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1. **INTRODUCTION**

SPL Consultants Limited (SPL) was retained by Geofirma Engineering Ltd. (Geofirma) to provide geotechnical recommendations for a high-rise building development at 460 St. Laurent Boulevard in Ottawa ON. The terms of reference for the project are as outlined in our proposal number P-11.10.032 dated October 24, 2011 as well as subsequent project correspondence.

This report presents the results of the investigation and provides geotechnical recommendations related to the design of the proposed building. This report does not contain recommendations related to environmental or hydrogeological issues at the site; environmental and hydrogeological investigations have been undertaken by Geofirma as part of the current project, and are submitted under separate cover.

2. **PROJECT AND SITE DESCRIPTION**

The site is located at 460 St. Laurent Boulevard in Ottawa, ON at the intersection of Dunbarton Crescent, and is shown in Figures 1 and 2. The site is generally flat and appears to have been raised slightly above the surrounding street level. The site is a former automotive service station; a small building exists on the south side of the site, and most of the remainder of the site is paved. The site is bordered on the north by Dunbarton Crescent, on the east by St. Laurent Boulevard, on the south by an existing 7 storey residential building and to the west by low-rise residential buildings.

The proposed development will include a high-rise building, on the order of 15 storeys which is understood to occupy the majority of the site. The building will include 2 to 3 levels of underground parking.

3. **INVESTIGATION PROCEDURES**

The geotechnical investigation was carried out by SPL in conjunction with Geofirma on November 21st to 23rd, 2011. The overall scope of work for this assignment included both geotechnical and environmental components. The field work for both aspects of the investigation was combined (i.e. the same boreholes which were drilled as part of the environmental investigations were also used to collect geotechnical data).

A total of 6 boreholes were drilled at the site (MW11-1 through MW11-5 and MW11-7; MW11-6 was deleted from the field program due to utility conflicts) at the locations shown in Figure 2. The boreholes were advanced using a truck-mounted drill rig retained and supervised by Geofirma. The boreholes were drilled to depths ranging from 6.8 m to 15.2 m below the existing ground surface, using “H” size rock coring techniques.

Standpipe piezometers were installed in all of the boreholes to allow for subsequent measurement of stabilized groundwater levels at the site, as well as environmental sampling and testing.
For the purposes of preparing borehole logs a local datum of approximately 100 m elevation was assigned to the site.

Field work was supervised by Geofirma, who also selected the borehole locations, obtained stabilized water levels and prepared the borehole logs.

Borehole locations are shown in Drawing No. 2. Borehole records prepared by Geofirma are included in Appendix A of this report.

Upon completion of drilling rock cores obtained from two of the boreholes (MW11-2 and MW11-3) were returned to SPL’s laboratory for further examination, geotechnical classification and testing (Unconfined Compressive Strength). The results of these tests are discussed in subsequent sections of this report.

A geophysical survey was also carried out at the site to confirm the shear-wave velocity of the limestone bedrock in order to assess the appropriate Site Classification for Seismic Site Response (NBCC 2010). The geophysical survey was carried out by Geophysics GPR International Inc. acting as a sub-consultant to SPL. The results of the survey are included in Appendix B and are discussed in further detail in Section 5.2 below.

4. SUBSURFACE CONDITIONS

The subsurface conditions at the site are discussed in the following sections. Descriptions of the stratigraphy encountered at each of the borehole locations are included in the individual borehole records included in Appendix A.

4.1 Soil and Bedrock Conditions

The following provides an overall description of the major soil and rock types and the general stratigraphy encountered across the site.

Fill

Fill material was encountered in all of the boreholes drilled at the site, and comprised primarily sand and gravel. Other materials (such as cobbles and boulders, concrete, construction debris, etc.) were not noted by Geofirma during the field investigation. Fill material is, however, by its nature highly variable and may contain materials other than the sand and gravel encountered at the borehole locations. The thickness of the fill ranged from 1.0 m to 2.5 m at the borehole locations. It is our understanding that previous environmental investigations identified the historical presence of underground storage tanks which have been removed from the site. At these locations the fill may be deeper.
Limestone Bedrock

Auger refusal was encountered on limestone rock in all six boreholes drilled as part of this investigation at depths ranging from 1.0 m to 2.5 m below the existing ground surface. At all locations the rock was cored using triple-tube “H” size coring equipment.

The rock at the site would be described as slightly weathered to fresh, strong-to-very strong limestone with shale partings and closely-to-moderately closely spaced horizontal joints and occasional vertical or steeply inclined joints.

Rock cores from two of the boreholes (MW11-2 and MW11-3) were provided to SPL for additional classification and testing. Tables 1 and 2 below summarize the Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) of the rock cores, as well as the average joint spacing over each run of core, at these two locations.

Table 1 – Rock Core Classification MW11-2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>TCR (%)</th>
<th>SCR (%)</th>
<th>RQD (%)</th>
<th>Rock Quality</th>
<th>Avg. Joint Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.02 – 1.40</td>
<td>100</td>
<td>100</td>
<td>0</td>
<td>Very Poor</td>
<td>Close to Very Close</td>
</tr>
<tr>
<td>1.40 – 2.90</td>
<td>100</td>
<td>71</td>
<td>70</td>
<td>Fair</td>
<td>Close</td>
</tr>
<tr>
<td>2.90 – 4.40</td>
<td>100</td>
<td>67</td>
<td>60</td>
<td>Fair</td>
<td>Close</td>
</tr>
<tr>
<td>4.40 – 5.90</td>
<td>100</td>
<td>91</td>
<td>87</td>
<td>Good</td>
<td>Close</td>
</tr>
<tr>
<td>5.90 – 7.40</td>
<td>100</td>
<td>78</td>
<td>71</td>
<td>Fair</td>
<td>Close</td>
</tr>
<tr>
<td>7.40 – 8.90</td>
<td>100</td>
<td>98</td>
<td>90</td>
<td>Good – Excellent</td>
<td>Close</td>
</tr>
<tr>
<td>8.90 – 10.40</td>
<td>100</td>
<td>61</td>
<td>61</td>
<td>Fair</td>
<td>Close</td>
</tr>
<tr>
<td>10.40 – 11.90</td>
<td>100</td>
<td>96</td>
<td>95</td>
<td>Excellent</td>
<td>Close</td>
</tr>
<tr>
<td>11.90 – 13.40</td>
<td>100</td>
<td>97</td>
<td>97</td>
<td>Excellent</td>
<td>Close to Mod. Close</td>
</tr>
<tr>
<td>13.40 – 14.90</td>
<td>98</td>
<td>95</td>
<td>90</td>
<td>Good – Excellent</td>
<td>Moderately Close</td>
</tr>
<tr>
<td>14.90 – 15.20</td>
<td>100</td>
<td>100</td>
<td>65</td>
<td>Fair</td>
<td>Close to Mod. Close</td>
</tr>
</tbody>
</table>

Table 2 – Rock Core Classification MW11-3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>TCR (%)</th>
<th>SCR (%)</th>
<th>RQD (%)</th>
<th>Rock Quality</th>
<th>Avg. Joint Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.50 – 2.89</td>
<td>99</td>
<td>68</td>
<td>63</td>
<td>Fair</td>
<td>Close</td>
</tr>
<tr>
<td>2.89 – 4.39</td>
<td>99</td>
<td>84</td>
<td>75</td>
<td>Fair - Good</td>
<td>Close to Mod. Close</td>
</tr>
<tr>
<td>4.39 – 5.89</td>
<td>100</td>
<td>94</td>
<td>94</td>
<td>Excellent</td>
<td>Moderately Close</td>
</tr>
<tr>
<td>5.89 – 7.39</td>
<td>97</td>
<td>90</td>
<td>77</td>
<td>Good</td>
<td>Close</td>
</tr>
<tr>
<td>7.39 – 8.89</td>
<td>99</td>
<td>87</td>
<td>79</td>
<td>Good</td>
<td>Close</td>
</tr>
<tr>
<td>8.89 – 10.39</td>
<td>97</td>
<td>93</td>
<td>91</td>
<td>Good – Excellent</td>
<td>Close</td>
</tr>
<tr>
<td>10.39 – 11.89</td>
<td>100</td>
<td>94</td>
<td>85</td>
<td>Good</td>
<td>Close</td>
</tr>
<tr>
<td>11.89 – 13.39</td>
<td>100</td>
<td>96</td>
<td>84</td>
<td>Good</td>
<td>Close</td>
</tr>
<tr>
<td>13.39 – 14.89</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>Excellent</td>
<td>Close to Mod. Close</td>
</tr>
<tr>
<td>14.89 – 15.24</td>
<td>100</td>
<td>94</td>
<td>90</td>
<td>Good - Excellent</td>
<td>Close</td>
</tr>
</tbody>
</table>
Photographs of the rock cores recovered from boreholes MW11-2 and MW11-3 are included in Appendix B of this report.

Unconfined Compressive Strength (UCS) testing was carried out on selected samples of the rock from boreholes MW11-2 and MW11-3. The results of these tests are presented in Table 3 below.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Depth (m)</th>
<th>Unit Weight (kg/m³)</th>
<th>UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW11-2</td>
<td>2.17–2.45</td>
<td>2,682</td>
<td>104.8</td>
</tr>
<tr>
<td>MW11-2</td>
<td>4.40–4.62</td>
<td>2,684</td>
<td>101.9</td>
</tr>
<tr>
<td>MW11-2</td>
<td>6.54–6.79</td>
<td>2,688</td>
<td>64.4</td>
</tr>
<tr>
<td>MW11-2</td>
<td>9.92–10.21</td>
<td>2,700</td>
<td>91.4</td>
</tr>
<tr>
<td>MW11-3</td>
<td>2.40–2.63</td>
<td>2,678</td>
<td>105.9</td>
</tr>
<tr>
<td>MW11-3</td>
<td>3.12–3.44</td>
<td>2,674</td>
<td>101.5</td>
</tr>
<tr>
<td>MW11-3</td>
<td>5.39–5.65</td>
<td>2,674</td>
<td>138.4</td>
</tr>
<tr>
<td>MW11-3</td>
<td>7.52–7.82</td>
<td>2,679</td>
<td>110.2</td>
</tr>
<tr>
<td>MW11-3</td>
<td>11.98–12.39</td>
<td>2,680</td>
<td>129.9</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>2,682</strong></td>
<td><strong>105.3</strong></td>
</tr>
</tbody>
</table>

4.2 Groundwater Conditions

A standpipe piezometer was installed in each of the boreholes. At boreholes MW11-2 and MW11-3, two piezometers were installed, a shallow piezometer at typically around 7 m depth below the ground surface and a deep piezometer at 15 m depth. All piezometers were sealed in the limestone bedrock. Groundwater measurements were obtained on November 23, 2011 (one to two days after installation). The measured groundwater level was found to be approximately 3.2 m to 5.5 m below the ground surface at that time. Groundwater measurements in boreholes MW11-2 and MW11-3 (where two piezometers are installed at different elevations) suggest there is no significant hydraulic gradient between the measurements at 7 m depth and 15 m depth.

It should be noted that groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events. Higher groundwater levels should be expected during wetter periods of the year (e.g. spring run-off and during periods of extended rainfall). Groundwater levels can also be affected over the long term by development in the area or by various uses at the site.

4.3 Summary

The following table provides an overview of the soil strata encountered at each of the borehole locations. Detailed descriptions are included on the relevant borehole records compiled by Geofirma and included in Appendix A.
Table 4 – Simplified Soil Profiles

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Elevation¹</th>
<th>Simplified Stratigraphy</th>
<th>Groundwater Level (Depth/Elevation¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sand and Gravel Fill</td>
<td>Limestone Bedrock</td>
</tr>
<tr>
<td>MW11-1</td>
<td>99.7</td>
<td>0.0 – 1.1</td>
<td>1.1 – 7.1</td>
</tr>
<tr>
<td>MW11-2</td>
<td>99.9</td>
<td>0.0 – 1.1</td>
<td>1.1 – 15.2</td>
</tr>
<tr>
<td>MW11-3</td>
<td>99.6</td>
<td>0.0 – 1.0</td>
<td>1.0 – 15.2</td>
</tr>
<tr>
<td>MW11-4</td>
<td>99.3</td>
<td>0.0 – 1.5</td>
<td>1.5 – 7.1</td>
</tr>
<tr>
<td>MW11-5</td>
<td>100.9</td>
<td>0.0 – 2.6</td>
<td>2.6 – 6.8</td>
</tr>
<tr>
<td>MW11-7</td>
<td>100.0</td>
<td>0.0 – 1.6</td>
<td>1.6 – 7.2</td>
</tr>
</tbody>
</table>

5. DISCUSSION AND RECOMMENDATIONS

5.1 General

This section of the report presents geotechnical recommendations for the proposed development. The recommendations included in this section are intended to provide the designer with the information required to select the most suitable foundation type(s) and to complete the design of the various components of the project. Where comments are made concerning construction considerations they are intended to provide the designer with an understanding of the geotechnical issues associated with the various aspects of the project. Those requiring detailed information regarding construction aspects of the project should review the factual information and draw their own conclusions as to how the subsurface conditions may affect their work.

5.2 Site Classification for Seismic Site Response

A shear wave velocity survey was carried out at the site as part of this project in order to assess the site classification for seismic site response. The survey was carried out by Geophysics GPR, acting as a sub-consultant to SPL. For a building which is founded on rock, the site may be considered to be Site Class “B” for the purposes of site-specific seismic response to earthquakes.

5.3 Frost Protection

Exterior foundations of heated structures should be provided with a minimum of 1.5 m of cover (or the thermal equivalent if insulation is used) for the purposes of protection from frost. Foundations of unheated structures should be provided with a minimum of 1.8 m of earth cover (or equivalent insulation).

¹ Elevations are based on a local datum established for the investigation.
5.4  Foundations

Details of the proposed structure (i.e. floor elevations, foundation layouts, etc.) are not available at the time of this report, however it is understood that the building will have two or three levels of underground parking (which would place the foundations roughly 6 m to 9 m below the existing ground surface). At this depth the building will be founded on limestone bedrock.

The building may be supported on shallow foundations (spread footings, strip footings, etc.). All foundations should be placed on sound limestone bedrock, in an undisturbed condition. For these conditions the following bearing capacities may be assumed:

- The unfactored ultimate geotechnical bearing resistance may be taken as 8 MPa. A resistance factor of 0.5 should be applied to this value for a factored ultimate bearing resistance of 4 MPa at ULS (Ultimate Limit States).

- The geotechnical resistance at Serviceability Limit State (SLS) may be taken as 2.5 MPa.

Provided that the rock surface is not disturbed during construction the total and differential settlements associated with the SLS resistance values are expected to be less than 25 mm and 15 mm, respectively.

All rock surfaces should be checked, evaluated and approved at the time of construction by SPL to ensure that the conditions encountered in the field are consistent with those assumed in preparing the above recommendations. Rock surfaces should be cleaned of any loose or broken rock, or other deleterious materials prior to placing foundation concrete. Caution should be taken to ensure that blasting does not result in excessive disturbance/fracturing of the bedrock, which may result in additional over-excavation and replacement.

5.5  Slabs-on-Grade

Concrete slabs-on-grade should be supported on at least 200 mm of compacted, free-draining, well graded crushed sand and gravel (Granular “A”). The crushed sand and gravel should be placed over a properly prepared rock subgrade and compacted to 100% of the materials Standard Proctor Maximum Dry Density (SPMDD) using a heavy vibratory roller.

To prevent build-up of hydrostatic pressures and moisture below the floor slab, it is recommended that the granular layer be drained by a series of 100 mm rigid pipe drains, wrapped in geotextile, with positive drainage to the City sewer or a suitable sump. For preliminary design the drains may be assumed to be at 6 m spacing, however, this spacing may be adjusted if warranted based on hydrogeological considerations.
5.6 Lateral Earth Pressures

The lateral earth pressure acting on below-grade walls, retaining walls, etc. may be calculated using the following expression:

\[ P = K(\gamma h + q) \]

Where \( P \) = lateral earth pressure (kPa) acting at depth \( h \)

\[ K = \text{earth pressure coefficient; for a wall which can tolerate some lateral movement use the coefficient of active earth pressure (} K_a \text{) equal to 0.3; for restrained walls which cannot tolerate movement use the coefficient of earth pressure at rest (} K_0 \text{) equal to 0.5} \]

\( \gamma \) = the density of the backfill; use 21 kN/m\(^3\) for compacted granular backfill

\( h \) = the depth to the point of interest (m)

\( q \) = the magnitude of any design surcharge at the ground surface; a minimum nominal surcharge of 10 kPa is recommended, a higher value should be used if appropriate for the building/site design

The above values assume that the wall will remain drained. If this is not the case, then the submerged unit weight should be used in the calculation and water pressures (as well as the potential for leakage) accounted for in the design of the wall and floor slab.

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the total earth pressure during a seismic event (including both the seismic and static components) may be assumed to be:

\[ \sigma_n(z) = K_a \gamma z + (K_{AE} - K_a) \gamma (H-z) \]

Where \( \sigma_n(z) \) = the total earth pressure at depth \( z \) (kPa);

\( K_a \) = the active earth pressure coefficient (0.3);

\( \gamma \) = the unit weight of soil (20 kN/m\(^3\));

\( K_{AE} \) = the seismic earth pressure coefficient (0.8);

\( H \) = the total height of the wall (m)

\( z \) = the depth below the top of the wall (m)

The above earth pressure values (both static and seismic) are unfactored values.

5.7 Foundation Wall Backfill and Drainage

The earth pressure values provided in Section 5.6 above assume free-draining backfill will be used. Where sufficient space exists between the formwork and the walls of the excavation, the backfill may
consist of free-draining sand and gravel (Granular “A” or “B”) compacted to 95% SPMDD in 300 mm lifts. If sufficient space does not exist between the formwork and the backfill to allow for compaction, then the backfill may consist of clear stone placed using a chute or similar method. Where this clear stone could come into contact with soil it should be wrapped with a non-woven geotextile to prevent migration of fines into the stone.

In either case the backfill should be provided with a perforated rigid pipe subdrain encased in 300 mm of clear stone, which is completely wrapped with a non-woven geotextile.

If the basement wall is to be cast against the excavated rock face or shoring then a suitable drainage board (such as Miridrain or DeltaDrain) must be placed between the rock and the basement wall to ensure the wall remains in a drained condition.

All drains should provide positive drainage to the City sewer or a suitable sump. Typical damp-proofing should be provided for below-grade walls.

5.8 Excavations and Groundwater Control

Excavations for the proposed building will be through 1m to 3 m of sand and gravel fill, as well as the underlying limestone rock.

All temporary excavations should be carried out in accordance with the most recent Occupation Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations. The soils encountered at the site include sand and gravel fill above the water table. For the purposes of excavation planning the sand and gravel fill may be considered Type 3 soil (i.e. 1:1 temporary slopes) however this classification should be confirmed by qualified individuals as the site is excavated and if necessary adjusted. If the site does not have sufficient space to accommodate sloped excavations then a shoring system will need to be designed and installed by a specialist contractor to support the upper soil layer. Shoring for this type of project would typically include soldier pile walls with steel lagging installed from the ground surface to a short distance below the soil/rock interface.

Excavations in rock would typically be carried out using drill and blast techniques. The rock quality at the site is generally good, and no unusual problems are anticipated with the majority of the rock excavations.

It is noted that the site is relatively small and is bordered on three sides by existing roads/buildings. Care should be taken when excavating near adjacent properties and structures in order to prevent disturbance as well as over-excavation. Line drilling of the perimeter of the excavation should be considered in order to control the limits of excavation and minimize over-excavation at the site.

The existing building on the south side of the site may have underground parking. The foundation elevation and layout of the existing building is not known at this time, nor is the extent of the currently proposed excavation. This information should be provided to the excavation and shoring contractor to ensure that the building excavations do not undermine or damage the foundations of the adjacent
existing structure and to determine the need for underpinning of the existing building. Depending upon the extent of the proposed excavation, limitations may need to be placed on blasting to prevent damage to the existing building. In areas immediately adjacent to the existing building the rock may need to be excavated mechanically, which is typically slower and more expensive. A pre-construction condition survey of the existing building is recommended.

If the existing building has underground parking it is likely that it is excavated into rock, and was constructed using drill and blast techniques. It is possible that blasting for the previous building left a zone of rock which has been damaged and is more fractured that the natural rock. The zone would normally only be a few meters thick (if at all), but it is likely that no records exist to confirm this. If previous blasting has damaged the rock adjacent to the existing building then additional measures such as rock bolts, wire mesh, shotcrete, etc. may be required to support the zone of rock between the existing building and the proposed excavation in this area. There may also be other areas of poor quality rock which were not identified during the drilling program which require similar localized support. It is not possible to identify these areas (if they exist) in detail until construction begins and the rock is exposed, however, contractors should be aware that there is a potential for previous disturbance to the rock at the south end of the site.

The foundation design parameters provided in Section 5.4 above assume that the rock subgrade is not unduly disturbed during excavation. Proper control of the blasting program will be required to ensure that the rock is not damage during excavation. Any loose or damaged rock should be removed and replaced with concrete fill.

Groundwater inflow is expected to be manageable using properly filtered sumps and ditches. Additional discussion related to the quantity and quality of groundwater which should be expected during excavation is provided in the associated environmental and hydrogeology reports prepared by Geofirma. It should be noted that the Geofirma report recommends a permit to take water be obtained before beginning excavation.

5.9 Site Services

Water-bearing services should be placed a minimum of 1.8 m below grade to provide protection from frost. Alternatively, equivalent insulation cover may be provided in lieu of burial.

Details of the proposed site services are not available at this time, however it is assumed that they will include localized shallow trenches throughout the site. Trenches in soil can be temporarily supported using sloped excavations (see Section 5.8) or trench boxes.

Bedding for site services should consist of a layer of Granular “A” compacted to 95% SPMDD which extends from 150 mm below the invert of the pipe to the spring line of the pipe. Where sewer trenches are based in rock this bedding should be increased to 300 mm. The use of clear stone as a bedding material is not recommended as the finer particles of the native soils and backfill may migrate into the voids of the clear stone, resulting in loss of pipe support. Cover material above the spring line should
consist of Granular “A” or Granular “B” material with a maximum particle size of 25 mm. Cover material should be compacted to a minimum of 95% SPMDD (100% if below the building structure or slab-on-grade).

5.10 Pavement Structures

Detailed traffic loads have not been provided at this time. It is, however, our understanding that the site will only experience low-volume residential traffic. Table 5 presents a preliminary pavement structure for low-volume roads and parking areas.

Table 5 – Preliminary Pavement Structure

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Concrete</td>
<td>80 mm</td>
</tr>
<tr>
<td>Base (OPSS Granular “A”)</td>
<td>150 mm</td>
</tr>
<tr>
<td>Sub-Base (OPSS Granular “B”)</td>
<td>200 mm</td>
</tr>
</tbody>
</table>

If required, SPL can provide further guidance during detailed design based on actual traffic loads.

At all locations any existing topsoil and other unsuitable soils should be stripped and the subgrade proof-rolled to confirm the competence of the subgrade soils prior to placement of the granular materials. Any soft or disturbed areas should be excavated and replaced with additional compacted granular fill.

Granular base and sub-base layers should be compacted to 100% SPMDD. Any fill required to raise the grade below the sub-base layer should be granular fill compacted to 98% SPMDD.

It is assumed (based on the existing grades and water levels at the site) that the pavement will remain on the order of 3 m or so above the water table. If portions of the pavements are to be significantly lowered for any reason, then drainage may be required to prevent frost heave. SPL can provide further guidance if portions of the site will be lowered.

6. GENERAL COMMENTS

It is understood that SPL Consultants Limited will provide a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the opportunity to undertake this review, SPL Consultants Limited will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and laboratory test results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.
7. LIMITATIONS OF REPORT

The limitations of this report are included in Appendix D.

8. CLOSURE

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

SPL CONSULTANTS LIMITED

[Signature]

Chris Hendry, M.Eng., P.Eng.

[Signature]

Shaheen Ahmad, M.A.Sc., P.Eng.
Drawings
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Project: Geotechnical Investigation, 460 St. Laurent Blvd, Ottawa, ON

SPL Consultants Limited
Geotechnical, Environmental, Materials, Hydrogeology
Figure 2
Site Layout

Projection: NAD 83 MTM Zone 9
Source: NCC, Geobase Canada

PROJECT No. 11-225-1
PROJECT
Phase II Environmental Site Assessment
460 St. Laurent Blvd., Ottawa, Ontario

DESIGN: MEB
CAD/GIS: MEB
CHECK: ACW
REV: 0
DATE: 20/12/2011
Appendix A

Borehole Records (prepared by Geofirma)
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

**Borehole Number:** MW11-1

**Project Number:** 11-225-1

**Client:** Brigil Construction

**Site Location:** 460 St. Laurent

**Coordinates:** 45 2643.974 N 75 3858.200 W

**Drilling Method:** Triple Tube, Diamond Bit

**MOE Well ID:** A122797

**Date Completed:** November 22, 2011

**Supervisor:** MEB

**Ground Surface Elevation:** 99.666 mARL

**Date of Water Level Measurement:** 23-Nov-11

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Borehole terminated at 7.10 mBGS

Prepared by: MEB
Reviewed by: RTS
Doc: 11-225-1_460 St. Laurent
Template: 2011 Geofirma Template
# BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

**Borehole Number:** MW11-2 (S/D)  
**Project Number:** 11-225-1  
**Client:** Brigil Construction  
**Site Location:** 460 St. Laurent  
**Ground Surface Elevation:** 99.90 mARL  
**Coordinates:** 45 2643.170 N 75 3858.110 W  
**Drilling Method:** Triple Tube, Diamond Bit  
**Date Completed:** November 22, 2011  
**MOE Well ID:** A122797  
**Supervisor:** RTS  
**Date of Water Level Measurement:** 22-Nov-11

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Prepared by: MEB  
Reviewed by: RTS  
Doc: 11-225-1_460 St. Laurent  
Template: 2011 Geofirma Template
## BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

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MOE Well ID: A122797  
Date Completed: November 22, 2011  
Supervisor: RTS  
Ground Surface Elevation: 99.90 mARL  
Date of Water Level Measurement: 22-Nov-11

Prepared by: MEB  
Reviewed by: RTS  
Doc: 11-225-1_460 St. Laurent  
Template: 2011 Geofirma Template
# Borehole MW11-2D (S/D)

**Project Number:** 11-225-1  
**Client:** Brigil Construction  
**Site Location:** 460 St. Laurent  
**Coordinates:** 45 2643.170 N 75 3858.110 W  
**Drilling Method:** Triple Tube, Diamond Bit  
**MOE Well ID:** A122797  
**Date Completed:** November 22, 2011  
**Supervisor:** RTS  
**Ground Surface Elevation:** 99.90 mARL  
**Date of Water Level Measurement:** 22-Nov-11

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Borehole MW11-2D terminated at 15.24 mBGS
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: MW11-3 (S/D)

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GROUND SURFACE

FILL
Brown sand and gravel.

Refusal on Bedrock

BEDROCK
Grey Limestone

25.4 mm diameter PVC riser

Bentonite

25.4 mm diameter PVC screen

Silica Sand

25.4 mm diameter borehole

MOE Well ID: A122797
Date Completed: November 21, 2011
Supervisor: RTS
Ground Surface Elevation: 99.609 mARL
Date of Water Level Measurement: 21-Nov-11

Prepared by: MEB
Reviewed by: RTS
Doc:  
Template: 2011 Geofirma Template
## BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

**Borehole Number:** MW11-3 (S/D)

- **Project Number:** 11-225-1
- **Client:** Brigil Construction
- **Site Location:** 460 St. Laurent
- **Coordinates:** 45 2643.962 N 75 3858.752 W
- **Drilling Method:** Triple Tube, Diamond Bit

**MOE Well ID:** A122797

- **Date Completed:** November 21, 2011
- **Supervisor:** RTS
- **Ground Surface Elevation:** 99.609 mARL
- **Date of Water Level Measurement:** 21-Nov-11

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Prepared by: MEB
Reviewed by: RTS

Page 2 of 3
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: MW11-3 (S/D)  MOE Well ID: A122797
Project Number: 11-225-1  Date Completed: November 21, 2011
Client: Brigil Construction  Supervisor: RTS
Site Location: 460 St. Laurent  Ground Surface Elevation: 99.609 mARL
Coordinates: 45 2643.962 N 75 3858.752 W  Date of Water Level Measurement: 21-Nov-11
Drilling Method: Triple Tube, Diamond Bit

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Borehole MW11-3D terminated at 49.9 mBGS

BOREHOLE TERMINATED

Prepared by: MEB
Reviewed by: RTS
Doc:
Template: 2011 Geofirma Template
**BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG**

**Borehole Number:** MW11-4

- **MOE Well ID:** A122797
- **Date Completed:** November 23, 2011
- **Supervisor:** MEB
- **Ground Surface Elevation:** 99.317 mARL
- **Date of Water Level Measurement:** 23-Nov-11

**Client:** Brigil Construction  
**Site Location:** 460 St. Laurent  
**Coordinates:** 45 2644.196 N 75 3858.352 W

**Drilling Method:** Triple Tube, Diamond Bit

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**GROUND SURFACE**
- Fill: Brown sand and gravel.
- Refusal on Bedrock
- Grey Limestone

**BOREHOLE TERMINATED**

- 31.8 mm diameter PVC screen
- 31.8 mm diameter PVC riser
- 96mm diameter borehole
- Silica Sand

Prepared by: MEB  
Reviewed by: RTS  
Doc: 11-225-1_460 St. Laurent  
Template: 2011 Geofirma Template

Page 1 of 1
**BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG**

**Borehole Number:** MW11-5

**Project Number:** 11-225-1
**Client:** Brigil Construction

**Site Location:** 460 St. Laurent

** Coordinates:** 45 2642.941 N 75 3858.920 W

**Drilling Method:** Triple Tube, Diamond Bit

**MOE Well ID:** A122797

**Date Completed:** November 23, 2011
**Supervisor:** MEB

**Ground Surface Elevation:** 100.857 mARL

**Date of Water Level Measurement:** 23-Nov-11

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*Prepared by: MEB
Reviewed by: RTS
Doc: 11-225-1_460 St. Laurent
Template: 2011 Geofirma Template*
## BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

**Borehole Number:** MW11-5  
**MOE Well ID:** A122797

**Project Number:** 11-225-1  
**Date Completed:** November 23, 2011

**Client:** Brigil Construction  
**Supervisor:** MEB

**Site Location:** 460 St. Laurent  
**Ground Surface Elevation:** 100.857 mARL

**Coordinates:** 45 2642.941 N 75 3858.920 W  
**Date of Water Level Measurement:** 23-Nov-11

**Drilling Method:** Triple Tube, Diamond Bit

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</table>

**STRATIGRAPHIC DESCRIPTION**

- Borehole terminated 6.77 mBGS

**INSTALLATION**

- BOREHOLE TERMINATED

Prepared by: MEB  
Reviewed by: RTS  
Doc: 11-225-1_460 St. Laurent  
Template: 2011 Geofirma Template
## Borehole Stratigraphic and Instrumentation Log

**Borehole Number:** MW11-7

**Project Number:** 11-225-1

**Client:** Brigil Construction

**Site Location:** 460 St. Laurent

**Coordinates:** 45 2642.828 N 75 3858.326 W

**Drilling Method:** Triple Tube, Diamond Bit

**MOE Well ID:** A122797

**Date Completed:** November 22, 2011

**Supervisor:** MEB

**Ground Surface Elevation:** 100.00 mARL

**Date of Water Level Measurement:** 22-Nov-11

### Stratigraphic Description

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<th>Samples</th>
<th>Lab Sample</th>
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<th>PID (ppm)</th>
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</table>

**GROUND SURFACE**
- **Fill:** Brown sand and gravel.
- **Refusal on Bedrock**
- **Bedrock:** Grey Limestone

**Borehole Terminated at 7.17 mBGS**

**Installation**
- 31.8 mm diameter PVC riser
- 31.8 mm diameter PVC screen
- 96mm diameter borehole
- Bentonite
- Silica Sand

Prepared by: MEB
Reviewed by: RTS

Doc: 11-225-1_460 St. Laurent
Template: 2011 Geofirma Template
Appendix B

Core Photographs (MW11-2 and MW11-3)
<table>
<thead>
<tr>
<th>Client:</th>
<th>Geofirma</th>
<th>Project:</th>
<th>Rock Coring - Borehole MW11-2</th>
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<td>DWG #:</td>
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<td>NT</td>
<td>Approved: CH</td>
<td>Geotechnical Investigation</td>
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<td>Scale: N.T.S.</td>
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Appendix C

Shear Wave Velocity Measurements
December 12, 2011

Chris Hendry, M.Eng., P.Eng.
SPL Consultants Limited
146 Colonnade Rd. Unit 17
Ottawa, ON
K2E 7Y1

RE: Shear-wave velocity soundings at 460 St Laurent Blvd., Ottawa, Ontario

Dear Mr. Hendry:

Geophysics GPR International Inc. has been requested by SPL Consultants Ltd. to carry out a shear-wave velocity sounding at 460 St. Laurent Blvd, Ottawa, Ontario (Figure 1).

The survey was performed November 28, 2011.

The investigation included both the multi-channel analysis of surface waves (MASW) and the Spatial Autocorrelation (SPAC) methods to generate shear-wave velocity profiles.

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in table and chart format.
MASW and MAM Surveys

Basic Theory

The Multi-channel Analysis of Surface Waves (MASW) and the Micro-tremor Array Measurements/Spatial Autocorrelation (MAM/SPAC) are seismic methods used to evaluate the shear-wave velocities of subsurface materials through the analysis of the dispersion properties of Rayleigh surface waves (“ground roll”). The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. Inversion of the Rayleigh wave dispersion curve yields a shear-wave (V_s) velocity depth profile (sounding). Figure 2 outlines the basic operating procedure for the MASW method. Figure 3 is an example image of a typical MASW record and resulting 1D V_s model. A more detailed description of the method can be found in the paper Multi-channel Analysis of Surface Waves, Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); P. 800–808.

Figure 1: MASW sounding location, 460 St. Laurent Blvd., Ottawa, ON
**Survey Design**

The geometry of an MASW survey is similar set to that of a seismic refraction investigation (i.e. 24 geophones in a linear array). The fundamental principle involves intentionally generating an acoustic wave at the surface and digitally recording the surface waves from the moment of source impact with a linear series of geophones on the surface. This is referred to as an “active source” method. A sledgehammer was used as the primary energy source with traces being recorded at 4 locations: approximately 1-6 and 10-25m off both ends. Data were collected with geophone spacings of 1 and 3 m. Unlike the refraction method, which produces a data point beneath each geophone, the shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread.

Although the theoretical maximum depth of penetration (34 m) is half of the seismic array length (69 m), in practice the maximum depth of penetration is often influenced by the geology.

**Interpretation Method**

The main processing sequence involved plotting, picking, and 1-D inversion of the MASW/MAM shot records using the SeisimagerSW™ software package. In theory, all MASW shot records should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation and localized surface variations. The results of the inversion process are inherently non-unique and the final model must be judged to be geologically realistic. The inversion modelling also assumes that all layering is flat/horizontal and laterally uniform.

The results of the MASW/MAM tests are presented in chart format as Figure 4. The chart presents the 1-D shear wave velocity values from the inversion models of the seismic records. The approximate location and orientation of the geophone arrays are presented in Figure 1.

The $V_{s30}$ value for the sounding is presented in Table 1. The $V_{s30}$ value is based on the harmonic mean of the shear wave velocities over the upper 30 m. The $V_{s30}$ value is calculated by dividing the total depth of interest (e.g. 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response. The estimated error in the average $V_{s30}$ value determined through MASW tests is typically +/-10 to 15%; however, as the shear-wave velocity of the rock is often poorly constrained by the MASW method alone, this error will be higher for sites with shallow bedrock.
Figure 2: MASW Operating Principle

Figure 3: Example of a typical MASW shot record, phase velocity/frequency curve and resulting 1D shear-wave velocity model.
Figure 4: Shear-wave Velocity Profile for Sounding 1
CONCLUSIONS

The approximate location of the shear-wave sounding is presented in Figure 1.

The background seismic noise levels were moderate to high.

The MASW shear-wave models are presented in Figure 4. The results are summarized in Table 1. The dispersion curves generated from the active seismic data sets were not all well defined. This is typical for sites with high velocity contrasts (e.g. soft clays over bedrock) where the shear-wave velocities of the lowest layer tend to be poorly constrained and underestimated. At this site the overburden is reported by SPL Consultants to be dominated by loose fill material. To provide a more accurate \( V_{s30} \) calculation, the shear-wave model can be constrained using measured P-wave velocities, borehole data and estimates of Poisson’s ratio.

The models presented incorporate measured P-wave velocities for what is interpreted to be the bedrock (approximately 3000 m/s), simple critical distance calculations for a depth to bedrock (approximately 4 to 6m) and an assumed Poisson’s ratio (approximately 0.3).

The \( V_{s30} \) values for the shear-wave models are presented in Table 1. The \( V_{s30} \) values are based on the harmonic mean of the shear wave velocities over the upper 30 m. The \( V_{s30} \) value is calculated by dividing the total depth of interest (e.g. 30m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

<table>
<thead>
<tr>
<th>Minimum</th>
<th>Median</th>
<th>Maximum</th>
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<tbody>
<tr>
<td>516</td>
<td>693</td>
<td>843</td>
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</table>

The calculated \( V_{s30} \) values from the 1D MASW sounding ranged from 516 to 843 m/s. The average \( V_{s30} \) value was 693 m/s.

Based on the average \( V_{s30} \) values (as determined through the MASW method) and table 4.1.8.4.A of the National Building Code of Canada, 2005 Edition, the investigated site area would be classified as category “C” (360 \(<\ V_{s30} \leq 760 \) m/s).

Due to space constraints, the seismic data could not be collected directly within the proposed building footprint. The client has indicated that the depth to bedrock within the building footprint is on the order of 1 to 2m depth and the proposed building is to be founded directly on the bedrock. The \( V_{s30}^* \) value has been calculated over the depth interval of 4 to 34m below grade taking the overburden material out of consideration.
The recalculated Vs30* values are presented in Table 2.

**Table 2: Calculated V\textsubscript{s30*} values (m/s) from the MASW data over the depth interval of 4 and 34 metres depth**

<table>
<thead>
<tr>
<th>Sounding</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
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<tbody>
<tr>
<td>1</td>
<td>869</td>
<td>1110</td>
<td>1307</td>
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</table>

The calculated average V\textsubscript{s30} values over the depth interval of 4 to 34m below grade from the 1D MASW soundings collected was 1110m/s +/- 15 to 20%. The estimated error is higher for the recalculated Vs30 value as there is a decrease in resolution with depth and the shear-wave velocities within rock are less well constrained.

The Vs30 values calculated for the minimum and the maximum envelopes ranged from 869 to 1307 m/s.

Based on the average V\textsubscript{s30*} values (as determined through the MASW method, the measured P-wave velocity and an estimate of Poisson Ratio) and table 4.1.8.4.A of the National Building Code of Canada, 2005 Edition, the investigated site area would be classified as category “B” (760 < V\textsubscript{s30} < 1500 m/s) when considering the bedrock only. Site classification “B” is conditional on there being less than 3m of overburden material regardless of the Vs30 value. This condition was not confirmed by the results of this investigation.

It must be noted that the site classification provided in this report is based solely on the Vs30 value and that it can be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, more than 3m of soft clays, high moisture content, etc. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2005 Edition for more information on the requirements for site classification.

This report has been written by Ben McClement, P.Eng.
Appendix D
Explanation of Terms Used in this Report
EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS ¯N.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60˚ CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

<table>
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<tr>
<th>c_u (kPa)</th>
<th>VERY SOFT</th>
<th>SOFT</th>
<th>FIRM</th>
<th>STIFF</th>
<th>VERY STIFF</th>
<th>HARD</th>
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DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

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<th>N (BLOWS/0.3m)</th>
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<th>LOOSE</th>
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<th>VERY DENSE</th>
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ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN.

THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

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<th>RQD (%)</th>
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<th>EXCELLENT</th>
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<td>75 – 90</td>
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JUXT AND BEDDING:

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<th>1m – 3m</th>
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<td>MOD. CLOSE</td>
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<td>VERY WIDE</td>
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<td>THIN</td>
<td>MEDIUM</td>
<td>THICK</td>
<td>VERY THICK</td>
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ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

| SS | SPLIT SPOON | TP | THINWALL PISTON | m_v | kPa⁻¹ | COEFFICIENT OF VOLUME CHANGE |
| WS | WASH SAMPLE | OS | OSTERBERG SAMPLE | c_v | 1 | COMPRESSION INDEX |
| ST | SLOTTED TUBE SAMPLE | RC | ROCK CORE | c_s | 1 | SWELLING INDEX |
| BS | BLOCK SAMPLE | PH | TW ADVANCED HYdraulically | c_a | 1 | RATE OF SECONDARY CONSOLIDATION |
| CS | CHUNK SAMPLE | PM | TW ADVANCED MANUALLY | c_r | m/s | COEFFICIENT OF CONSOLIDATION |
| TW | THINWALL OPEN | FS | FOIL SAMPLE | H | m | DRAINAGE PATH |
|   |               |   |             | T_v | 1 | TIME FACTOR |

STRESS AND STRAIN

<table>
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<th>kPa</th>
<th>PORE WATER PRESSURE</th>
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<td>kPa</td>
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<td>k</td>
<td>kPa</td>
<td>SHEAR STRESS</td>
</tr>
<tr>
<td>c</td>
<td>kPa</td>
<td>PRINCIPAL STRESSES</td>
</tr>
<tr>
<td>v</td>
<td>%</td>
<td>LINEAR STRAIN</td>
</tr>
<tr>
<td>c_l, c_s, c_u</td>
<td>%</td>
<td>PRINCIPAL STRAINS</td>
</tr>
<tr>
<td>G</td>
<td>kPa</td>
<td>MODULUS OF SHEAR DEFORMATION</td>
</tr>
<tr>
<td>μ</td>
<td>1</td>
<td>COEFFICIENT OF FRICTION</td>
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MECHANICALL PROPERTIES OF SOIL

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<th>DENSITY OF SOLID PARTICLES</th>
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<tbody>
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<td>kPa</td>
<td>UNIT WEIGHT OF SOLID PARTICLES</td>
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<tr>
<td>D</td>
<td>mm</td>
<td>GRAIN DIAMETER</td>
</tr>
<tr>
<td>C</td>
<td>1</td>
<td>UNIFORMITY COEFFICIENT</td>
</tr>
<tr>
<td>q</td>
<td>mm/m</td>
<td>RATE OF DISCHARGE</td>
</tr>
<tr>
<td>i</td>
<td>%</td>
<td>SENSITIVITY = c_u / c_s</td>
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PHYSICAL PROPERTIES OF SOIL

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<tr>
<td>P_n</td>
<td>kg/m³</td>
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<td>m/s</td>
<td>SEEPAGE FORCE</td>
</tr>
<tr>
<td>μ</td>
<td>1</td>
<td>UNITS OF SUBMERGED SOIL</td>
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</table>

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

| SS | SPLIT SPOON | TP | THINWALL PISTON | m_v | kPa⁻¹ | COEFFICIENT OF VOLUME CHANGE |
| WS | WASH SAMPLE | OS | OSTERBERG SAMPLE | c_v | 1 | COMPRESSION INDEX |
| ST | SLOTTED TUBE SAMPLE | RC | ROCK CORE | c_s | 1 | SWELLING INDEX |
| BS | BLOCK SAMPLE | PH | TW ADVANCED HYdraulically | c_a | 1 | RATE OF SECONDARY CONSOLIDATION |
| CS | CHUNK SAMPLE | PM | TW ADVANCED MANUALLY | c_r | m/s | COEFFICIENT OF CONSOLIDATION |
| TW | THINWALL OPEN | FS | FOIL SAMPLE | H | m | DRAINAGE PATH |
|   |               |   |             | T_v | 1 | TIME FACTOR |

STRESS AND STRAIN

<table>
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<tr>
<th>w_d</th>
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<tr>
<td>c_u</td>
<td>kPa</td>
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<tr>
<td>c_s</td>
<td>kPa</td>
<td>EFFECTIVE NORMAL STRESS</td>
</tr>
<tr>
<td>k</td>
<td>kPa</td>
<td>SHEAR STRESS</td>
</tr>
<tr>
<td>c</td>
<td>kPa</td>
<td>PRINCIPAL STRESSES</td>
</tr>
<tr>
<td>v</td>
<td>%</td>
<td>LINEAR STRAIN</td>
</tr>
<tr>
<td>c_l, c_s, c_u</td>
<td>%</td>
<td>PRINCIPAL STRAINS</td>
</tr>
<tr>
<td>G</td>
<td>kPa</td>
<td>MODULUS OF SHEAR DEFORMATION</td>
</tr>
<tr>
<td>μ</td>
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<td>COEFFICIENT OF FRICTION</td>
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MECHANICALL PROPERTIES OF SOIL

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<th>P_s</th>
<th>kg/m³</th>
<th>DENSITY OF SOLID PARTICLES</th>
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<tr>
<td>c_u</td>
<td>kPa</td>
<td>UNIT WEIGHT OF SOLID PARTICLES</td>
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<tr>
<td>D</td>
<td>mm</td>
<td>GRAIN DIAMETER</td>
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<tr>
<td>C</td>
<td>1</td>
<td>UNIFORMITY COEFFICIENT</td>
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<tr>
<td>q</td>
<td>mm/m</td>
<td>RATE OF DISCHARGE</td>
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<tr>
<td>i</td>
<td>%</td>
<td>SENSITIVITY = c_u / c_s</td>
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PHYSICAL PROPERTIES OF SOIL

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<tr>
<td>P</td>
<td>kg/m³</td>
<td>DENSITY OF SATURATED SOIL</td>
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<tr>
<td>P_n</td>
<td>kg/m³</td>
<td>DENSITY OF SUBMERGED SOIL</td>
</tr>
<tr>
<td>v</td>
<td>m/s</td>
<td>SEEPAGE FORCE</td>
</tr>
<tr>
<td>μ</td>
<td>1</td>
<td>UNITS OF SUBMERGED SOIL</td>
</tr>
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Appendix E

Limitations of this Report
LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to SPL Consultants Limited at the time of preparation. Unless otherwise agreed in writing by SPL Consultants Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SPL Consultants Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.