific to the existing rail bridge was not available at the time of the current geotechnical investigation. Given the conditions encountered in the single borehole advanced at the location of the rail bridge and the existing report provided by the City of Barrie, it is inferred (though not confirmed) that the abutments are likely constructed as spread footings placed on the native silty clay, native silty sand, or possibly on engineered fill. There is a possibility that the abutments are supported on a pile foundation though this is considered somewhat less likely.

Visual observations at the time of the geotechnical investigation did not identify any obvious signs of distress in the exposed face of the abutment or the wing walls.

6.4.2 Subsurface Conditions and Geotechnical Parameters for Use in Preliminary Design

Borehole BH3 was advanced at the location of the railway bridge and Table 6.3 below outlines the soils encountered during the investigation. The groundwater level was recorded at a depth of approximately 2.3 m below grade in the open borehole on completion of drilling.

Geotechnical parameters for consideration in the preliminary evaluation and design of new abutments have been developed based on the conditions encountered in the borehole, the geotechnical model provided in a preceding section above and the parameters outlined in Section 6 of the 2006 Canadian Highway Bridge Design Code. The parameters are summarized in Table 6.3 below.

<table>
<thead>
<tr>
<th>Material/Soil Type</th>
<th>Depth Range (m)</th>
<th>Bulk Density (kN/m³)</th>
<th>Effective Friction Angle (°)</th>
<th>Undrained Shear Strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill: Silty Sand</td>
<td>0 to 2.6</td>
<td>19.5</td>
<td>28</td>
<td>N/A</td>
</tr>
<tr>
<td>Very Stiff to Hard Silty Clay</td>
<td>2.6 to 5.6</td>
<td>22</td>
<td>32</td>
<td>50</td>
</tr>
<tr>
<td>Compact to Very Dense Silty Sand</td>
<td>5.6 to 20.4</td>
<td>21</td>
<td>32</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Note: N/A Not Applicable

The undrained shear strength and friction angles provided in the table have been developed based on the soil conditions encountered (SPTN-values and the laboratory index test results) and consideration of literature references for similar soils. Project-specific field or laboratory testing has not been undertaken as a component of this investigation to confirm the values presented.
The design of the foundations for the rail bridge should be conducted in accordance with the most recent AREMA standards.

### 6.4.3 Foundation Options

Details associated with the likely configuration of the extended/expanded rail bridge were not available at the time of preparation of this report. It has therefore been assumed that the extended/expanded bridge will be a single span structure. In consideration of the conditions encountered in the single borehole advanced for the current investigation, it is anticipated that the use of conventional spread footings will be satisfactory for the support of the abutments. An alternative for the use of a pile foundation system can be considered if the design requirements warrant. A brief summary of foundation options is provided in Table 6.4 below for reference general; the table includes a description of relevant advantages and disadvantages to the options described and an indication of the inferred relative cost and risk/consequence.

**Table 6.4: Foundation Options for Proposed Bridge**

<table>
<thead>
<tr>
<th>Option</th>
<th>Advantages</th>
<th>Disadvantages</th>
<th>Relative Cost</th>
<th>Risk/Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow foundation on engineered fill</td>
<td>Limited excavation involved</td>
<td>May necessitate large footing area</td>
<td>Low to Medium</td>
<td>Potential differential settlement</td>
</tr>
<tr>
<td>Shallow foundation on native silty clay</td>
<td>Limited excavation involved</td>
<td>May necessitate large footing area</td>
<td>Low to Medium</td>
<td>Potential differential settlement</td>
</tr>
<tr>
<td>Piles driven to practical refusal in very dense till soils</td>
<td>Reduces risk of differential settlement Suitable for integral abutment</td>
<td>Possible cobbles/boulders in till leading to difficult pile driving Possible penetration through very dense layer and hence lower capacity</td>
<td>Medium to High</td>
<td>Possible pile damage during installation; Possible lower capacity</td>
</tr>
</tbody>
</table>

### 6.4.4 Conventional Spread Footing Foundations

#### 6.4.4.1 Founding Stratum

Given the conditions encountered the borehole, it is anticipated that conventional spread footings for abutments would be founded in the native compact to very dense silty sand soil.

#### 6.4.4.2 Geotechnical Vertical Resistance & Reaction

The geotechnical resistances and reactions provided in Table 6.5 below may be used in the preliminary design of the conventional spread footings for the abutments if placed on the native very stiff to hard silty clay soil or on engineered fill. For preliminary design purposes, Stantec has assumed a foundation in the order of 3.0 m x 6.0 m.
Table 6.5: Geotechnical Resistance and Reaction for Shallow Foundation

<table>
<thead>
<tr>
<th>Element and Founding Strata</th>
<th>Foundation Dimension Width x Length (m)</th>
<th>Factored Geotechnical Resistance at ULSf (kPa)</th>
<th>Geotechnical Reaction at SLS (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railway Bridge Abutment founded at 1.5 m on engineered fill</td>
<td>3.0 x 6.0</td>
<td>450</td>
<td>225</td>
</tr>
<tr>
<td>Railway Bridge Abutment founded at 3.0 m on native very stiff to hard silty clay</td>
<td>3.0 x 6.0</td>
<td>450</td>
<td>275</td>
</tr>
</tbody>
</table>

In accordance with Section 6.6.2 of the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS).

The axial reaction at SLS should be considered as net value and corresponds to a total vertical deflection (settlement) not exceeding 25 mm.

6.4.4.3 Geotechnical Horizontal Resistance (Sliding)

The unfactored horizontal resistance of a spread footing can be calculated using the following unfactored coefficients of friction:

- 0.45 Between new engineered fill consisting of OPSS Granular A or B (Type II) and cast-in-place concrete
- 50 kPa Adhesion between concrete and the very stiff to hard silty clay material

In accordance with Table 6.1 of the CHBDC, a resistance factor against sliding of 0.8 should be applied to obtain the resistance at ULSf.

6.4.5 Pile Foundation

6.4.5.1 Founding Conditions

Although the use of conventional spread footings would likely be the preferred approach, consideration could be given to the use of a pile foundation system. For general consideration and frame of reference, the following sections provide guidance and preliminary recommendations for a pile foundation system.

Relatively short piles could be founded in the underlying native very dense silty sand soil. The very dense zone was encountered at a depth interval of approximately 6 m to 14 m below grade. Based on this, for purposes of this preliminary assessment, it is anticipated that piles could be founded at a depth in the order of 10 m below grade (measured from existing grade).
6.4.5.2 Geotechnical Axial Resistance

For purposes of this report, consideration has been given to the use of HP310 x 110 piles. As referenced in the preceding section, the piles would be advanced to relatively shallow depth, in the order of 10 m below grade, to be founded in the very dense zone of the underlying native silty sand soil.

For purposes of preliminary evaluation of the foundation component of new abutments and based on the conditions described, a factored axial resistance in compression of 800 kN at Ultimate Limits State (ULS) can be considered for an HP310x110 pile.

The ULS value provided above includes a geotechnical resistance factor of 0.4 in accordance with Table 8.1, CFEM.

Assuming the pile would be driven to practical refusal in the very dense native silty sand soil, the ULS condition would govern the design and therefore an axial reaction at Serviceability Limits States (SLS) would not be applicable. This being the case, the magnitude of settlement for a HP 310 X 110 founded in the very dense silty sand soil is estimated to be less than 10 mm.

6.4.5.3 Design and Construction Considerations

The potential presence of cobbles and boulders in the sandy silty clay till and sandy silt till and the very dense condition of these soils may pose difficulties in driving piles. This should be evaluated at the time of detailed design.

If a pile foundation is ultimately adopted, it is recommended that the capacities of the piles be confirmed in the field using a Pile Driving Analyzer (PDA).

It is typical to include an allowance in the construction contract for the length (depth) of pile installation, as it is reasonable to anticipate some variation in the installation depth reflecting variability in the subsurface conditions.

6.5 TEMPORARY PROTECTION SYSTEMS

6.5.1 Excavation Geometry & Consideration of Protection Systems

With respect to new culverts or the foundations for the rail bridge abutments, if sub-excavation of the existing fill and replacement with engineered fill is contemplated or if localized removal of fill materials containing organic inclusions or deleterious materials is required, the extent of the required excavation may be considerable. For reference, this could conceivably require an excavation depth in the order of 3.4 m below existing grade (the latter depth consistent with the deepest extent of the existing fill encountered in the borehole). Temporary roadway protection would likely be required for this purpose.
Table 6.6 below provides an overview of the available roadway protection options.

### Table 6.6: Comparison of Roadway Protection Systems

<table>
<thead>
<tr>
<th>Option</th>
<th>Advantages</th>
<th>Disadvantages</th>
<th>Relative Cost</th>
<th>Risk &amp; Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench Box</td>
<td>• Simple and quick installation procedure</td>
<td>Potential for caving prior to installation</td>
<td>Low</td>
<td>• Disturbance, horizontal displacement and cracks in road (high risk)</td>
</tr>
<tr>
<td></td>
<td>• Ease of handling and movement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H-Piles with timber lagging (struts/rakers/tiebacks not anticipated to be required)</td>
<td>• Relatively simple installation procedure</td>
<td>Time consuming installation</td>
<td>Medium to High</td>
<td>• Disturbance beyond excavation (low to moderate risk)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Requires additional equipment and space</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Relatively difficult to install/remove in hard soil with cobbles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cantilevered Steel sheet pile (SSP); (rakers/tieback anchors or internal bracing not anticipated to be required)</td>
<td>• Minimum disturbance beyond excavation</td>
<td>Time consuming installation</td>
<td>Medium to High</td>
<td>• Damage to sheet piles during installation</td>
</tr>
<tr>
<td></td>
<td>• Minimum unwatering required during and after installation</td>
<td>Difficult to install in hard soil with cobbles Must be removed post-construction</td>
<td></td>
<td>• Disturbance beyond excavation (low to moderate risk)</td>
</tr>
</tbody>
</table>

#### 6.5.2 Design and Construction Considerations

The roadway protection system must meet the requirements of OPSS 539, including developing and/or defining and using the appropriate geotechnical design parameters.

The design of the temporary shoring system should meet the requirements of Performance Level 2 in accordance with OPSS 539. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm.

Given that the anticipated excavation depth is likely to be in the order of 2 m to 3.5 m and will likely encounter groundwater (further comment with respect to the depth to groundwater is provide below) the use of more rigorous system than a trench box, such as a soldier pile wall or sheet pile wall may be required.

Given the limited depth of the excavation required, it may be possible to design the temporary shoring system without the use of rakers or tiebacks. A cantilevered (unbraced) retaining system could be considered provided that it meets the requirements of the Performance Level referenced above. If this is not practical, then the design must incorporate the use of rakers or tiebacks to provide the required horizontal support.
The presence of cobbles and/or boulders was not inferred or confirmed during the drilling program. However, the nature of glacial till soils is such that the presence of cobbles and boulders is possible. The presence of cobbles and/or boulders can result in damage to sheet piles and could obstruct installation of sheet piles or present difficulties to the pre-augering of holes for the installation of H-piles for a soldier pile shoring method.

Consistent with OPSS 539, horizontal movement of the ground surface and/or the temporary shoring system should be monitored throughout the construction process. OPSS 539 provides additional guidance in this respect.

On completion of drilling groundwater was recorded in Boreholes BH1, BH2 and BH4 at depths of approximately 2.4 m, 1.5 m and 3.0 m below grade. Given the anticipated depth of excavations required for the culverts and/or conventional foundations

The anticipated required excavation for the culvert replacements will be in the order of approximately 1.8 m to 3.4 m, translating to a maximum of approximately 1.9 m below the highest groundwater level recorded. Groundwater was also recorded at a depth of approximately 2.3 m in borehole BH3 advanced for the railway bridge abutment. Should a shallow foundation option be selected for the bridge abutment, penetration below the static groundwater table is also anticipated for this location. As such, low to moderate seepage into the excavations should be anticipated. Unwatering of the open excavation will likely be required and should be manageable through the use of sump pits and contractor’s pumps. The presence, management, handling, treatment, and discharge of surface water is beyond the scope of this geotechnical report.

6.6 LATERAL EARTH PRESSURES

6.6.1 Lateral Earth Pressures under Static Conditions

Computation of lateral earth pressures should be conducted in accordance with Section 6.9 of the CHBDC. For the shallow foundations, the at-rest earth pressure should be used in design.

For the protection system (temporary shoring system) the distribution of earth pressures can be estimated using the methods available in the Canadian Foundation Engineering Manual (CFEM). For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design.

The unfactored soil parameters provided below in Table 6.7 may be used for design of the culvert box, shallow foundations for the bridge abutment and for the protection systems, assuming a horizontal backfill condition.

Where appropriate, the effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC. In addition, and where
appropriate, the design should consider a traffic live load in accordance with Clause 6.9.5 in the CHBDC.

### Table 6.7: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>OPSS Gran A and Gran B Type II</th>
<th>Sand with Gravel Fill</th>
<th>Silty Sand Fill</th>
<th>Native Silty Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Unit Weight, $\gamma$ (kN/m$^3$)</td>
<td>22</td>
<td>21</td>
<td>19.5</td>
<td>22</td>
</tr>
<tr>
<td>Effective Friction Angle</td>
<td>$35^\circ$</td>
<td>$32^\circ$</td>
<td>$28^\circ$</td>
<td>$32^\circ$</td>
</tr>
<tr>
<td>Coefficient of Earth Pressure at Rest ($K_r$)</td>
<td>0.43</td>
<td>0.47</td>
<td>0.53</td>
<td>0.47</td>
</tr>
<tr>
<td>Coefficient of Active Earth Pressure ($K_a$)</td>
<td>0.27</td>
<td>0.31</td>
<td>0.36</td>
<td>0.31</td>
</tr>
<tr>
<td>Coefficient of Passive Earth Pressure ($K_p$)</td>
<td>3.7</td>
<td>3.3</td>
<td>2.3</td>
<td>3.3</td>
</tr>
</tbody>
</table>

#### 6.6.2 Lateral Earth Pressures under Seismic Conditions

Given the location of the project in Southern Ontario, a low PGA will likely apply with respect to lateral earth pressures due to seismic loads. As a result, the lateral earth pressures due to seismic loads are anticipated to be small. The final design should consider the earth pressures induced under seismic loading conditions.

### 6.7 EMBANKMENTS

#### 6.7.1 General Requirements

A change to the existing road profiles is not contemplated as a component of the culvert replacement or the expansion/addition to the rail bridge. However, some minor widening of the existing road embankments may be required to accommodate construction staging or extensions to the existing culverts or rail bridge.

Embankment slopes should be constructed at no steeper than 2H:1V. Where widening of the embankment is required, the existing slopes should be benched in accordance with OPSD 208.010. New fill materials required for widening of the embankment should consist of OPSS Select Subgrade Material (or better) placed in 300 mm thick lifts compacted to at least 95% Standard Proctor Maximum Dry Density.

#### 6.7.2 Stability of Slopes

No sign of embankment instability was observed in the road embankments at the time of the current investigation.

Provided that the road embankments are constructed in accordance with the recommendations in the preceding section, side slopes at 2H:1V or flatter should be stable with respect to deep seated global stability failure.
Surficial and toe failures could occur if proper erosion protection and control is not provided for any area of embankment widening.

### 6.7.3 Embankment Settlement

As referenced above, the profile and footprint of the existing embankment is not anticipated to be significantly altered. As a result, consolidation settlement of the underlying native soils should not be a concern.

With respect to new fill material placed for widening of the embankment, consolidation under its own weight is anticipated to be less than 25 mm provided the embankment material is placed as engineered fill. The bulk of this settlement should occur during the period of construction.

Silt fences and erosion control should be provided throughout the duration of the construction.

Vegetation of the finished slopes (road embankment or side-slopes of new abutments where wing walls are not constructed) should be established as soon as possible after completion of earthworks construction to control surficial erosion.

A clay seal should be provided at the inlet and outlet of the culverts to prevent seepage through the culvert backfill material. The design and construction of the clay seal should meet the following criteria:

- Clay material should meet the requirements of OPSS 1205.
- Be at least 0.6 m thick.
- Extend from 0.3 m above the high water level to the full depth of excavation.
- Should not be located immediately beneath the pavement structure.

The exposed slope within 3 m of the culvert inlet and outlet should be surfaced with rip-rap. The rip rap should be a minimum of 300 mm thick and placed on a Class II non-woven filter fabric securely anchored to the underlying sub-grade soils. The size of the rip rap shall be as specified in Table 8 of OPSS 1004.

### 6.8 Construction Staging

The culvert replacements are anticipated to involve staged construction procedures. This will involve the closure of one lane at a time using appropriate traffic control.

### 6.9 Excavation, Bedding and Backfilling

Excavation and backfill for the new culvert should be carried out in accordance with:

- OPSS 902
- OPSS 803.010
Side slopes for open cut excavations (if and where required) should conform to Occupational Health & Safety Act & Regulations (OH&S Act) for Construction Projects. The existing fill materials above the static groundwater table (silty sand and sand with gravel fill) encountered in boreholes should be classified as Type 3 Soil. The excavation walls should be sloped from the base of the excavation at a slope not steeper than 1H:1V.

Below the static groundwater table, the native stiff to hard silty clay soil can also be classified as Type 3 soil. The fill material and silty sand soils below the water table should be classified as Type 4 soils. The side slopes in unsupported excavations in Type 4 materials must not be steeper than 3:1 (Horizontal: Vertical) in accordance with the OH&S Act.

Bedding and backfill material for the culvert replacements should be provided in accordance with the OPSD specification. The bedding material should consist of OPSS Granular A. The backfill material adjacent the culvert should consist of free-draining, non-frost susceptible granular material such as OPSS Granular A or Granular B Type II.

The bedding and backfill material should be placed in 300 mm thick lifts compacted to at least 95% SPMDD.

OPSD 3090.101 indicates that the frost penetration depth at the site is 1.2 m. Provided that the existing depth of cover over the culverts is maintained (currently in the order of 0.5 m) a frost taper will be required in accordance with OPSD 803.010.

6.10 UNWATERING

The conditions encountered in the boreholes indicated variable conditions with respect to the presence of groundwater. Groundwater was recorded in Boreholes BH1, BH2, BH3 and BH4 at depths of approximately 2.4 m, 1.5 m, 2.3 m and 3.0 m below grade respectively in the boreholes on completion of drilling. The anticipated required excavation for the culvert replacements and bridge abutments will be in the order of up to approximately 3.4 m, translating to approximately 1.9 m below the highest groundwater level recorded. Groundwater was also recorded at a depth of approximately 2.3 m in borehole BH3 advanced for the railway bridge abutment. Should a shallow foundation option be selected for the bridge abutment, penetration below the static groundwater table is also anticipated for this location. As such, low to moderate seepage into the excavations should be anticipated. Unwatering of the open excavation will likely be required and should be manageable through the use of sump pits and contractor’s pumps. The presence, management, handling, treatment, and discharge of surface water is beyond the scope of this geotechnical report.

For general reference where excavations encounter groundwater, the groundwater level should be lowered to at least 0.5 m below the bottom of the excavation to provide a stable base during placement of culvert bedding material.
6.11 PAVEMENTS

All four of the boreholes encountered fill materials underlying the ground surface cover. The fill materials consisted predominantly of silty sand with gravel in some proportion. The SPTs conducted in the fill materials yielded N-values ranging from 3 to 18. This variation is representative of the nature and condition of the fill materials present and presumably reflects the conditions (lift thickness and compaction effort) under which the material was placed at the time of original construction. The conditions recorded, in combination with the observed presence of topsoil inclusions and wood debris, indicates the quality control at the time of original construction may not have been consistent with placement of the fill material as “engineered fill”.

It must be anticipated that future improvements and/or development will require proof rolling and compacting the exposed granular surface of the existing roadway(s) as a minimum. In some cases, it is anticipated that localized sub-excavation and re-compaction may be required to provide uniform support to new pavements.

The Multi-Modal Active Transportation Master Plan (MMATMP) provided a detailed commentary on pavements intended for Local Roads, Collector Roads, and Arterial Roads. A summary of the thicknesses recommended for the various designs is provided in Table 6.8 below.
Table 6.8: Summary of Pavement Design/Thickness Recommendations in MMATMP

<table>
<thead>
<tr>
<th>Material</th>
<th>City of Barrie Standards</th>
<th>Recommended Design Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Local Roads</td>
<td></td>
</tr>
<tr>
<td></td>
<td>AADT &lt; 4000</td>
<td>2500 AADT</td>
</tr>
<tr>
<td>HL3 Asphalt</td>
<td>40 mm</td>
<td>40 mm</td>
</tr>
<tr>
<td>HL4 or HL8 Asphalt</td>
<td>70 mm</td>
<td>70 mm</td>
</tr>
<tr>
<td>Granular A</td>
<td>150 mm</td>
<td>150 mm</td>
</tr>
<tr>
<td>Granular B</td>
<td>350 mm</td>
<td>450 mm</td>
</tr>
</tbody>
</table>

|                 | Collector Roads          |                               |
|                 | 4000 – 1000 AADT         | 10000 – 20000 AADT            | 7500 AADT                     | 9900 AADT                     | 15000 AADT                    | 19900 AADT                    |
| HL3             | 40 mm                    | 40 mm                         | 40 mm                         | 40 mm                         | 40 mm                         | 40 mm                         |
| HL4 or HL8 Asphalt | 100 mm                  | 100 mm                        | 100 mm                        | 100 mm                        | 100 mm                        | 100 mm                        |
| Granular A      | 150 mm                   | 150 mm                        | 150 mm                        | 150 mm                        | 150 mm                        | 150 mm                        |
| Granular B      | 400 mm                   | 500 mm                        | 540 mm                        | 590 mm                        | 650 mm                        | 690 mm                        |

|                 | Arterial Roads           |                               |
|                 | 20000 – 30000 AADT       | 25000 AADT                    | 29900 AADT                    | 35000 AADT                    | 39900 AADT                    |
| HL3 Asphalt     | 40 mm                    | 40 mm                         | 40 mm                         | 40 mm                         | 40 mm                         |
| HL4 or HL8 Asphalt | 100 mm                  | 100 mm                        | 100 mm                        | 100 mm                        | 100 mm                        |
| Granular A      | 150 mm                   | 150 mm                        | 150 mm                        | 150 mm                        | 150 mm                        |
| Granular B      | 500 mm – 600 mm          | 1070 mm                       | 1120 mm                       | 1130 mm                       | 1150 mm                       |

Notes:

1. The designs provided in the MMATMP included alternatives for the material thicknesses. For purposes of this report, the initial option for each of the four AADTs has been included herein. Reference is made to the MMATMP for the alternatives.

As the values in the table indicate, the recommended thickness for the Granular B in the designs included in the MMATMP typically exceed the thickness for the Granular B in the City of Barrie Standards.

Given the details included in the MMATMP with respect to the analysis and design of the pavements, and the conditions encountered in the boreholes advanced for the current project, it is recommended that preliminary design consider the recommended designs included in the MMATMP for the respective road designations as indicated.
7.0 SPECIFICATIONS

The following specifications are referenced in this report:

**Table 7.1: Specifications Referenced in Report**

<table>
<thead>
<tr>
<th>Document</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPSD 3090.101</td>
<td>Foundation, Frost Depths for Southern Ontario</td>
</tr>
<tr>
<td>OPSS 539</td>
<td>Construction Specification for Temporary Protection System</td>
</tr>
<tr>
<td>OPSS 902</td>
<td>Construction Specification for Excavation and Backfilling - Structures</td>
</tr>
<tr>
<td>OPSD 803.010</td>
<td>Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m</td>
</tr>
<tr>
<td>OPSS 1205</td>
<td>Material Specification for Clay Seal</td>
</tr>
<tr>
<td>OPSS 511</td>
<td>Construction Specification for Rip Rap, Rock Protection and Granular Sheeting</td>
</tr>
<tr>
<td>OPSS 1004</td>
<td>Material Specification for Aggregates - Miscellaneous</td>
</tr>
</tbody>
</table>

8.0 REFERENCES


9.0 CLOSURE

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

This report was prepared by Adam Hatch, and reviewed by Khashayar Refahi and John J. Brisbois.

Respectfully submitted,

STANTEC CONSULTING LTD.

Prepared by ____________________________

Adam Hatch, P.Eng.
Geotechnical Engineer

Reviewed by ____________________________

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Geotechnical Engineer

Reviewed by ____________________________

Principal – Geotechnical Engineering

Stantec
Appendix A

A.1 STATEMENT OF GENERAL CONDITIONS
STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or subsurface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc.), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.
Appendix B

B.1 DRAWING NO. 1 – KEY PLAN ILLUSTRATING INFRASTRUCTURE AND BOREHOLE LOCATIONS

B.2 DRAWING NO. 2 – LOCATION OF BOREHOLE BH1 – HURONIA ROAD CULVERT

B.3 DRAWING NO. 3 – LOCATION OF BOREHOLE BH2 – LOCKHART ROAD CULVERT

B.4 DRAWING NO. 4 – LOCATION OF BOREHOLE BH3 – RAILWAY BRIDGE

B.5 DRAWING NO. 5 – LOCATION OF BOREHOLE BH4 – ESSA ROAD CULVERT
KEY PLAN ILLUSTRATING INFRASTRUCTURE AND BOREHOLE LOCATIONS FOR PRELIMINARY GEOTECHNICAL INVESTIGATION

- Culvert under Essa Road south of Athabaska Road (BH4)
- Railway bridge over Lockhart Road (BH3)
- Culvert under Lockhart Road (BH2)
- Culvert under Meltona Road south of Lockhart Road (BH1)
- Salem Secondary Boundary
- Existing Property Boundary
- CULVERT UNDER ESSA ROAD SOUTH OF ATHABASKA ROAD (BH4)
- RAILWAY BRIDGE OVER LOCKHART ROAD (BH3)
- CULVERT UNDER LOCKHART ROAD (BH2)
- CULVERT UNDER MELTONA ROAD SOUTH OF LOCKHART ROAD (BH1)
- SALEM SECONDARY BOUNDARY
- EXISTING PROPERTY BOUNDARY

M1 (27m ROW)
M2 (34m ROW)
BY OTHERS (MMM)

V1 (34m ROW)
V2 (34m ROW)

H1 (27m ROW)
M3 (34m ROW)
BY OTHERS (MMM)

L1 (34m ROW)
S1 (27m ROW)
E1 (27m ROW)

DECEMBER 2016
PROJECT NO. 165011003
CITY OF BARRIE
TRANSPORTATION ENVIRONMENTAL ASSESSMENT
HEWITT GROWTH DEVELOPMENT AREA - ASSIGNMENT #3

Stantec
300-575 Cochrane Drive West Tower
Markham, ON, Canada, L3R 0B8
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Appendix C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS
SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rootmat</td>
<td>vegetation, roots and moss with organic matter and soil topsoil typically forming a mattress at the ground surface</td>
</tr>
<tr>
<td>Topsoil</td>
<td>mixture of soil and humus capable of supporting vegetative growth</td>
</tr>
<tr>
<td>Peat</td>
<td>mixture of visible and invisible fragments of decayed organic matter</td>
</tr>
<tr>
<td>Till</td>
<td>unstratified glacial deposit which may range from clay to boulders</td>
</tr>
<tr>
<td>Fill</td>
<td>material below the surface identified as placed by humans (excluding buried services)</td>
</tr>
</tbody>
</table>

Terminology describing soil structure:

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Desiccated</td>
<td>having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.</td>
</tr>
<tr>
<td>Fissured</td>
<td>having cracks, and hence a blocky structure</td>
</tr>
<tr>
<td>Varved</td>
<td>composed of regular alternating layers of silt and clay</td>
</tr>
<tr>
<td>Stratified</td>
<td>composed of alternating successions of different soil types, e.g. silt and sand</td>
</tr>
<tr>
<td>Layer</td>
<td>&gt; 75 mm in thickness</td>
</tr>
<tr>
<td>Seam</td>
<td>2 mm to 75 mm in thickness</td>
</tr>
<tr>
<td>Parting</td>
<td>&lt; 2 mm in thickness</td>
</tr>
</tbody>
</table>

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<table>
<thead>
<tr>
<th>Proportion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace, or occasional</td>
<td>Less than 10%</td>
</tr>
<tr>
<td>Some</td>
<td>10-20%</td>
</tr>
<tr>
<td>Frequent</td>
<td>&gt; 20%</td>
</tr>
</tbody>
</table>

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly “relative density”), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

<table>
<thead>
<tr>
<th>Compactness Condition</th>
<th>SPT N-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>&lt;4</td>
</tr>
<tr>
<td>Loose</td>
<td>4-10</td>
</tr>
<tr>
<td>Compact</td>
<td>10-30</td>
</tr>
<tr>
<td>Dense</td>
<td>30-50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt;50</td>
</tr>
</tbody>
</table>

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by in situ vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Undrained Shear Strength (kips/sq. ft.)</th>
<th>Undrained Shear Strength (kPa)</th>
<th>Approximate SPT N-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt;0.25</td>
<td>&lt;12.5</td>
<td>&lt;2</td>
</tr>
<tr>
<td>Soft</td>
<td>0.25 - 0.5</td>
<td>12.5 - 25</td>
<td>2-4</td>
</tr>
<tr>
<td>Firm</td>
<td>0.5 - 1.0</td>
<td>25 - 50</td>
<td>4-8</td>
</tr>
<tr>
<td>Stiff</td>
<td>1.0 - 4.0</td>
<td>50 - 100</td>
<td>8-15</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>&gt;4.0</td>
<td>&gt;200</td>
<td>&gt;30</td>
</tr>
</tbody>
</table>
ROCK DESCRIPTION


Terminology describing rock quality:

<table>
<thead>
<tr>
<th>RQD</th>
<th>Rock Mass Quality</th>
<th>Alternate (Colloquial) Rock Mass Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-25</td>
<td>Very Poor Quality</td>
<td>Very Severely Fractured</td>
</tr>
<tr>
<td>25-50</td>
<td>Poor Quality</td>
<td>Severely Fractured</td>
</tr>
<tr>
<td>50-75</td>
<td>Fair Quality</td>
<td>Fractured</td>
</tr>
<tr>
<td>75-90</td>
<td>Good Quality</td>
<td>Moderately Jointed</td>
</tr>
<tr>
<td>90-100</td>
<td>Excellent Quality</td>
<td>Intact</td>
</tr>
</tbody>
</table>

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

<table>
<thead>
<tr>
<th>Spacing (mm)</th>
<th>Discontinuities</th>
<th>Bedding</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;6000</td>
<td>Extremely Wide</td>
<td>-</td>
</tr>
<tr>
<td>2000-6000</td>
<td>Very Wide</td>
<td></td>
</tr>
<tr>
<td>600-2000</td>
<td>Wide</td>
<td>Thick</td>
</tr>
<tr>
<td>200-600</td>
<td>Moderate</td>
<td>Medium</td>
</tr>
<tr>
<td>60-200</td>
<td>Close</td>
<td>Thin</td>
</tr>
<tr>
<td>20-60</td>
<td>Very Close</td>
<td>Very Thin</td>
</tr>
<tr>
<td>&lt;20</td>
<td>Extremely Close</td>
<td></td>
</tr>
<tr>
<td>&lt;6</td>
<td>-</td>
<td>Thinly Laminated</td>
</tr>
</tbody>
</table>

Terminology describing rock strength:

<table>
<thead>
<tr>
<th>Strength Classification</th>
<th>Grade</th>
<th>Unconfined Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Weak</td>
<td>R0</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Very Weak</td>
<td>R1</td>
<td>1 – 5</td>
</tr>
<tr>
<td>Weak</td>
<td>R2</td>
<td>5 – 25</td>
</tr>
<tr>
<td>Medium Strong</td>
<td>R3</td>
<td>25 – 50</td>
</tr>
<tr>
<td>Strong</td>
<td>R4</td>
<td>50 – 100</td>
</tr>
<tr>
<td>Very Strong</td>
<td>R5</td>
<td>100 – 250</td>
</tr>
<tr>
<td>Extremely Strong</td>
<td>R6</td>
<td>&gt;250</td>
</tr>
</tbody>
</table>

Terminology describing rock weathering:

<table>
<thead>
<tr>
<th>Term</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh</td>
<td>W1</td>
<td>No visible signs of rock weathering. Slight discoloration along major</td>
</tr>
<tr>
<td></td>
<td></td>
<td>discontinuities</td>
</tr>
<tr>
<td>Slightly</td>
<td>W2</td>
<td>Discoloration indicates weathering of rock on discontinuity surfaces. All</td>
</tr>
<tr>
<td></td>
<td></td>
<td>the rock material may be discolored.</td>
</tr>
<tr>
<td>Moderately</td>
<td>W3</td>
<td>Less than half the rock is decomposed and/or disintegrated into soil.</td>
</tr>
<tr>
<td>Highly</td>
<td>W4</td>
<td>More than half the rock is decomposed and/or disintegrated into soil.</td>
</tr>
<tr>
<td>Completely</td>
<td>W5</td>
<td>All the rock material is decomposed and/or disintegrated into soil.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>The original mass structure is still largely intact.</td>
</tr>
<tr>
<td>Residual Soil</td>
<td>W6</td>
<td>All the rock converted to soil. Structure and fabric destroyed.</td>
</tr>
</tbody>
</table>
STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

- Boulders
- Cobbles
- Gravel
- Sand
- Silt
- Clay
- Organics
- Asphalt
- Concrete
- Fill
- Igneous Bedrock
- Metamorphic Bedrock
- Sedimentary Bedrock

SAMPLE TYPE

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>Split spoon sample (obtained by performing the Standard Penetration Test)</td>
</tr>
<tr>
<td>ST</td>
<td>Shelby tube or thin wall tube</td>
</tr>
<tr>
<td>DP</td>
<td>Direct-Push sample (small diameter tube sampler hydraulically advanced)</td>
</tr>
<tr>
<td>PS</td>
<td>Piston sample</td>
</tr>
<tr>
<td>BS</td>
<td>Bulk sample</td>
</tr>
<tr>
<td>HQ, NQ, BQ, etc.</td>
<td>Rock core samples obtained with the use of standard size diamond coring bits.</td>
</tr>
</tbody>
</table>

WATER LEVEL MEASUREMENT

- measured in standpipe, piezometer, or well
- inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to ‘A’ size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>Sieve analysis</td>
</tr>
<tr>
<td>H</td>
<td>Hydrometer analysis</td>
</tr>
<tr>
<td>k</td>
<td>Laboratory permeability</td>
</tr>
<tr>
<td>γ</td>
<td>Unit weight</td>
</tr>
<tr>
<td>Gs</td>
<td>Specific gravity of soil particles</td>
</tr>
<tr>
<td>CD</td>
<td>Consolidated drained triaxial</td>
</tr>
<tr>
<td>CU</td>
<td>Consolidated undrained triaxial with pore pressure measurements</td>
</tr>
<tr>
<td>UU</td>
<td>Unconsolidated undrained triaxial</td>
</tr>
<tr>
<td>Ds</td>
<td>Direct Shear</td>
</tr>
<tr>
<td>C</td>
<td>Consolidation</td>
</tr>
<tr>
<td>Qc</td>
<td>Unconfined compression</td>
</tr>
<tr>
<td>Ip</td>
<td>Point Load Index (Ip on Borehole Record equals Ip(50) in which the index is corrected to a reference diameter of 50 mm)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Description</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Single packer permeability test; test interval from depth shown to bottom of borehole</td>
<td>Double packer permeability test; test interval as indicated</td>
</tr>
<tr>
<td>Falling head permeability test using casing</td>
<td>Falling head permeability test using well point or piezometer</td>
</tr>
</tbody>
</table>
### BOREHOLE RECORD

**BH 1**

| DEPTH (m) | STRATA DESCRIPTION | WATER LEVEL
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1</td>
<td>Paved Road</td>
<td>WATER LEVEL</td>
</tr>
<tr>
<td></td>
<td>75 mm ASPHALT</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FILL: brown sand and gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- topsoil and wood debris inclusions</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- dry to moist</td>
<td></td>
</tr>
<tr>
<td>1-3</td>
<td>Loose to compact, brown, silty SAND (SM)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- some clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- moist</td>
<td></td>
</tr>
<tr>
<td>3-8</td>
<td>- grey</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- dense</td>
<td></td>
</tr>
<tr>
<td>8-10</td>
<td>END OF BOREHOLE at approximately 9.8 m below existing grade.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Borehole caved at approximately 2.7 m below grade.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Water level measured at approximately 2.4 m below grade on completion of drilling.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>WATER CONTENT &amp; ATTERBERG LIMITS</th>
<th>DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m</th>
<th>STANDARD PENETRATION TEST, BLOWS/0.3m</th>
<th>JANUARY 2003 Edition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10-30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50-70</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70-90</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>90-110</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**REMARKS & GRAIN SIZE DISTRIBUTION (%)**

- **W**
- **P**
- **G**
- **S**
- **A**
- **R**
- **Q**
- **D**
- **RQD**

**GEOLOGICAL STRATA PLOT**

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>NUMBER</th>
<th>N-VALUE</th>
<th>RECOVERY (%)</th>
<th>TCR (%)</th>
<th>SCR (%)</th>
<th>GRANULAR GROUP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS 1</td>
<td>410</td>
<td>610</td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>SS 2</td>
<td>530</td>
<td>610</td>
<td>13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>SS 3</td>
<td>410</td>
<td>610</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>SS 4</td>
<td>230</td>
<td>610</td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>SS 5</td>
<td>610</td>
<td>610</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>SS 6</td>
<td>460</td>
<td>610</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>SS 7</td>
<td>610</td>
<td>610</td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>SS 8</td>
<td>410</td>
<td>610</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>SS 9</td>
<td>380</td>
<td>610</td>
<td>37</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**UNDRAINED SHEAR STRENGTH (kPa)**

- **Field Vane Test, kPa**
- **Remoulded Vane Test, kPa**
- **Pocket Penetrometer Test, kPa**
### BOREHOLE RECORD

**BH 2**

<table>
<thead>
<tr>
<th>CLIENT</th>
<th>City of Barrie</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOCATION</td>
<td>Salem Road, Barrie, ON</td>
</tr>
<tr>
<td>DATES: BORING</td>
<td>Nov. 14, 2016</td>
</tr>
<tr>
<td>PROJECT No.</td>
<td>165011003</td>
</tr>
<tr>
<td>DATUM</td>
<td></td>
</tr>
</tbody>
</table>

#### STRATA DESCRIPTION

**SHORT GRASS (EDGE OF ROAD)**

- **FILL: brown silty sand, trace gravel**
  - topsoil inclusions
  - moist to wet

- **Very dense, brown to grey, silty SAND (SM)**
  - moist to wet

  - **grey**

**END OF BOREHOLE** at approximately 9.4 m below existing grade.

Water level measured at approximately 1.5 m below grade in the open borehole on completion of drilling.

#### STRATA PLOT

**WATER LEVEL**

- **WATER CONTENT & ATTERBERG LIMITS**
- **DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m**
- **STANDARD PENETRATION TEST, BLOWS/0.3m**
- **REMARKS & GRAIN SIZE DISTRIBUTION (%)**
- **UNDRAINED SHEAR STRENGTH (kPa)**
- **TCR(%) / SCR(%)**

#### TABLE: SAMPLES

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>STRATA DESCRIPTION</th>
<th>WATER LEVEL</th>
<th>WATER LEVEL</th>
<th>SAMSPE NUMER</th>
<th>RECOVERY (mm)</th>
<th>TCR / SCR</th>
<th>N-VALUE OR RQD (%)</th>
<th>W</th>
<th>P</th>
<th>W L</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>150 mm TOPSOIL</td>
<td></td>
<td></td>
<td>SS 1</td>
<td>410</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td>SS 2</td>
<td>300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>SS 3</td>
<td>380</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td>SS 4</td>
<td>230</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>SS 5</td>
<td>610</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>SS 6</td>
<td>530</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>SS 7</td>
<td>610</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td>SS 8</td>
<td>610</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td>SS 9</td>
<td>610</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### REMARKS

- **Field Vane Test, kPa**
- **Remoulded Vane Test, kPa**
- **Pocket Penetrometer Test, kPa**
## BOREHOLE RECORD

**BH 3**

**CLIENT**  City of Barrie

**LOCATION**  Salem Road, Barrie, ON

**DATES: BORING**  Nov. 11, 2016

**WATER LEVEL**

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>STRATA DESCRIPTION</th>
<th>WATER LEVEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Tall Grass (Edge of Road)</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>FILL: brown silty sand, trace gravel - topsoil inclusions - occasional silty clay layers - moist</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Very stiff to hard, grey, silty CLAY (CL-ML) - moist</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>Very dense, brown, silty SAND (SM) - moist to wet - 0.6 m thick sand and gravel layer at approximately 6.1 m below grade</td>
<td>3</td>
</tr>
</tbody>
</table>

### WATER CONTENT & ATTERBERG LIMITS

<table>
<thead>
<tr>
<th>SAMPLES</th>
<th>RECOVERY (%)</th>
<th>N-VALUE (%)</th>
<th>TGR(%)</th>
<th>SCR(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS 1</td>
<td>360</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 2</td>
<td>610</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 3</td>
<td>510</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 4</td>
<td>330</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 5</td>
<td>610</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 6</td>
<td>610</td>
<td>46</td>
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<td></td>
</tr>
<tr>
<td>SS 7</td>
<td>560</td>
<td>56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 8</td>
<td>560</td>
<td>82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 9</td>
<td>250</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 10</td>
<td>250</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS 11</td>
<td>510</td>
<td>50</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### UNDRAINED SHEAR STRENGTH (kPa)

- Field Vane Test, kPa
- Remoulded Vane Test, kPa
- Pocket Penetrometer Test, kPa

### REMARKS

- Non Plastic
- 8 58 21 13
- 9 6 56 29

---

**Continued Next Page**
<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>STRATA DESCRIPTION</th>
<th>STRATA PLOT</th>
<th>WATER LEVEL</th>
<th>SAMPLES</th>
<th>UNDRAINED SHEAR STRENGTH (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>Loose to very dense, brown, silty SAND (SM)</td>
<td></td>
<td></td>
<td>SS 12 610</td>
<td>TCR(%) / SCR(%)</td>
</tr>
<tr>
<td>14</td>
<td>- grey</td>
<td>SS 13 610</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>- loose</td>
<td></td>
<td></td>
<td>SS 14 610</td>
<td>TCR(%) / SCR(%)</td>
</tr>
<tr>
<td>16</td>
<td>compact</td>
<td>SS 15 610</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td>SS 16 610</td>
<td>TCR(%) / SCR(%)</td>
</tr>
<tr>
<td>18</td>
<td>END OF BOREHOLE at approximately 20.4 m below existing grade.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Water level measured at approximately 2.3 m below grade in the open borehole on completion of drilling.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Water content & Atterberg limits:
- Wp
- W
- WL

Dynamic cone penetration test, blows/0.3m
- TC
- CR

Standard penetration test, blows/0.3m
- TP
- CR

Pocket penetrometer test, kPa
- SM
- RM

Remoulded vane test, kPa
- PV
- RMV

Field vane test, kPa
- TV
- TVR

Grain size distribution (%)
- SA
- SI
- CL

N-value or RQD (%)
- NR
- RQD
**Gravel Road Shoulder**

- **FILL:** brown sand and gravel
  - occasional topsoil inclusions
  - moist

- **Stiff to very stiff, brown, silty CLAY (CL-ML)**
  - moist
  - brown to grey

- **Very dense, brown, silty SAND (SM)**
  - wet

**END OF BOREHOLE** at approximately 9.6 m below existing grade.

Borehole caved at approximately 2.4 m below grade.

Water level measured at approximately 3.0 m below grade on completion of drilling.
Appendix D

D.1 LABORATORY TEST RESULTS
### Remarks:

Salem Road Class EA  
**Location:** Salem Road, Barrie, ON  
**Project No.:** 165011003  

---

**U.S. STANDARD SIEVE OPENING IN INCHES**

<table>
<thead>
<tr>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>0.01</td>
<td>0.1</td>
<td>1</td>
<td>10</td>
<td>100</td>
<td>150</td>
<td>200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**U.S. STANDARD SIEVE NUMBERS**

| 1.5 | 3 | 4 | 5 | 6 | 8 | 10 | 14 | 16 | 20 | 30 | 40 | 50 | 70 | 100 | 140 | 200 |
|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

**GRADATION CURVE (ASTM D422)**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Description</th>
<th>W%</th>
<th>Wp</th>
<th>Ip</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH 3</td>
<td>1.1</td>
<td>FILL: silty sand</td>
<td>13</td>
<td>NP</td>
<td>NP</td>
<td>8</td>
<td>58</td>
<td>21</td>
<td>13</td>
</tr>
</tbody>
</table>

---

**UNIFIED CLASSIFICATION SYSTEM**

<table>
<thead>
<tr>
<th>BLDs</th>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>SILT &amp; CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>coarse</td>
<td>fine</td>
<td>coarse</td>
</tr>
</tbody>
</table>

---

**Project:** Salem Road Class EA  
**Location:** Salem Road, Barrie, ON  
**Project No.:** 165011003  

---
## ATTERBERG LIMITS

### (ASTM D4318)

**Figure:** 3

**Remarks:**

### Specimen Details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Depth (m)</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Fines</th>
<th>W%</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH 3</td>
<td>3.4</td>
<td>28</td>
<td>20</td>
<td>8</td>
<td>85</td>
<td>21</td>
<td>silty CLAY (CL-ML)</td>
</tr>
<tr>
<td>BH 4</td>
<td>3.4</td>
<td>23</td>
<td>13</td>
<td>10</td>
<td>59</td>
<td>14</td>
<td>silty CLAY (CL-ML)</td>
</tr>
</tbody>
</table>

---

**Project:** Salem Road Class EA

**Location:** Salem Road, Barrie, ON

**Project No.:** 165011003
Project: Salem Road Class EA
Location: Salem Road, Barrie, ON
Project No.: 165011003

Remarks:

Salem Road Class EA

HYDROMETER

GRADATION CURVE (ASTM D422)

Figure: 4

Remarks:
PLASTICITY CHART

LOW          MEDIUM          HIGH

60               CL               CH
50               "A"LINE
40
30
20
10
0

LIQUID LIMIT (LL)

CL
ML or OL
MH or OH

CL-ML

ML

PLASTICITY INDEX (PI)

 Specimen | Depth (m) | LL | PL | PI | Fines | W% | Classification
---|---|---|---|---|---|---|---
BH 1     | 3.4 | 13 | 10 | 3  | 44  | 10 | silty SAND (SM)

Remarks:

Project: Salem Road Class EA
Location: Salem Road, Barrie, ON
Project No.: 165011003

Figure: 5
Remarks:
Appendix E

E.1 SITE PHOTOS
SITE PHOTOS - Location of Culvert Replacements and Railway Bridge Abutment.

**Photo 1:** CSP Culvert under Huronia Road.

**Photo 2:** Double Box Culvert under Lockhart Road.
Photo 3: Single Span Railway Bridge over Lockhart Road.

Photo 4: Open Footed Box Culvert under Essa Road.
Appendix F

F.1 GEOTECHNICAL OVERVIEW LETTER
December 2, 2015
File: 165011003

Attention: Bala Araniyasundaran, P. Eng., PMP, LEED GA
MHPM Project Management Inc.
Program Coordinator - Growth Development Projects
City of Barrie, Engineering Department
70 Collier Street, PO Box 400
Barrie, ON, L4M 4T5

Mr. Araniyasundaran,

Reference: Preliminary Geotechnical Overview
Transportation Environmental Assessment
Hewitt Growth Development Area – Assignment #3

INTRODUCTION
Stantec Consulting Ltd. (Stantec) was commissioned by the City of Barrie to undertake a Transportation Environmental Assessment for the project captioned above.

The Assessment included a geotechnical component consisting of a limited number of boreholes at the locations of 2 required At-Grade CN Rail Crossings and 3 required watercourse crossings. This geotechnical investigation work is pending at the time of preparation of this letter.

As background, the City of Barrie provided a number of geotechnical reports (10) for projects in the general area of the subject property. Stantec completed a review of the Ministry of Transportation (MTO) Geocres database to identify additional geotechnical reports in proximity to the Hewitt Growth Development Area. This letter provides a summary of the factual subsurface conditions referenced in the reports provided and obtained from the MTO database, and offers general comments regarding the implications of the conditions referenced in the context of future development in the area.

SUMMARY OF EXISTING GEOTECHNICAL REPORTS

Area of the North Limit of the Study Area - Report No.s 42/182/212/187/186

Fill materials were encountered in a number of the investigation holes. The fills were associated with the presence of the existing roads and associated buried services/utilities. The typical depth of the fill materials ranged from less than 1 m to approximately 2 m. The fill materials typically consisted of sand to silty sand with trace gravel to sandy silt.

Design with community in mind
The predominant native soils encountered in the investigation holes consisted of the following:

- **Sand/Silty Sand/Sandy Silt/Silt** – described as predominantly granular [sic] soils extending to depths commonly in the order of 5 m to 10 m though thinner zones do exist. The granular soils have a compactness ranging from very loose to very dense, though the very loose conditions are typically at the upper contact and do not extend to depths beyond 1 m to 2 m below existing grade. Below this depth the granular soils are typically in a compact state as a minimum and more commonly in a dense to very dense state.

- **Glacial Till** – described as commonly consisting of more granular [sic] materials including sandy silt and silt with varying amounts of gravel and clay (typically less than 10%) though zones of slightly cohesive soils such as clayey silt do exist. The till contains occasional boulders. The granular till is typically in a compact to very dense state and the cohesive clayey silt till typically has a stiff to hard consistency. The thickness varies but is often in the order of 1 m to 5 m thick. There are locations where the overlying granular soils consisting of sand/silty sand/sandy silt/silt are not present and the glacial till is present at or near the ground surface or at shallow depth; this appears to more common to the extreme east of the area.

Bedrock was not encountered to the maximum depth (i.e. 18.6 m) of the investigation holes.

Groundwater was often present in the overlying granular soils; depths in the order of 2 m and 7 m below grade were common.

One of the investigations encountered a 300 mm thick layer of peat in one borehole, though this was considered isolated.

**Beyond the North Limit of the Study Area - Report No.s 172/246/063/091**

Fill materials were encountered in a large number of the investigation holes. The fills were associated with existing roads and buried services/utilities. The typical depth of the fill materials ranged from less than 1 m to approximately 2 m. The fill materials typically consisted of sand to silty sand with trace gravel to sandy silt.

The predominant native soils encountered in the investigation holes consisted of the following:

- **Silty Sand to Sandy Silt** – predominantly granular [sic] soils (sometimes described as till) extending to depths commonly in the order of 5 m or less but a thickness in excess of 10 m was recorded. The granular soils have a compactness ranging from compact to very dense.

- **Clayey Silt to Silty Clay** – predominantly cohesive soil (typically described as glacial till) though zones of cohesionless sandy silt to silty sand were encountered in some boreholes. The cohesive soil has a firm to hard consistency; the cohesionless soil was in a compact to dense state.
state. The till typically contains trace to some gravel and occasional cobbles. The thickness varies considerably; a maximum of 6.4 m was recorded.

There are locations where the overlying granular soils consisting of silty sand to sandy silt are not present and the cohesive glacial till is present at or near the ground surface or at shallow depth; this appears to occur to the extreme east of the area.

Bedrock was not encountered to the maximum depth (11.1 m) of the investigation holes.

Groundwater wells were not included in the investigations referenced here. Seepage and wet conditions were reported within a depth of 1 m to 4 m below grade in a number of boreholes; it was inferred that this represented perched groundwater conditions.

Soils described as “organic silt” were encountered in a number of the boreholes advanced for one of the investigations. Peat was also encountered in a single borehole in one of the investigations. The organic silt was present to a depth of 2 m and the peat was present underlying the existing pavement structure.

**Area to the West of the Study Area – Report No. 232**

The predominant native soils encountered in the investigation holes consisted of the following:

- **Sand and Gravel** – this soil contained trace organics. The soil was in a loose to compact state. The sand and gravel extended to depths of approximately 0.5 m to 1.4 m.
- **Organic Silt** – the organic silt was in a soft to firm condition. The stratum was encountered in two boreholes over depth intervals of 1.4 m to 2 m and 0.5 m to 2.9 m.
- **Sand** – the sand was in a compact to dense state. The sand extended to depths of 2.3 m to 2.9 m below grade.
- **Silty Clay** – the cohesive silty clay soil was encountered in all the boreholes. The silty clay had a very stiff to hard consistency. All the boreholes terminated in this soil strata.

Groundwater levels were recorded in wells installed in the boreholes. The levels were approximately 1.4 m below grade.
To supplement the information made available by the City of Barrie, Stantec has undertaken a review of the Ministry of Transportation of Ontario (MTO) Geocres database for relevant information. Three reports were identified in the general area of the project site. Reference to these reports, with a brief summary of the subsurface conditions encountered in the investigation holes, is provided below.

- **31D00-169 Investigation of Highway 400 overpass at Highway 27 (3 km north of Lockhart Road)**
  Subsurface conditions reported were as follows:
  - Loose (surficial) to very dense sand to sand with silt to a depth of 28 feet (8.5 m)
  - Compact to very dense granular glacial till to a maximum depth of 47 feet (14.3 m)
  - Groundwater levels were recorded at depths ranging from 3 feet to 14 feet (0.9 m to 4.3 m) below grade

- **31D00-362 Molson Park Drive Bridge Replacement over Highway 400 (1 km north of Lockhart Road)**
  Subsurface conditions reported were as follows:
  - Compact to very dense silty sand to silt with trace to some sand to a depth in the order of 6.9 m
  - Stiff to hard silty clay (localized) to a depth of 2.4 m
  - The boreholes were dry on completion

- **31D00-186 Investigation of Highway 400 crossing of CNR (2 km south of McKay Road)**
  Subsurface conditions reported were as follows:
  - Very dense gravelly sand
GEOTECHNICAL CONSIDERATIONS

The following geotechnical considerations are set forth based on the information reviewed:

- Overburden stratigraphy across the Study Area typically consists of a stratum of granular soil (typically silty sand, sandy silt, and silt) underlain by a stratum of either granular or cohesive glacial till (typically silty sand with gravel to silty clay).
- Groundwater has been recorded at relatively shallow depth, often less than 5 m below grade.
- Bedrock was not encountered in the investigation holes (advanced to maximum depths in the order of 11 m to 19 m below grade).
- Several boreholes encountered the presence of organic silt or peat. The presence of these materials should not be ruled out in the Study Area.
- The surficial zone of the granular silty sand/sandy silt/silt soil (typically to a depth in the order of 1 m to 2 m below grade) is often described as very loose or loose. Conventional spread footing foundations should not be placed in this zone.
- The lower zone in the granular silty sand/sandy silt/silt soils (typically below a depth in the order of 1 m to 2 m) is often described as compact to very dense granular soils. Conventional spread footing foundations placed in this zone have been used to support a range of infrastructure components and development.
- Pile foundations have been used to support higher loads and/or where specific infrastructure requirements dictate (e.g. integral abutments for road bridges).
- The reports indicate that 3 general conditions can be anticipated with respect to excavations for construction of below-grade infrastructure:
  - Shallow excavations in the granular soils – Provided that the excavations do not extend below the static groundwater table level, some sloughing of the sidewalls and only minor seepage and infiltration should be anticipated;
  - Deeper excavations in the granular soils – For excavations that extend below the groundwater table in the granular soils, dewatering will likely be required; and,
  - Excavations in the dense granular glacial till and/or more cohesive glacial till will likely encounter minor seepage and infiltration.
December 2, 2015
Bala Araniyasundaran
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Reference: Preliminary Geotechnical Overview
Transportation Environmental Assessment
Hewitt Growth Development Area – Assignment #3

CLOSURE

The summary provided herein is based on the sources of information as referenced. Stantec does
not assume any responsibility for the accuracy of the information provided.

If you have any questions regarding the content of this letter, please do not hesitate to contact
the undersigned.

Regards,

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Design with community in mind