Appendix “E”
Geotechnical Investigation
GEOTECHNICAL INVESTIGATION
PROPOSED BRYNE DRIVE REALIGNMENT
BARRIE, ONTARIO
For
THE CORPORATION OF THE CITY OF BARRIE

PETO MacCALLUM LTD.
19 CHURCHILL DRIVE
BARRIE, ONTARIO
L4N 8Z5
PHONE: (705) 734-3900
FAX: (705) 734-9911
EMAIL: barrie@petomaccallum.com

Distribution:
3 cc: Client (+email)
2 cc: Airley and Associates Limited (+email)
1 cc: PML Barrie
1 cc: PML Toronto

PML Ref.: 08BF064
Report: 1
November 2008
November 4, 2008

Mr. Ralph Scheunemann
The Corporation of The City of Barrie
Engineering Department
P.O. Box 400
70 Collier Street
Barrie, Ontario
L4M 4T5

Geotechnical Investigation
Proposed Bryne Drive Realignment
Barrie, Ontario

We are pleased to present the results of the geotechnical investigation recently completed at the above noted project site. Authorization for this work was provided by Mr. R. Scheunemann in the signed Engineering Services Agreement, PML Ref.: 08BF064, dated September 30, 2008.

Two alternatives are being considered to realign Bryne Drive near the Whiskey Creek storm water management pond. The planned five lane municipal roadway will either be constructed by widening (if feasible) the existing berm for the Whiskey Creek storm water management pond or alternatively traverse over the existing berm and pond via a bridge.

The purpose of this investigation was to assess the subsurface conditions at the site, and based on this information, provide comments and geotechnical engineering recommendations for the widening the existing berm and the design of the bridge foundations. Geo-environmental assessment of the site was not within the terms of reference, and no work has been carried out in this regard.

The comments and recommendations provided in this report are based on the conditions identified at the time of the investigation, and are applicable only to the proposed development as described in the report. Any change in development plans, grading, or layout, must be submitted for review by Peto MacCallum Ltd. (PML) to ensure applicability of the recommendations contained in the report, and may require modifications to the recommendations, additional investigation and/or analysis.
INVESTIGATION PROCEDURES

The field work for the investigation was carried out on September 22, 2008, and comprised five boreholes. Boreholes 1 and 5 were drilled to 3.5 m depth and Boreholes 2 to 4 were drilled to 10.9 to 11.1 m depth at the locations shown on Drawing 1, appended.

Co-ordination of underground utility clearances was provided by PML.

The boreholes were advanced using continuous flight solid stem augers powered by a track mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full time supervision of a member of our engineering staff.

Representative samples of the overburden were recovered at frequent depth intervals for identification purposes using a conventional split spoon sampler. Standard penetration tests were carried out simultaneously with the sampling operations to assess the strength characteristics of the substrata. Ground water conditions were closely monitored during the course of the field work. Standpipes were installed in Boreholes 2, 3 and 4 to monitor ground water levels. It is noted that the standpipe in Borehole 2 was decommissioned the same day it was installed at the request of the landowner of the property.

The location and ground surface elevation at each borehole were established in the field by PML. Elevations were referred to the following temporary benchmark established in the field.

<table>
<thead>
<tr>
<th>TBM:</th>
<th>Top of overflow structure of Whiskey Creek storm water management pond</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Elevation 100.00 (metric, assigned)</td>
</tr>
</tbody>
</table>

Upon completion, the boreholes were backfilled in accordance with O. Reg. 903.

All recovered samples were returned to our laboratory for routine moisture content determination and detailed examination to verify field classification. Three samples were submitted to our
laboratory for grain size analysis. Two samples were sent to an external laboratory for soluble sulphate content analyses.

**SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions, including soil classifications, inferred stratigraphy, standard penetration test N values, standpipe installation details, ground water observations and the results of laboratory moisture content determinations.

The boreholes encountered surficial topsoil, over fill, overlying a major sand/sand and gravel deposit. Local clayey silt and silt layers were noted. A description of the distribution and characteristics of the various soil units together with ground water observations is presented below.

**Topsoil**

Boreholes 1 to 4 revealed surficial silt topsoil varying from 100 to 400 mm thick.

**Fill**

Fill was encountered in all boreholes. In Boreholes 1 and 2 the fill was relatively thin beneath the topsoil, extending to 0.4 and 0.5 m depth, respectively. Boreholes 3 and 4, advanced through the storm water management pond berm, encountered fill to 7.0 and 6.0 m depth, respectively. In Borehole 5 the fill extended from the surface to 0.5 m depth. The material comprised brown sand, with varying clay, silt and gravel content. The material was moist becoming saturated at the base of the unit in Boreholes 3 and 4.

**Sand**

A major basal sand deposit was encountered in Boreholes 2 to 5. Below the fill materials or other local soil units the sand extended to the depth of exploration. The deposit was interrupted by a clayey silt layer between 7.0 and 9.0 m depth in Borehole 2. The deposit comprised sand with
varying amounts of silt and gravel, locally sand and gravel in Borehole 3. The relative density was compact to very dense. The material was moist becoming saturated just above the clayey silt layer in Borehole 2 (5.8 m depth) and at 7.4 m depth in Borehole 4. Figures 1 and 2 present grain size analysis results conducted on two samples of the sand.

**Silt**

A silt trace sand layer was encountered from 0.5 to 1.5 m depth in Borehole 5. The silt was compact and had a moisture content of 14%.

**Clayey Silt**

A local clayey silt layer was encountered in Boreholes 2 and 3 from 7.0 to 9.0 m depth. The material was hard and contained trace sand. The moisture contents were 21 and 24%. Atterberg limit testing indicated a liquid limit of 29 and a plastic limit of 16. The results of a grain size analysis conducted on a sample of the clayey silt is presented on Figure 3.

**Sandy Silt**

A local sandy silt layer was revealed beneath the fill in Borehole 1 to the 3.5 m depth of exploration. The unit was loose to compact and contained silty sand seams. Moisture contents were around 20%.

**Ground Water**

Upon completion of augering, wet cave was recorded at 1.5, 7.9, 6.1, and 7.3 m depth in Boreholes 1 to 4, respectively. No free water was noted in Borehole 5. Ground water level readings of 5.8, 6.7 and 7.4 m depth were recorded on September 24, 2008, in standpipes in Boreholes 2 to 4, respectively. In general, ground water appears to be perched within pervious soils overlying less pervious strata.
The ground water level will fluctuate seasonally, in response to variations in precipitation and corresponds closely to the adjacent creek water level.

ENGINEERING CONSIDERATIONS

Widening of Existing Storm Water Management Pond Berm

The purpose of the investigation was to determine if the existing berm for the storm water management pond could be widened to support the newly planned road without impacting the berm or the storm water management pond.

Boreholes 3 and 4 encountered the fill materials that comprise the existing berm. The standard penetration test N values for the fill typically showed the fill to be in a compact condition. Based on this, widening of the existing berm for use as a roadway appeared favourable.

A slope stability analysis was carried out using the Bishop’s Simplified Method on Geo Studio slope stability software. The model for the analysis was based on the borehole information and cross section drawings provided by Ainley and Associates Limited.

Based on the boreholes the following parameters were used in stability analysis.

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>UNIT WEIGHT k/N / m³</th>
<th>ANGLE OF INTERNAL FRICITION (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing Fill</td>
<td>20</td>
<td>32</td>
</tr>
<tr>
<td>Engineered Fill</td>
<td>20</td>
<td>32</td>
</tr>
<tr>
<td>Clayey Silt</td>
<td>21</td>
<td>35</td>
</tr>
<tr>
<td>Sand/Sand and Gravel</td>
<td>21</td>
<td>35</td>
</tr>
</tbody>
</table>
Based on the cross section drawings provided by Ainley and Associates Limited the section at Station 4+140 was utilized in the model which indicated a 2 horizontal to 1 vertical slope for the outside of the newly planned slope.

Several trials were assessed using varying ground water/storm water levels and traffic loading in order to explore/examine the stability of the newly proposed embankment. In general, a factor of safety of 1.3 or greater is considered acceptable. The analysis indicated factor of safety of 1.34 where the model incorporated the worst case scenario.

Widening of the existing berm in order to support the planned roadway is feasible provided that certain measures are taken during the construction including engineered fill (discussed below) and erosion control.

Side slopes of the new embankment should be protected from surface erosion by sodding or by seed and mulch as soon as possible following construction. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

**Engineered Fill**

Widening of the existing storm water management pond berm to support the newly planned road must be constructed as engineered fill. General guidelines for engineered fill construction are provided in Appendix A. The following are highlighted.

- Existing fill, topsoil, organic, frozen or otherwise, deleterious materials should be removed down to competent native mineral soils

- The exposed subgrade should be proof-rolled using a static roller to ensure 95% Standard Proctor maximum dry density.

- The engineered fill should be benched into the existing berm to prevent future slip surfaces. Refer to OPSD 208.010.
• Once the subgrade has been approved, the design grade can then be achieved using select on site or approved imported granular material (OPSS Granular B or equivalent) placed in maximum 200 mm thick lifts and uniformly compacted to 98% Standard Proctor maximum dry density.

• Survey controls must be implemented to ensure the engineered fill pad is of sufficient extent to accommodate the widened berm.

• The engineered fill construction should be reviewed on a full time basis by PML to verify subgrade preparation, approve backfill material, placement and compaction procedures and ensure the specified compaction is achieved uniformly throughout.

**Bridge Option**

**Foundations**

In the event that the existing berm will not be used to construct the embankment for the newly planned road a bridge is planned in order to span over the storm water management pond and berm. It is understood that the bridge construction would be planned so there is no impact to the existing storm water management pond or berm.

Borehole 2 was advanced near the north abutment. Based on Borehole 2, footings supported on the native sand below elevation 100.0 can be designed for a bearing resistance of 350 kPa at Serviceability Limit State (SLS) and a factored bearing resistance of 525 kPa at Ultimate Limit State (ULS).

Berm fill was encountered in Borehole 4, near the planned south abutment, extending to a depth of 6.0 m. The fill is considered unsuitable for support of foundations for the bridge. As noted above the bridge construction is planned so there is no impact to the existing storm water management pond or berm.

Based on Borehole 4 footings for the south abutment (assumed to be south of Borehole 4) supported by native sand deeper than 3.0 m below existing grades can be designed for a bearing resistance of 300 kPa at SLS and a factored bearing resistance of 500 kPa at ULS.
For stability reasons the footings should not be placed above a line extending from the base of the storm water management pond extending back at a slope of 3:1. Also, the values provided for ULS should be reviewed once the elevations of the bridge foundations are established.

Footings subject to frost action should be provided with a minimum 1.2 m of earth cover or equivalent.

Prior to placement of structural concrete, all founding surfaces must be inspected by PML to check that the design bearing resistance is available, or to reassess the design parameters based on the actual conditions.

**Seismic Considerations**

The seismic site coefficient, S, for the stratigraphic conditions at the site is 1.2 (soil profile Type II, CHBDC, CAN/CSA-S6-06, clause 4.4.6).

Based on the type and relative density of the soil cover at the site, there is a low potential for liquefaction of the soils to occur (CHBDC 4.6.2).
Abutment Wall Design

Abutment walls must be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, $p$, may be computed using the equivalent fluid pressures presented in Section 6.9.2 of the Canadian Highway Bridge Design Code (CAN/CSA-S6-06) or employing the following equation, assuming a triangular pressure distribution:

$$ p = K (\gamma h + q) $$

Where

- $K$ = lateral earth pressure coefficient
- $\gamma$ = unit weight of free draining granular backfill ($\text{kN/m}^3$)
- $h$ = depth below final grade (m)
- $q$ = surcharge adjacent to the wall (kPa)

In addition, there should be allowance for seismic events (CHBDC clause 4.6.4) and compaction pressure (CHBDC clause 6.9.3).

Free draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

<table>
<thead>
<tr>
<th></th>
<th>Granular A</th>
<th>Granular B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of Internal Friction (degrees)</td>
<td>35</td>
<td>32</td>
</tr>
<tr>
<td>Unit Weight ($\text{kN/m}^3$)</td>
<td>22.8</td>
<td>21.2</td>
</tr>
<tr>
<td>Active Earth Pressure Coefficient ($K_a$)</td>
<td>0.27</td>
<td>0.31</td>
</tr>
<tr>
<td>At Rest Earth Pressure Coefficient ($K_o$)</td>
<td>0.43</td>
<td>0.47</td>
</tr>
<tr>
<td>Passive Earth Pressure Coefficient ($K_p$)</td>
<td>3.70</td>
<td>3.22</td>
</tr>
</tbody>
</table>

A weeping tile system and/or weeping holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.
Backfill should be placed in thin lifts compacted to a minimum 95% Standard Proctor maximum dry density.

**Approach Fill**

In general, approach fill embankments should be constructed in accordance with OPSS 206 and OPSD 200.01. The side slopes of the approach embankments should be inclined no steeper than 3H:1V for earth fill.

Backfill adjacent to the structure should be carried out in conformance with Ontario Provincial Standards Specification for granular backfill.

Side slopes should be protected from surface erosion by sodding or by seed and mulch as soon as possible following construction. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

**Pavement Design and Construction**

Based on the boreholes the pavement subgrade predominately is expected to comprise sand. The following pavement structure is recommended.

<table>
<thead>
<tr>
<th>Pavement Structure</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalitic Concrete</td>
<td>140</td>
</tr>
<tr>
<td>Granular A Base (mm)</td>
<td>150</td>
</tr>
<tr>
<td>Granular B Subbase (mm)</td>
<td>350</td>
</tr>
</tbody>
</table>

It is noted that no traffic numbers were not available at the time of this report and the recommended design is based the City of Barrie standard for an 18.0 m asphalt road (five lanes).

Imported material for the granular base and subbase should conform to OPS gradation specifications for Granular A and Granular B, respectively, and should be compacted to
100% Standard Proctor maximum dry density. Asphaltic concrete should be compacted to 92 to 96.5% of maximum relative density in accordance with OPSS 310.

Subgrade preparation should comprise stripping of topsoil and other obvious deleterious materials, followed by proofrolling using a heavy compactor. Any unsuitable/unstable zones identified during this process should be removed and replaced with select material placed in maximum 200 mm thick lifts and compacted to at least 95% Standard Proctor maximum dry density.

The pavement design considers the construction will be carried out during the dry time of the year and the subgrade is stable and not heaving under construction traffic. If wet unstable conditions are encountered, additional granular subbase material may be required, subject to geotechnical field review.

For the pavement to function properly, it is essential that provisions be made for water to drain out of and not collect in the base material. In this regard, crowning/grading of the subgrade and final surface should be included to promote drainage away from the structure. Backfill adjacent to catchbasins and manholes should comprise fine draining Granular B, to assist in drainage of the pavement.

**Excavation and Ground Water Control**

Based on the boreholes, excavation is expected to be carried out through topsoil, fill and into the major sand deposit, locally in to the discontinuous silt units. In general, excavation within the overburden materials should be straightforward using conventional equipment.

Ground water was typically below the anticipated excavation depths. Locally perched ground water or surface water runoff is believed to be of nuisance quantity that can be handled by conventional sump pumping.
The site soils should be considered Type 3 soil conditions, requiring side walls to be constructed at no steeper than 1H:1V from the base of the excavation, in accordance with the Ontario Health and Safety Act.

**Sulphate Testing**

Two soil samples were submitted to an external laboratory for analysis of soluble sulphates. The Certificate of Analysis is presented in Appendix B.

In accordance with Canadian Standard Association, CSA-A23.1-04, Table 3, the results indicate a negligible potential degree of sulphate attack on buried concrete. Accordingly, the use of normal portland cement is indicated.
Geotechnical Review and Construction Inspection and Testing

It is recommended that the final design drawings be submitted for review by PML prior to finalization to ensure the recommendations contained in this report are properly interpreted and the design is compatible with the site subsurface conditions identified.

Prior to placement of structural concrete, all founding surfaces must be inspected by PML to verify the design bearing capacity is available, or to reassess the design parameters based on the actual conditions.

Earthworks operations should be carried out under the supervision of PML, to approve subgrade preparation, backfill materials, placement and compaction procedures, and verify the specified degree of compaction is achieved uniformly throughout fill materials.

The comments and recommendations provided in the report are based on the information revealed at the boreholes. Soil conditions at other locations may vary, particularly in the vicinity of the existing foundations, and therefore, geotechnical review during construction should be ongoing to confirm the subsurface conditions are substantially similar to those identified in this investigation, which may otherwise require modification to the original recommendations.
CLOSURE

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.

Geoffrey R. White, P.Eng
Manager, Geotechnical and Geoenvironmental Services

Marian S. Molodecki, P.Eng.
Senior Consultant,
Geotechnical and Geoenvironmental Services

JH/GRW:jlb

Enclosures:
Figures 1 to 3
List of Abbreviations
Log of Borehole No's 1 to 5
Borehole Location Plan
Appendix A – Engineered Fill
Appendix B – Certificate of Analysis
Borehole 2, Sample 6, 4.5 - 5.0 m depth
SAND, Some Silt
Borehole 4, Sample 8, 7.6 - 8.1 m depth
SAND, Some Silt
LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: - The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow.

DESCRIPTION OF SOIL

The consistency of cohesive soils and the relative density or denseness of cohesionless soils are described in the following terms:

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>N (blows/0.3 m)</th>
<th>c (kPa)</th>
<th>DENSENESS</th>
<th>N (blows/0.3 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>0 - 2</td>
<td>0 - 12</td>
<td>Very Loose</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>12 - 25</td>
<td>Loose</td>
<td>4 - 10</td>
</tr>
<tr>
<td>Firm</td>
<td>4 - 8</td>
<td>25 - 50</td>
<td>Compact</td>
<td>10 - 30</td>
</tr>
<tr>
<td>Stiff</td>
<td>8 - 15</td>
<td>50 - 100</td>
<td>Dense</td>
<td>30 - 50</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>15 - 30</td>
<td>100 - 200</td>
<td>Very Dense</td>
<td>&gt; 50</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WTPL</td>
<td>Wetter Than Plastic Limit</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>APL</td>
<td>About Plastic Limit</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DTPL</td>
<td>Drier Than Plastic Limit</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TYPE OF SAMPLE

| SS     | Split Spoon   | TW  | Thinwall Open |
| WS     | Washed Sample | TP  | Thinwall Piston |
| SB     | Scraper Bucket Sample | OS  | Oesterberg Sample |
| AS     | Auger Sample  | FS  | Foil Sample   |
| CS     | Chunk Sample  | RC  | Rock Core     |
| ST     | Slotted Tube Sample | PH  | Sample Advanced Hydraulically |
|        |                | PM  | Sample Advanced Manually |

SOIL TESTS

| Qu   | Unconfined Compression | LV   | Laboratory Vane |
| Q    | Undrained Triaxial     | FV   | Field Vane      |
| Qcu  | Consolidated Undrained Triaxial | C   | Consolidation  |
# LOG OF BOREHOLE NO. 1

**PROJECT:** Proposed Bryne Drive Realignment  
**LOCATION:** Barrie, Ontario  
**BORING METHOD:** Continuous Flight Solid Stem Augers  
**BORING DATE:** September 22, 2008  
**ENGINEER:** GW  
**TECHNICIAN:** JH

## SOIL PROFILE

<table>
<thead>
<tr>
<th>ELEV. DEPTH (m)</th>
<th>DESCRIPTION</th>
<th>STRAT PLOT</th>
<th>NUMBER</th>
<th>TYPE</th>
<th>&quot;N&quot; VALUES</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>TOPSOIL: Black to brown, silt, moist</td>
<td>-</td>
<td>1</td>
<td>SS</td>
<td>4</td>
</tr>
<tr>
<td>0.4</td>
<td>FILL: Brown, sand, trace silt, trace organics, moist to wet</td>
<td>-</td>
<td>2</td>
<td>SS</td>
<td>21</td>
</tr>
<tr>
<td>1.4</td>
<td>SANDY SILT: Loose to compact, brown, sandy silt, occasional silty sand seams, saturated</td>
<td>-</td>
<td>3</td>
<td>SS</td>
<td>8</td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td>-</td>
<td>4</td>
<td>SS</td>
<td>15</td>
</tr>
<tr>
<td>3.5</td>
<td></td>
<td>-</td>
<td>5</td>
<td>SS</td>
<td>24</td>
</tr>
<tr>
<td>3.5</td>
<td>BOREHOLE TERMINATED AT 3.5 m</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## SHEAR STRENGTH (kPa)

- **UNCONFined:** Pocket Penetrometer
- **FIELD VANE:**

### Dynamic Cone Penetration:
- **STANDARD PENETRATION TEST**

### PlastiC LIMIT:

**NATURAL MOISTURE CONTENT:**

**LIQUID LIMIT:**

### WATER CONTENT (%)

- **UNIT WEIGHT:**

### REMARKS & GRAIN SIZE DISTRIBUTION (%)

Upon completion of augering  
Wet cave at 1.5 m
LOG OF BOREHOLE NO. 2

SOIL PROFILE

ELEV. IN METERS

DESCRIPTION

TOPSOIL: Black, silt, moist
FILL: Brown to black, sand, some silt, moist
SAND: Compact to very dense, light brown, sand, trace gravel, trace silt, moist
Becoming saturated
CLAYEY SILT: Hard, gray, clayey silt, trace sand, WTP
SAND: Very dense, light brown, sand, trace gravel, trace silt, moist

BOREHOLE TERMINATED AT 11.1 m

SHEAR STRENGTH kPa

UNCONFINED  FIELD VANE
ROCKET PENETROMETER

DYNAMIC CONE PENETRATION
STANDARD PENETRATION TEST

WATER CONTENT (%) UNIT WEIGHT

PLASTIC LIMIT LIQUID LIMIT

GRAIN SIZE DISTRIBUTION (%)

REMARKS

Native backfill
Bentonite seal
Slotting pipe
Filter sand

Upon completion of augering
Wet cave at 7.9 m
Water Level Readings
Date Depth (m)
Sept. 22/08 5.8
Standpipe removed and borehole decommissioned at request of private land owner

PROJECT Proposed Byrne Drive Realignment
LOCATION Barrie, Ontario
BORING METHOD Continuous Flight Solid Stem Augers
BORING DATE September 22, 2008
ENGINEER GW
TECHNICIAN JH

OUR PROJECT NO. 088F064

1 of 1
LOG OF BOREHOLE NO. 3

PROJECT: Proposed Byrne Drive Realignment
LOCATION: Barrie, Ontario
BORING METHOD: Continuous Flight Solid Stem Augers

ELEV. DEPTH: m

<table>
<thead>
<tr>
<th>STRAT PLOT</th>
<th>NUMBER</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS</td>
<td>TOPSOIL: Brown to black, silt, moist</td>
</tr>
<tr>
<td>2</td>
<td>SS</td>
<td>FILL: Brown, sandy silt, trace clay, trace organics, with wood pieces, moist</td>
</tr>
<tr>
<td>3</td>
<td>SS</td>
<td>Becoming brown, sand, some silt, trace gravel, moist</td>
</tr>
<tr>
<td>4</td>
<td>SS</td>
<td>With grey clayey silt pockets</td>
</tr>
<tr>
<td>5</td>
<td>SS</td>
<td>Becoming saturated</td>
</tr>
<tr>
<td>6</td>
<td>SS</td>
<td>CLAYEY SILT: Hard, grey, clayey silt, trace sand, WTP PL</td>
</tr>
<tr>
<td>7</td>
<td>SS</td>
<td>SAND AND GRAVEL: Very dense, brown to grey, sand and gravel, trace silt, moist</td>
</tr>
<tr>
<td>8</td>
<td>SS</td>
<td>94/250</td>
</tr>
<tr>
<td>9</td>
<td>SS</td>
<td>94/250</td>
</tr>
<tr>
<td>10</td>
<td>SS</td>
<td>BOROHEOLE TERMINATED AT 10.9 m</td>
</tr>
</tbody>
</table>

SHEAR STRENGTH kPa
- UNCONFINED
- FIELD VANE
- POCKET PENETROMETER

DYNAMIC ZONE PENETRATION
STANDARD PENETRATION TEST

NATURAL MOISTURE CONTENT w
LIQUID LIMIT w'  
WATER CONTENT (%)  

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

Native backfill
Bentonite seal
Slotted pipe Filter sand

Upon completion of augering
Wet cave at 1.5 m
Water Level Readings
Date: Sept 22 08
Depth (m): 6.7
LOG OF BOREHOLE NO. 4

PROJECT: Proposed Byrne Drive Realignment
LOCATION: Barrie, Ontario
BORING METHOD: Continuous Flight Solid Stem Augers

SOIL PROFILE

<table>
<thead>
<tr>
<th>ELEV. DEPTH (m)</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>TOPSOIL: Brown to dark brown, silt, moist</td>
</tr>
<tr>
<td>0.8</td>
<td>FILL: Brown, sand, some silt to silty, trace to some clay, trace gravel, trace organics, moist</td>
</tr>
<tr>
<td>0.6</td>
<td>SAND: Light brown, sand, trace to some silt, trace gravel, moist to saturated</td>
</tr>
</tbody>
</table>

BORING TERMINATED AT 11.1 m

SHEAR STRENGTH kPa
- UNCONFINED
- FIELD VANE
- POCKET PENETROMETER

DYNAMIC CONE PENETRATION & STANDARD PENETRATION TEST

REMARKS
- GRAIN SIZE DISTRIBUTION

Upon completion of augering
- Wet cave at 7.3 m

Water Level Readings
- Date: Sept. 22/08
- Depth: 7.4 m

Native backfill
Bentonite seal
Slotted pipe Filter sand
# LOG OF BOREHOLE NO. 5

**PROJECT**: Proposed Byrde Drive Realignment  
**LOCATION**: Barrie, Ontario  
**BORING METHOD**: Continuous Flight Stem Augers  
**BORING DATE**: September 22, 2008  
**ENGINEER**: GW  
**TECHNICIAN**: JH  

<table>
<thead>
<tr>
<th>ELEV. DEPTH</th>
<th>DESCRIPTION</th>
<th>STRAT PLOT</th>
<th>NUMBER</th>
<th>N' VALUES</th>
<th>SHEAR STRENGTH kPa</th>
<th>PLASTIC LIMIT</th>
<th>NATURAL MOISTURE</th>
<th>LIQUID LIMIT</th>
<th>DYNAMIC CONE PENETRATION</th>
<th>WATER CONTENT (%)</th>
<th>GRAIN SIZE DISTRIBUTION (%)</th>
<th>REMARKS</th>
</tr>
</thead>
</table>
| 0.0         | FILL: Brown, sandy silt, trace clay, trace organics, moat | 1 SS | 1 | 120 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | OR, SA, SI, CL | Upon completion of augering  
No free water  
No cave |
| 0.5         | SILT: Compact, brown, silt, trace sand, trace gravel, moist | 2 SS | 10 | 99 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | OR, SA, SI, CL |
| 1.8         | SAND: Compact to very dense, brown, sand, some gravel, some silt, stratified, moist | 3 SS | 24 | 68 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | OR, SA, SI, CL |
| 3.5         | BOREHOLE TERMINATED AT 3.5 m | 4 SS | 75/220 | 67 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | OR, SA, SI, CL |
| 4.0         | | 5 SS | 44 | | | | | | | | | | |

- **UNCONFINED**:  
- **FIELD VANE**:  
- **POCKET PENETROMETER**:  
- **STANDARD PENETRATION TEST**:  
- **DYNAMIC CONE PENETRATION TEST**:  

**ELEVATION SCALE**:  
- **ELEVATION**: 20, 40, 60, 80  
- **N' VALUES**:  
- **SHEAR STRENGTH**:  
- **PLASTIC LIMIT**:  
- **NATURAL MOISTURE CONTENT**:  
- **LIQUID LIMIT**:  
- **WATER CONTENT**:  
- **GRAIN SIZE DISTRIBUTION**:  
- **REMARKS**: Upon completion of augering  
No free water  
No cave
APPENDIX A

ENGINEERED FILL
ENGINEERED FILL

The information presented in this appendix is intended for general guidance only. Site specific conditions and prevailing weather may require modification of compaction standards, backfill type or procedures. Each site must be discussed, and procedures agreed with Peto MacCallum Ltd. prior to the start of the earthworks and must be subject to ongoing review during construction. This appendix is not intended to apply to embankments. Steeply sloping ravine residential lots require special consideration.

For fill to be classified as engineered fill suitable for supporting structural loads, a number of conditions must be satisfied, including but not necessarily limited to the following:

1. **Purpose**

   The site specific purpose of the engineered fill must be recognized. In advance of construction, all parties should discuss the project and its requirements and agree on an appropriate set of standards and procedures.

2. **Minimum Extent**

   The engineered fill envelope must extend beyond the footprint of the structure to be supported. The minimum extent of the envelope should be defined from a geotechnical perspective by:

   - at founding level, extend a minimum 1.0 m beyond the outer edge of the foundations, greater if adequate layout has not yet been completed as noted below; and

   - extend downward and outward at a slope no greater than 45° to meet the subgrade

   All fill within the envelope established above must meet the requirements of engineered fill in order to support the structure safely. Other considerations such as survey control, or construction methods may require an envelope that is larger, as noted in the following sections.

   Once the minimum envelope has been established, structures must not be moved or extended without consultation with Peto MacCallum Ltd. Similarly, Peto MacCallum Ltd. should be consulted prior to any excavation within the minimum envelope.

3. **Survey Control**

   Accurate survey control is essential to the success of an engineered fill project. The boundaries of the engineered fill must be laid out by a surveyor in consultation with engineering staff from Peto MacCallum Ltd. Careful consideration of the maximum building envelope is required.

   During construction it is necessary to have a qualified surveyor provide total station control on the three dimensional extent of filling.
4. **Subsurface Preparation**

Prior to placement of fill, the subgrade must be prepared to the satisfaction of Peto MacCallum Ltd. All deleterious material must be removed and in some cases, excavation of native mineral soils may be required.

Particular attention must be paid to wet subgrades and possible additional measures required to achieve sufficient compaction. Where fill is placed against a slope, benching may be necessary and natural drainage paths must not be blocked.

5. **Suitable Fill Materials**

All material to be used as fill must be approved by Peto MacCallum Ltd. Such approval will be influenced by many factors and must be site and project specific. External fill sources must be sampled, tested and approved prior to material being hauled to site.

6. **Test Section**

In advance of the start of construction of the engineered fill pad, the Contractor should conduct a test section. The compaction criterion will be assessed in consultation with Peto MacCallum Ltd. for the various fill material types using different lift thicknesses and number of passes for the compaction equipment proposed by the Contractor.

Additional test sections may be required throughout the course of the project to reflect changes in fill sources, natural moisture content of the material and weather conditions.

The Contractor should be particularly aware of changes in the moisture content of fill material. Site review by Peto MacCallum Ltd. is required to ensure the desired lift thickness is maintained and that each lift is systematically compacted, tested and approved before a subsequent lift is commenced.

7. **Inspection and Testing**

Uniform, thorough compaction is crucial to the performance of the engineered fill and the supported structure. Hence, all subgrade preparation, filling and compacting must be carried out under the full time inspection by Peto MacCallum Ltd.

All founding surfaces must be inspected and approved by Peto MacCallum Ltd. prior to placement of structural concrete.

8. **Protection of Fill**

Fill is generally more susceptible to the effects of weather than natural soil. Fill placed and approved to the level at which structural support is required must be protected from excessive
wetting, drying, erosion or freezing. Where adequate protection has not been provided, it may be necessary to provide deeper footings or to strip and recompact some of the fill.

9. **Construction Delay Time Considerations**

The integrity of the fill pad can deteriorate due to the harsh effects of our Canadian weather. Hence, particular care must be taken if the fill pad is constructed over a long time period.

It is necessary therefore, that all fill sources are tested to ensure the material compactability prior to the soil arriving at site. When there has been a lengthy delay between construction periods of the fill pad, it is necessary to conduct subgrade proof rolling, test pits or boreholes to verify the adequacy of the exposed subgrade to accept new fill material.

When the fill pad will be constructed over a lengthy period of time, a field survey should be completed at the end of each construction season to verify the areal extent and the level at which the compacted fill has been brought up to, tested and approved.

In the following spring, subexcavation may be necessary if the fill pad has been softened attributable to ponded surface water or freeze/ thaw cycles.

A new survey is required at the beginning of the next construction season to verify that random dumping and / or spreading of fill has not been carried out at the site.

10. **Approved Fill Pad Surveillance**

It should be appreciated that once the fill pad has been brought to final grade and documented by field survey, there must be ongoing surveillance to ensure that the integrity of the fill pad is not threatened.

Grading operations adjacent to fill pads can often take place several months or years after completion of the fill pad.

It is imperative that all site management and supervision staff, the staff of Contractors and earthwork operators be fully aware of the boundaries of all approved engineered fill pads.

Excavation into an approved engineered fill pad should never be contemplated without the full knowledge, approval and documentation by the geotechnical consultant.

If the fill pad is knowingly built several years in advance of ultimate construction, the areal limits of the fill pad should be substantially overbuilt laterally to allow for changes in possible structure location and elevation and other earthwork operations and competing interests on the site. The overbuilt distance required is project and / or site specified.

Iron bars should be placed at the corner / intermediate points of the fill pad as a permanent record of the approved limits of the work for record keeping purposes.
11. **Unusual Working Conditions**

Construction of fill pads may at times take place at night and/or during periods of freezing weather conditions because of the requirements of the project schedule. It should be appreciated therefore, that both situations present more difficult working conditions. The Owner, Contractor, Design Consultant and Geotechnical Engineer must be willing to work together to revise site construction procedures, enhance field testing and surveillance, and incorporate design modifications as necessary to suit site conditions.

When working at night there must be sufficient artificial light to properly illuminate the fill pad and borrow areas.

Placement of material to form an engineered fill pad during winter and freezing temperatures has its own special conditions that must be addressed. It is imperative that each day prior to placement of new fill, the exposed subgrade must be inspected and any overnight snow or frozen material removed. Particular attention should be given to the borrow source inspection to ensure only non frozen fill is brought to the site.

The Contractor must continually assess the work program and have the necessary spreading and compacting equipment to ensure that densification of the fill material takes place in a minimum amount of time. Changes may be required to the spreading methods, lift thickness, and compaction techniques to ensure the desired compaction is achieved uniformly throughout each fill lift.

The Contractor should adequately protect the subgrade at the end of each shift to minimize frost penetration overnight. Since water cannot be added to the fill material to facilitate compaction, it is imperative that densification of the fill be achieved by additional compaction effort and an appropriate reduced lift thickness. Once the fill pad has been completed, it must be properly protected from freezing temperatures and ponding of water during the spring thaw period.

If the pad is unusually thick or if the fill thickness varies dramatically across the width or length of the fill pad, Peto MacCallum Ltd. should be consulted for additional recommendations. In this case, alternative special provisions may be recommended, such as providing a surcharge preload for a limited time or increase the degree of compaction of the fill.
APPENDIX B

CERTIFICATE OF ANALYSIS
## Certificate of Analysis

**AGAT WORK ORDER:** 08T295111  
**PROJECT NO:** 08BF064  
**ATTENTION TO:** John Hagan

### Sulphate in Soil

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Soluble Sulphate (2:1)</td>
<td>Unit</td>
<td>BH2 SS3 1087264</td>
<td>BH4 SS3 1087290</td>
</tr>
<tr>
<td></td>
<td>µg/g</td>
<td>RDL 2.09</td>
<td>2.86</td>
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</tbody>
</table>

**Comments:**  
RDL - Reported Detection Limit;  
G / S - Guideline / Standard

---

**Certified By:** [Signature]

Results relate only to the items tested