Geotechnical Investigation
Residential Houses
106 Ontario Street, Ottawa, ON

Prepared for:
Pan Art and Science Inc.

Prepared by:
Stantec Consulting Ltd.
200 – 2781 Lancaster Road
Ottawa, ON K1B 1A7

Report No. 161612663

July 2012
Table of Contents

1.0 INTRODUCTION ................................................................................................................ 1

2.0 SITE DESCRIPTION AND BACKGROUND ....................................................................... 1

3.0 SCOPE OF WORK ............................................................................................................. 1

4.0 METHOD OF INVESTIGATION.......................................................................................... 2

4.1 GEOTECHNICAL FIELD INVESTIGATION......................................................................... 2

4.2 SURVEYING....................................................................................................................... 2

5.0 RESULTS OF INVESTIGATION......................................................................................... 3

5.1 SUBSURFACE INFORMATION.......................................................................................... 3

5.1.1 Surficial Materials ................................................................................................. 3

5.1.2 Fill ........................................................................................................................ 3

5.1.3 Till ........................................................................................................................ 3

5.1.4 Bedrock ................................................................................................................ 3

5.2 GROUNDWATER ............................................................................................................... 4

6.0 DISCUSSION AND RECOMMENDATIONS ....................................................................... 4

6.1 SITE GRADING AND PREPARATION................................................................................ 4

6.1.1 Building Footprint ................................................................................................. 4

6.1.2 Paved Areas.......................................................................................................... 5

6.2 FOUNDATIONS .................................................................................................................. 5

6.3 TEMPORARY EXCAVATIONS AND BACKFILLING ........................................................... 6

6.3.1 General Excavations ............................................................................................ 6

6.3.2 Foundation Backfill ............................................................................................... 6

6.3.3 Pipe Bedding and Backfill .................................................................................... 6

6.3.4 Groundwater ........................................................................................................ 7

6.4 CONCRETE FLOOR SLABS .......................................................................................... 7

6.5 CEMENT TYPE AND CORROSION POTENTIAL ............................................................... 7

6.6 PAVEMENT STRUCTURE RECOMMENDATIONS ............................................................ 8

6.7 SEISMIC SITE CLASSIFICATION ..................................................................................... 8

6.8 LATERAL EARTH PRESSURES ......................................................................................... 9

7.0 CLOSURE .........................................................................................................................12
List of Tables

Table 6.1: Geotechnical Bearing Resistance for Foundations on Native Till .............................. 5
Table 6.2: pH, Sulphate, Chloride and Resistivity Analysis Results ........................................... 7
Table 6.3: Recommended Pavement Design ............................................................................ 8
Table 6.4: Parameters for Seismic Site Classification ............................................................... 9
Table 6.5: Lateral Earth Pressure Parameters .......................................................................... 9
Table 6.6: Unfactored Friction Coefficients ............................................................................. 9
Table 6.7: Combined Coefficients of Static and Seismic Earth Pressure ................................. 10

List of Appendices

APPENDIX A  Statement of General Conditions
APPENDIX B  Key Plan
  Borehole Location Plan with Existing Site
APPENDIX C  Symbols and Terms Used on Borehole and Test Pit Records
  Borehole Records
APPENDIX D  Laboratory Test Results
1.0 Introduction

This report presents the results of the Geotechnical Investigation carried out for the proposed three unit townhouse at 106 Ontario Street in Ottawa, Ontario. The work was carried out in general accordance with the scope of work for a geotechnical investigation outlined in Stantec’s proposal 1224-B11221 dated April 30, 2012.

This report has been prepared specifically and solely for the project described herein. It presents the factual results of the investigation and provides geotechnical recommendations for the design and construction of the proposed building.

2.0 Site Description and Background

It is understood that the proposed building is to be located at the civic address 106 Ontario Street in Ottawa, Ontario. The proposed footprint of the building has an approximate area of 290 m². The building will be a three-storey structure with one below grade level and three parking spots located at the front of the building connecting to Ontario Street. The building design is assumed to include strip or spread footings.

The site for the proposed building is currently occupied by a single storey residential house. The footprint of the existing house is approximately 90 m². The area surrounding the house consists of a grassed area and an asphalt driveway located within the north-west corner of the lot. Ontario Street is positioned north of the site, and is an urban cross section with a concrete curb situated between the site and the road.

Based on soil mapping of the area, the subsurface conditions consist of a thin layer of glacial till underlain by shale bedrock. Shale bedrock of the Billings Formation is anticipated to be between 3 and 6 m below ground surface.

3.0 Scope of Work

The scope of work for this geotechnical investigation included the following:

- Advance three boreholes within the footprint of the building to 6 m below ground surface or auger refusal, if shallower.
- Perform standard penetration tests (SPT) while collecting soil samples at regular intervals.
- Install a monitoring well within one borehole. Decommissioning the well is not included within the scope of the project.
- Characterize the soils with laboratory tests including soil resistivity and pH, gradation and moisture contents.
- Survey the ground surface elevations at the borehole locations using a Trimble GPS unit.
• Prepare a Geotechnical Investigation Report for the proposed building. The report will include; a summary of the field investigation results and observations, laboratory test results, a borehole location plan, and geotechnical engineering recommendations for the design and construction of the project including:
  • Soil and bedrock conditions;
  • Site preparation, demolition, excavation and backfilling;
  • Groundwater levels and dewatering recommendations;
  • Seismic Site Class;
  • ULS and SLS Geotechnical Resistances for foundations;
  • Pavement structure for paved areas

4.0 Method of Investigation

4.1 GEOTECHNICAL FIELD INVESTIGATION

Prior to carrying out the investigation, Stantec Consulting Limited (Stantec) personnel marked out the proposed borehole locations at the site. As a component of our standard procedures and due diligence, Stantec contracted USL-1 to ensure that all borehole locations were clear of all public and private underground utilities.

The field drilling program was carried on June 4, 2012. The three boreholes were advanced, at the locations shown on Drawing No. 2 in Appendix B, with a track mounted Geoprobe drill rig. The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec personnel while performing Standard Penetration Tests (SPT). Split spoon samples were collected at regular intervals in each borehole; all samples were stored in moisture-proof bags and returned to Stantec’s Ottawa laboratory for further classification and testing.

A 31 mm diameter monitoring well was installed within borehole MW12-3. The monitoring well construction consisted of, well screen from 55.5 m to 58.5 m, silica sand to approximately 0.3 m above the screen, 0.9 m thick bentonite seal and backfilled with auger cutting to ground surface.

Samples will be stored for a period of one (1) month after issuance of the report unless directed otherwise by the client.

4.2 SURVEYING

The ground surface elevation at each test hole was surveyed using a Trimble GPS unit with decimeter accuracy. Geodetic elevations are shown on the borehole logs in Appendix C.
5.0 Results of Investigation

The location of the site is shown on Drawing No. 1 in Appendix B. At the time of the investigation, the site for the proposed building was occupied with one residential house. The area surrounding the house consisted of grassed areas and an asphalt driveway as shown on Drawing No. 2 in Appendix B.

5.1 SUBSURFACE INFORMATION

In general, the subsurface profile at this site consisted of a surficial layer of topsoil or asphalt over fill underlain by a deposit of silty sand with gravel till over bedrock.

The subsurface conditions observed are presented on the Borehole Records in Appendix C. An explanation of symbols and terms used is provided. The results of the laboratory testing are presented in Appendix D and on the borehole records in Appendix C.

5.1.1 Surficial Materials

Topsoil was encountered at ground surface in two of the boreholes, the topsoil varied from 100 mm to 150 mm in thickness. A layer of asphalt was encountered at ground surface of borehole BH12-1, the thickness of the asphalt encountered was 50 mm.

5.1.2 Fill

A fill material was encountered beneath the surficial materials in all boreholes. The fill generally consisted of a very loose to loose brown sand with silt to silty sand with gravel. The thickness of the fill varied from 0.8 to 1.2 m. Standard Penetration Test ‘N’ values on this material ranged from 1 to 14 indicating a very loose to compact state. Gradation tests performed on this material yielded 23 to 24% gravel, 39 to 48% sand and 28 to 38% fines. The gradation results are presented on Figure No. 1 in Appendix D. The moisture content of this material ranged from 10% to 20%.

5.1.3 Till

A layer of till was encountered in all boreholes. The till consisted of a silty sand with gravel and occasional cobbles and pieces of shale. Standard Penetration Test ‘N’ values ranged from 18 to 124 which indicates a compact to very dense state. The moisture content of this material ranged from 6% to 11%. Gradation tests performed on this material yielded the following results; 23 to 28% gravel, 49 to 53% sand and 20 to 27% fines. The gradation results are presented on Figure No. 2 in Appendix D. This material can be classified as a silty sand with gravel (SM), according to the Unified Soil Classification System (USCS).

5.1.4 Bedrock

All boreholes terminated with refusal on inferred bedrock. The depth to refusal ranged from 2.3 to 4.7 m below ground surface.
5.2 GROUNDWATER

Groundwater was observed in the open borehole BH12-2 approximately 3.3 m below ground surface. The groundwater level was measured on June 26, 2012 three weeks after the completion of drilling. The groundwater level in MW12-3 was measured at 4.7 m, which corresponds to a groundwater elevation of 55.4.

Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

6.0 Discussion and Recommendations

The following geotechnical issues should be considered during design activities:

- Conventional spread footings founded on native material (till) are appropriate for the design of structures at this site.
- Groundwater was encountered at depths within the proposed depth of construction. It is anticipated that groundwater elevations will fluctuate throughout the year and will likely rise to the level of the basement. The building design should include a perimeter and floor slab drainage system and damp-proofing.
- The recommended Site Classification for Seismic Site Response for the site is Site Class C in accordance with NBCC 2006.
- The existing building and foundations will need to be removed.

6.1 SITE GRADING AND PREPARATION

6.1.1 Building Footprint

There is currently one single-storey residential unit on-site, with an approximate footprint of 90 m². Underground services have been located on site within the footprint of the proposed building. Water and sewer services are connected to the existing unit, as well as a gas line that runs north-west from the eastern side of the unit. All existing utilities will have to be removed from the footprint of the new building.

The area surrounding the unit consists of grassed areas and an asphalt driveway. All existing topsoil, asphalt, concrete (foundations), services, fill and any deleterious materials should be removed from beneath the footprint of the building, the footing and the zone of influence of all footings. The zone of influence is defined by a line drawn at 1 horizontal to 1 vertical, outward and downward from the edge of the footings.

Prepared subgrade surfaces should be inspected by experienced geotechnical personnel prior to placement of either Structural Fill or concrete. All soft or disturbed areas revealed during subgrade excavation or inspection should be removed and replaced with approved Structural Fill, as defined below.
Structural Fill should conform to the requirements of OPSS Granular B Type II or OPSS Granular A. Structural Fill placed beneath building should contain no recycled materials such as concrete or asphalt. It should be compacted in lifts no thicker than 300 mm to at least 100% Standard Proctor Maximum Dry Density (SPMDD). This material should be tested and approved by a Geotechnical Engineer prior to delivery to the site.

Earth removals should be inspected by a geotechnical engineer to ensure that all unsuitable materials are removed prior to placement of fill. Inspection and testing services will be critical to ensure that all fill used is suitable and is placed and compacted to the required degree.

6.1.2 Paved Areas

All vegetation, topsoil, asphalt and other deleterious material should be removed from beneath pavement areas. The subgrade should be proof rolled in the presence of geotechnical personnel. All soft areas revealed during proof rolling or subgrade inspections should excavated to a maximum depth of 500 mm and replaced with compacted Subgrade Fill.

6.2 FOUNDATIONS

The foundations for the proposed building may be supported on spread footings provided that the foundation preparation work described in Section 6.1 above is carried out. Spread footings should be placed on clean undisturbed native till.

Table 6.1 provides Geotechnical Bearing Resistances for shallow foundations on native till. The values have been calculated assuming a footing embedment depth of 0.5 m.

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Footing Width (m)</th>
<th>Geotechnical Resistances - Till</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ULS (kPa)</td>
</tr>
<tr>
<td>Strip Footing</td>
<td>0.8</td>
<td>160</td>
</tr>
<tr>
<td>Strip Footing</td>
<td>1.0</td>
<td>175</td>
</tr>
<tr>
<td>Square Footing</td>
<td>1.0</td>
<td>200</td>
</tr>
<tr>
<td>Square Footing</td>
<td>2.0</td>
<td>250</td>
</tr>
<tr>
<td>Square Footing</td>
<td>3.0</td>
<td>310</td>
</tr>
</tbody>
</table>

The factored geotechnical bearing resistance at Ultimate Limit States (ULS) incorporates a resistance factor of 0.5. The geotechnical reaction at Serviceability Limit States (SLS) is the bearing pressure that corresponds to 25 mm of settlement.

The design frost depth for this site is 1.8 m. All exterior spread footings and footings for unheated structures should be protected from frost action by a minimum soil cover of 1.8 m or equivalent insulation. Perimeter footings and interior footings within 1.5 m of perimeter walls of heated structures should be protected by a minimum soil cover of 1.5 m or equivalent insulation. Where proposed footings have insufficient soil cover for frost protection, the use of insulation will be required.
The base of all footing excavations should be inspected by a geotechnical engineer prior to placing concrete to confirm the design pressures and to ensure that there is no disturbance of the founding soils.

Where construction is undertaken during winter conditions, all footing subgrades should be protected from freezing. Foundation walls and columns should be protected against heave due to soil adfreeze.

6.3 TEMPORARY EXCAVATIONS AND BACKFILLING

6.3.1 General Excavations

The native silty sand till present at the site is considered a Type 3 soil in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. Temporary excavations in the overburden may be supported or should be sloped at 1 horizontal to 1 vertical from the base of the excavation and as per the requirements of OHSA.

6.3.2 Foundation Backfill

Backfill within the footprint of the proposed buildings should consist of Structural Fill placed as described in Section 6.1. Exterior foundation backfill should consist of a material meeting the requirements of OPSS Granular B Type I.

The Foundation backfill must be placed in lifts no thicker than 300 mm and compacted using suitable compaction equipment to at least 95% of SPMDD. Care should be taken immediately adjacent to the foundation walls to avoid overcompaction of the soil which could result in damage to the walls.

6.3.3 Pipe Bedding and Backfill

Bedding for utilities should be placed in accordance with the pipe design requirements. It is recommended that a minimum of 150 mm to 200 mm of OPSS Granular A be placed below the pipe invert as bedding material. Granular pipe backfill placed above the invert should consist of Granular A material. A minimum of 300 mm vertical and side cover should be provided. These materials should be compacted to at least 95% of SPMDD.

Backfill for service trenches in landscaped areas may consist of excavated material replaced and compacted in lifts. Where the service trenches extend below paved areas, the trench should be backfilled with OPSS Select Subgrade material from the top of the pipe cover to within 1.2 m of the proposed pavement surface, placed in lifts and compacted to at least 95% of SPMDD. The material used within the upper 1.2 m and below the subgrade line should be similar to that exposed in the trench walls to prevent differential frost heave, placed in lifts and compacted to at least 95% of SPMDD. Different abutting materials within this zone will require a 3 horizontal to 1 vertical frost taper in order to minimize the effects of differential frost heaving.

It should be noted that reuse of the site generated material will be highly dependent on the material’s moisture content at time of placement.
Backfill should be compacted in lifts not exceeding 300 mm.

### 6.3.4 Groundwater

Groundwater was encountered during this geotechnical investigation below the depths of the anticipated excavations. The groundwater level was measured at 3.3 and 4.7 m below the ground surface. However, groundwater elevations will fluctuate seasonally and may rise to the level of the basement.

Foundation walls should be protected with damp-proofing and backfilled with free-draining granular material such as OPSS Granular B Type I. The zone of free-draining backfill should extend a horizontal distance of at least 500 mm out from the foundation wall. It is recommended that a perimeter drain and underslab drainage system be installed. The drainage system should be designed to allow positive drainage to a frost free outlet.

If dewatering is required during construction will likely be possible using conventional sump and pump techniques.

### 6.4 CONCRETE FLOOR SLABS

Conventional slab-on-grade units are suitable for use for the proposed structure provided the floor slab areas are prepared as outlined in Section 6.1. A layer of free-draining granular material such as OPSS Granular A, at least 200 mm in thickness should be placed immediately beneath the floor slab for leveling and support purposes. This material should be compacted to at least 100% SPMDD. The installation of a vapor barrier below the floor slab is recommended.

The floor slabs constructed as recommended above may be designed using a soil modulus of subgrade reaction, k, of 50 MPa/m, based on a loaded area of 0.3 m by 0.3 m. The slab-on-grade units should float independently of all load-bearing walls and columns.

### 6.5 CEMENT TYPE AND CORROSION POTENTIAL

One representative soil sample was submitted to Paracel Laboratories Ltd. in Ottawa, Ontario, for pH, chloride, sulphate and resistivity testing. The test results are summarized in Table 6.2.

<table>
<thead>
<tr>
<th>Borehole/ Sample No.</th>
<th>Depth</th>
<th>pH</th>
<th>Sulphate</th>
<th>Resistivity</th>
<th>Chloride</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW 12-3/ SS5</td>
<td>2.4 m – 3.0 m</td>
<td>7.46</td>
<td>361 µg/g</td>
<td>36.5 ohm•m</td>
<td>5 µg/g</td>
</tr>
</tbody>
</table>

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate result was 361 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU Portland Cement should therefore be suitable for use in concrete at this site.
The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH was 7.46 which is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The test results provided in the Table 6.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

6.6 PAVEMENT STRUCTURE RECOMMENDATIONS

It has been assumed that any parking areas will be used mostly by passenger vehicles.

The subgrade in paved areas should be prepared as described in Section 6.1 above. The minimum pavement recommendations for the standard parking areas are included in Table 6.3.

Table 6.3: Recommended Pavement Design

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard Duty Parking Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>HL-3 Asphaltic Concrete</td>
<td>50 mm</td>
</tr>
<tr>
<td>Granular Base Course, OPSS Granular A</td>
<td>150 mm</td>
</tr>
<tr>
<td>Granular Subbase Course, OPSS Granular B Type II</td>
<td>300 mm</td>
</tr>
</tbody>
</table>

It is estimated that the service life prior to major rehabilitation for the above pavement structures is 15 years provided they are properly maintained. The pavement surface and the underlying subgrade should be graded to direct runoff water towards suitable drainage.

All granular materials should be tested and approved by a geotechnical engineer prior to delivery to the site. Both base and subbase materials should be compacted to at least 100% SPMDD. Asphalt should be compacted to at least 97% Marshal bulk density.

It is recommended that the lateral extent of the subbase and base layers not be terminated in a vertical fashion immediately behind the curb line. A taper with a grade of 5 horizontal to 1 vertical is recommended in the subgrade line to minimize differential frost heave problems under sidewalks.

6.7 SEISMIC SITE CLASSIFICATION

Liquefaction Induced Settlements

An assessment for seismic liquefaction has been carried out for this site. Seismic liquefaction is the sudden loss in stiffness and strength of soil due to the loading effects of an earthquake. Liquefaction can cause significant settlements and structural failure.

The analysis followed was the one set forth in the Canadian Foundation and Engineering Manual, 2006 (CFEM). For the analysis, a magnitude 6.2 design earthquake with a Peak Ground Acceleration of 0.42g, were assumed. Based on the SPT N for the soil, plots of Factor of Safety against Liquefaction (FSL) with depth were developed for the site. Our analysis indicates that the site soil is not considered susceptible to liquefaction.
Seismic Site Classification

As outlined in the 2006 Ontario Building Code, building foundations must be designed to resist a minimum earthquake force. In accordance with Table 4.1.8.4.A of the 2006 Ontario Building Code the seismic site response for the site is Class C - Stiff Soil. The site class is based on the Average Standard Penetration Resistance shown in Table 6.4.

Table 6.4: Parameters for Seismic Site Classification

<table>
<thead>
<tr>
<th>Depth</th>
<th>Soil</th>
<th>N&lt;sub&gt;60&lt;/sub&gt; Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 m to 5 m</td>
<td>Till</td>
<td>45</td>
</tr>
<tr>
<td>5 m to 31.5 m</td>
<td>Bedrock</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Design N&lt;sub&gt;60&lt;/sub&gt;</td>
<td>83</td>
</tr>
</tbody>
</table>

6.8 LATERAL EARTH PRESSURES

The earth pressures recommended in Table 6.5 are based on the assumption that a permanent horizontal back slope will be utilized behind the wall. In order to use the coefficients of pressures for the granular materials, the granular backfill must be provided within a wedge extending from the base of the wall at 45 degrees (or smaller) to the horizontal. If a smaller wedge is used, the coefficients of earth pressures of the materials outside the backfill wedge must be used for lateral pressure design calculations.

For walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied structures, the at rest pressure should be used for design, unless the wall can deflect enough (approximately 0.05% of the wall height) to establish the active pressure.

Lateral earth pressures may be calculated using parameters provided in Table 6.5.

Table 6.5: Lateral Earth Pressure Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Native Till</th>
<th>OPSS Granular A</th>
<th>OPSS Granular B Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight (kN/m&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>22.0</td>
<td>22.8</td>
<td>21.2</td>
</tr>
<tr>
<td>Angle of Internal Friction, Φ</td>
<td>34°</td>
<td>35°</td>
<td>32°</td>
</tr>
<tr>
<td>Coefficient of Passive Earth Pressure, K&lt;sub&gt;p&lt;/sub&gt;</td>
<td>3.5</td>
<td>3.7</td>
<td>3.3</td>
</tr>
<tr>
<td>Coefficient of at Rest Earth Pressure, K&lt;sub&gt;o&lt;/sub&gt;</td>
<td>0.44</td>
<td>0.43</td>
<td>0.47</td>
</tr>
<tr>
<td>Coefficient of Active Earth Pressure, K&lt;sub&gt;a&lt;/sub&gt;</td>
<td>0.28</td>
<td>0.27</td>
<td>0.31</td>
</tr>
</tbody>
</table>

Sliding resistance can be calculated using the following unfactored friction coefficients, outlined in Table 6.6.

Table 6.6: Unfactored Friction Coefficients

<table>
<thead>
<tr>
<th>Condition</th>
<th>Unfactored Friction Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between Concrete and Structural Fill</td>
<td>0.55</td>
</tr>
<tr>
<td>Between Concrete and Native Soil</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Seismic Design Parameters

For retaining structures total active and passive thrusts under earthquake conditions can be calculated using the following equations:

\[ P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \]
\[ P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v) \]

where;

- \( K_{AE} \) = active earth pressure coefficient (combined static and seismic)
- \( K_{PE} \) = passive earth pressure coefficient (combined static and seismic)
- \( H \) = height of wall
- \( k_h \) = horizontal acceleration coefficient
- \( k_v \) = vertical acceleration coefficient
- \( \gamma \) = total unit weight

For this site, the following design parameters were used to develop the recommended \( K_{AE} \) and \( K_{PE} \) values (assumes Horizontal Backslope to wall).

- Zonal Acceleration Ratio, A 0.42
- Horizontal Acceleration Coefficient, \( k_h \) 0.21
- Vertical Acceleration Coefficient, \( k_v \) 0.14

The above \( k_h \) value corresponds to \( \frac{1}{2} \) of the A value, and the \( k_v \) value corresponds to 0.67 of the \( k_h \) value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 6.7: Combined Coefficients of Static and Seismic Earth Pressure

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Native Till</th>
<th>OPSS Granular A</th>
<th>OPSS Granular B Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Unit Weight, ( \gamma ) (kN/m³)</td>
<td>22.0</td>
<td>22.8</td>
<td>21.2</td>
</tr>
<tr>
<td>Effective Friction Angle</td>
<td>34°</td>
<td>35°</td>
<td>32°</td>
</tr>
<tr>
<td>Angle of Internal Friction between wall and backfill</td>
<td>0 degrees</td>
<td>0 degrees</td>
<td>0 degrees</td>
</tr>
<tr>
<td>Yielding Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Active Earth Pressure (( K_{AE} ))</td>
<td>0.45</td>
<td>0.43</td>
<td>0.48</td>
</tr>
<tr>
<td>Height of Application of ( P_{AE} ) from base as a ratio of wall height (H)</td>
<td>0.403</td>
<td>0.404</td>
<td>0.401</td>
</tr>
<tr>
<td>Passive Earth Pressure (( K_{PE} ))</td>
<td>3.04</td>
<td>3.19</td>
<td>2.78</td>
</tr>
<tr>
<td>Height of Application of ( P_{PE} ) from base as a ratio of wall height (H)</td>
<td>0.239</td>
<td>0.241</td>
<td>0.236</td>
</tr>
</tbody>
</table>
If the wall is designed as non-yielding wall it could be designed with the Wood (1973) method:

\[
\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_p
\]

- \(\Delta P_{eq}\): Steady state dynamic thrust
- \(\gamma\): 22 kN/m³
- \(H\): Height of wall (m)
- \(a_h\): Amplitude of harmonic base acceleration = 0.42 m/s²
- \(g\): Acceleration due to gravity (m/s²) = 9.81 m/s²
- \(F_p\): Dimensionless thrust factor = 1.1
- \(h_{eq}\): 0.63H
7.0 Closure

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Pan Art and Science, who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying of unexpected site conditions
- Planning, design or construction

This report has been prepared by Katurah Firdawsi and reviewed by Chris McGrath.

Respectfully submitted,

STANTEC CONSULTING LTD.

Katurah Firdawsi, EIT, B.Sc.Eng

Chris McGrath, P.Eng.
Associate – Senior Geotechnical Engineer
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

Key Plan

Borehole Location Plan on Existing Site
APPENDIX C

Symbols and Terms Used on Borehole and Test Pit Records

Borehole Records
SOIL DESCRIPTION

Terminology describing common soil genesis:

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<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 burned 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 oxidization 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 2489). The classification excludes particles larger than 76 mm (3 inches). 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 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

<table>
<thead>
<tr>
<th>Trace, or occasional</th>
<th>Less than 10%</th>
</tr>
</thead>
<tbody>
<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 N-Value (also known as N-Index). 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.

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Undrained Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kips/sq.ft.</td>
</tr>
<tr>
<td>Very Soft</td>
<td>&lt; 0.25</td>
</tr>
<tr>
<td>Soft</td>
<td>0.25 - 0.5</td>
</tr>
<tr>
<td>Firm</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td>Stiff</td>
<td>1.0 - 2.0</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>2.0 - 4.0</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 4.0</td>
</tr>
</tbody>
</table>
### Terminology describing rock quality:

<table>
<thead>
<tr>
<th>RQD</th>
<th>Rock Mass Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-25</td>
<td>Very Poor</td>
</tr>
<tr>
<td>25-50</td>
<td>Poor</td>
</tr>
<tr>
<td>50-75</td>
<td>Fair</td>
</tr>
<tr>
<td>75-90</td>
<td>Good</td>
</tr>
<tr>
<td>90-100</td>
<td>Excellent</td>
</tr>
</tbody>
</table>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from in situ fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

### Terminology describing rock mass:

<table>
<thead>
<tr>
<th>Spacing (mm)</th>
<th>Joint Classification</th>
<th>Bedding, Laminations, Bands</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>Very Thick</td>
</tr>
<tr>
<td>800-2000</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>Laminated</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>Unconfined Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Weak</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>Very Weak</td>
<td>1 – 5</td>
</tr>
<tr>
<td>Weak</td>
<td>5 – 25</td>
</tr>
<tr>
<td>Medium Strong</td>
<td>25 – 50</td>
</tr>
<tr>
<td>Strong</td>
<td>50 – 100</td>
</tr>
<tr>
<td>Very Strong</td>
<td>100 – 250</td>
</tr>
<tr>
<td>Extremely Strong</td>
<td>&gt; 250</td>
</tr>
</tbody>
</table>

### Terminology describing rock weathering:

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh</td>
<td>No visible signs of rock weathering. Slight discolouration along major discontinuities.</td>
</tr>
<tr>
<td>Slightly Weathered</td>
<td>Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.</td>
</tr>
<tr>
<td>Moderately Weathered</td>
<td>Less than half the rock is decomposed and/or disintegrated into soil.</td>
</tr>
<tr>
<td>Highly Weathered</td>
<td>More than half the rock is decomposed and/or disintegrated into soil.</td>
</tr>
<tr>
<td>Completely Weathered</td>
<td>All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.</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>SS</th>
<th>Split spoon sample (obtained by performing the Standard Penetration Test)</th>
</tr>
</thead>
<tbody>
<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>WS</td>
<td>Wash sample</td>
</tr>
<tr>
<td>HQ, NQ, MQ 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 (64 kg) hammer falling 30 inches (750 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. 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 value 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 (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

**OTHER TESTS**

<table>
<thead>
<tr>
<th>S</th>
<th>Sieve analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>Hydrometer analysis</td>
</tr>
<tr>
<td>k</td>
<td>Laboratory permeability</td>
</tr>
<tr>
<td>y</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>

- Single packer permeability test; test interval from depth shown to bottom of borehole
- Double packer permeability test; test interval as indicated
- Falling head permeability test using casing
- Falling head permeability test using well point or piezometer
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.3</td>
<td>50 mm Asphalt</td>
</tr>
<tr>
<td></td>
<td>Fill: Brown sand with gravel trace silt</td>
</tr>
<tr>
<td>59.1</td>
<td>Dense to very dense dark brown to black silt with gravel (SM) TILL</td>
</tr>
<tr>
<td></td>
<td>- occasional cobbles / shale pieces</td>
</tr>
</tbody>
</table>

**End of Borehole**

**Auger Refusal on Inferred Bedrock**

<table>
<thead>
<tr>
<th>STRATA PLOT</th>
<th>WATER LEVEL</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Type: SS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
</tr>
</tbody>
</table>

**Undrained Shear Strength - kPa**

**Dynamic Penetration Test, Blows 0.3m**

**Standard Penetration Test, Blows 0.3m**

**WATER CONTENT & ATTERBERG LIMITS**

- $W_p$
- $W_r$
- $W_l$

**Inferred Groundwater Level**

**Groundwater Level Measured in Standpipe**

**Field Vane Test, kPa**

**Remoulded Vane Test, kPa**

**App'd**

**Pocket Penetrometer Test, kPa**

**Date**
BOREHOLE RECORD

CLIENT: Pan Art and Science Inc

LOCATION: 160 Ontario St, Ottawa, ON

DATES: BORING: June 4, 2012

WATER LEVEL:

DEPTH (m) | ELEVATION (m) | SOIL DESCRIPTION | STRATA PLOT | WATER LEVEL | SAMPLES | UNDRAINED SHEAR STRENGTH - kPa
---|---|---|---|---|---|---
0 | 59.90 | 150 mm Topsoil
| 59.8 | FILL: Brown sand some silt trace rootlets
| 59.1 | FILL: Brown to dark brown silty sand with gravel
| 59.0 | - trace rootlets
| | - occasional cobbles and shale pieces
| | Compact to very dense dark brown to black silty sand with gravel (SM) TILL
| | - occasional cobbles with shale pieces
| 56.6 | End of Borehole
| | Auger Refusal on Inferred Bedrock

STANDARD PENETRATION TEST, BLOWS/0.3m

WATER CONTENT & ATTERBERG LIMITS

DYNAMIC PENETRATION TEST, BLOWS/0.3m

80 mm

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa
MONITORING WELL RECORD

Mw 12-3

CLIENT: Pan Art and Science Inc
LOCATION: 106 Ontario St, Ottawa, ON

DEPTH (m)

ELEVATION

60.10

100 mm Topsoil

59.9

FILL: Brown sand some silt

FILL: Brown to black silty sand with gravel (SM)
- occasional cobbles with shale pieces

59.2

Dense to very dense dark brown to black silty sand with gravel (SM) TILL
- occasional cobbles with shale particles

55.4

End of Borehole

Auger Refusal on Inferred Bedrock

SOIL DESCRIPTION

STRATA PLOT

WATER LEVEL

SAMPLES

RECOVERY (mm)

VALUE OR RQD

UNDRAINED SHEAR STRENGTH - kPa

WATER CONTENT & ATTERBERG LIMITS

DYNAMIC PENETRATION TEST, BLOWS/0.3m

STANDARD PENETRATION TEST, BLOWS/0.3m

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

Inferred Groundwater Level

Groundwater Level Measured in Standpipe