Geotechnical and Hydrogeological Investigation

*Dillon Consulting Limited*

**Project Name:**
Forest View Development
101 Meadowlily Road South
London, Ontario

**Project Number:**
LON-00017363-GE

**Prepared By:**
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Geotechnical Investigation

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1. Introduction and Background

1.1 Introduction

EXP Services Inc. (EXP) was retained by Dillon Consulting Limited to carry out a geotechnical investigation and prepare a geotechnical report relating to the proposed Forest View development to be located 101 Meadowlily Road South in London, Ontario, hereinafter referred to as the ‘Site’.

It is understood that the development will consist of a condo development including single family residential lots with basements, 4-plex townhomes, storm sewers and a sanitary pump station. Additionally, the development includes a watermain and sanitary forcemain extension from the Site Commissioners Road East.

Based on an interpretation of the factual test hole data and a review of soil and groundwater information from test holes advanced at the site, EXP has provided geotechnical engineering guidelines to support the proposed development.

1.2 Terms of Reference

The geotechnical investigation was generally completed in accordance with the scope of work outlined via email communications. Authorization to proceed with this investigation was received from Mr. Jason Johnson of Dillon Consulting Limited through email correspondence.

The purpose of the investigation was to examine the existing soil and groundwater conditions at the site by advancing a series of boreholes at the locations chosen by EXP and illustrated on the attached Borehole Location Plan (Drawing 1).

Based on an interpretation of the factual borehole data, and a review of soil and groundwater information from test holes excavated at the site, EXP has provided engineering guidelines for the geotechnical design and construction of the proposed development. More specifically, this report provides comments on excavations, dewatering, site preparation, foundations, slab-on-grade construction, bedding and backfill, earthquake design considerations, pavement recommendations, and curbs and sidewalks.

This report is provided on the basis of the terms of reference presented above, and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design.

The information in this report in no way reflects on the environmental aspects of the soil. Should specific information in this regard be needed, additional testing may be required.

Reference is made to Appendix D of this report, which contains further information necessary for the proper interpretation and use of this report.
2. Methodology

The fieldwork was conducted between August 20 and 21, 2019 and consisted of advancing seven (7) boreholes at the approximate locations shown on Drawing 1. The boreholes are labeled as BH1 through BH7.

Prior to the investigation, buried service clearances were obtained for the boreholes by EXP.

The boreholes and dynamic cone tests were completed by a specialist drilling subcontractor under the full-time supervision of EXP geotechnical staff. The boreholes were advanced utilizing a track-mounted drill rig equipped with continuous flight solid and hollow stem augers, soil sampling and soil testing equipment. In each borehole, disturbed soil samples were recovered at depth intervals of 0.75 m and 1.5 m using conventional split spoon sampling equipment. Standard Penetration Tests (SPT’s) were also performed throughout the boreholes to assess the compactness or consistency of the underlying soils, and to obtain representative samples. The boreholes are described on the Borehole Logs (see Appendix A).

During the drilling, the stratigraphy within the boreholes was examined and logged in the field by EXP geotechnical personnel.

Short-term groundwater levels within the open boreholes were observed. These observations pertaining to groundwater conditions at the test hole locations are recorded in the borehole logs found in Appendix A. Following the drilling, the boreholes were backfilled with the excavated materials and bentonite, to satisfy the requirements of O.Reg. 903. A 50 mm diameter PVC monitoring well was installed in Boreholes BH3, BH5 and BH7. Details of the monitoring well construction are provided on the attached Borehole Log. Groundwater measurements in the monitoring well were obtained on August 28 and September 20, 2019.

Representative samples of the various soil strata encountered at the test locations were taken to our laboratory in London for further examination by a geotechnical engineer and laboratory classification testing. Laboratory testing for this investigation comprised routine moisture content determinations (results presented on the borehole logs in Appendix A) and gradation analysis.

Samples remaining after the classification testing will be stored for a period of three months following the issuance of report. After this time, they will be discarded unless prior arrangements have been made for longer storage.

The location of each test hole was established in the field in conjunction with reference to the proposed development configuration. Ground surface elevation at each borehole location was interpolated based on topographic mapping provided by AGM.
3. Site and Subsurface Conditions

3.1 Site Description

The site is bounded by a wood-lot with Meadowlily Road South representing the east limit, Sunningdale Road to the south, agricultural land to the north and Powell Lands Subdivision to the west. An aerial photograph (taken from Google Maps, September 2019) is provided below for reference.

At the time of the fieldwork, the bulk of the site was generally being used for agricultural purposes. The site also has a residence and ancillary structures. Non-building areas of the site are generally characterized by open fields.
3.2 Soil Stratigraphy – Meadowlily Road Services

Boreholes BH1 and BH2 were located within the proposed sewer and watermain alignment on Meadowlily Road South. The detailed stratigraphy encountered in each borehole is shown on the borehole logs found in Appendix A and summarized in the following paragraphs. It must be noted that the boundaries of the soil indicated on the borehole logs are inferred from non-continuous sampling and observations during excavation. These boundaries are intended to reflect transition zones for geotechnical design and should not be interpreted as exact planes of geological change.

Asphalt and Granular Fill

Asphalt was penetrated at the ground surface at the location of all boreholes, measuring between 75 mm and 100 mm in thickness.

Beneath the asphalt a layer of sand and gravel fill was encountered, measuring between 500 mm and 800 mm in thickness. The granular fill was noted to be in a compact and moist state.

It should be noted that asphalt and fill quantities should not be established from the information provided at the borehole locations only.

Clayey Silt Till

Beneath the asphalt and granular fill in each borehole was a stratum of glacial till. The till predominantly comprised of clayey silt and was typically brown in colour. The till contained trace sand and gravel and has a stiff to very stiff consistency (SPT N Values of 9 to 21 blows per 300 mm split spoon penetration). The in situ moisture content of the till ranges from 6 to 16 percent indicating damp to wet conditions.

Sandy Silt

Beneath the till in Borehole BH2, sandy silt was encountered at a depth of 2.4 m below ground. The sandy silt was generally described as brown in colour and dilatant. The sandy silt was compact with SPT N-values ranging from 12 to 26 blows per 300 mm sample spoon penetration. The sandy silt was in a wet state with in situ moisture contents ranging from 17 to 22 percent.
3.3 Soil Stratigraphy – Residential Subdivision

Boreholes BH3 to BH7 were located within the proposed residential subdivision at 101 Meadowlily Road South. The detailed stratigraphy encountered in each borehole is shown on the borehole logs found in Appendix A and summarized in the following paragraphs. It must be noted that the boundaries of the soil indicated on the borehole logs are inferred from non-continuous sampling and observations during excavation. These boundaries are intended to reflect transition zones for geotechnical design and should not be interpreted as exact planes of geological change.

Topsoil

The test holes were each surfaced with a layer of topsoil. The topsoil, generally described as brown/black sandy loam, loose, and moist. The topsoil extended to depths ranging between about 75 mm and 400 mm.

In cultivated areas, it should be anticipated that surficial topsoil has been blended into the underlying subgrade soils. In wooded areas, the topsoil may be thicker, and contain areas with significant roots.

Sandy Silt

Beneath the topsoil in each borehole, with the exception of Borehole BH4 and BH7, was a stratum of sandy silt. The sandy silt was typically brown to grey in colour, weathered in the upper limits and contained dilatant layering throughout. The compactness of the sandy silt is loose to very dense based on SPT N Values of 7 to 58 blows per 300 mm split spoon penetration. The in situ moisture content of the sandy silt ranges from 15 to 30 percent indicating moist to wet conditions.

Clayey Silt

Within Borehole BH3, clayey silt was encountered below the sandy silt at a depth of about 3.7 m below ground surface. The clayey silt was generally described as grey in colour and firm to stiff with SPT N-values ranging from 8 to 9 blow per 300 mm sample spoon penetration. The in situ moisture content of the clayey silt ranges from 18 to 24 percent, indicative of moist conditions.

Clayey Silt Till

Beneath the topsoil at Borehole BH4 was a stratum of glacial till. The till predominantly comprised of clayey silt and was brown in colour. The till contained trace sand and gravel, and has a very stiff consistency (SPT N Values of 17 to 20 blows per 300 mm split spoon penetration). The in situ moisture content of the till ranges from 14 to 16 percent indicating damp to moist conditions.

Silty Sand

Beneath the clayey silt at Borehole BH3 and the topsoil at Borehole BH7, a deposit of silty sand was encountered. The silty sand was noted to be brown, weathered in the upper limits and contained clayey silt layering. The compactness condition of the silty sand is loose to very dense, based on Standard Penetration Test (SPT) N-values from 9 to greater than 50 blows per 300 mm penetration of the split-spoon sampler. The in situ moisture content of the silty sand ranges from 3 to 27 percent, indicative of moist to wet conditions.
3.4 Groundwater Conditions

Details of the groundwater conditions observed within the boreholes are provided on the attached Borehole Logs.

As part of the Geotechnical Investigation three (3) monitoring well were installed. The wells were installed to depths ranging between about 6.3 and 10.2 m bgs. The summary of well construction details and stabilized groundwater levels are presented in the tables below.

**Table 1 – Monitoring Well Construction Details**

<table>
<thead>
<tr>
<th>Borehole ID</th>
<th>Ground Surface Elevation (m)</th>
<th>Completion Depth (m bgs)</th>
<th>Screen Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH3/MW</td>
<td>270.3</td>
<td>10.15</td>
<td>3.0</td>
</tr>
<tr>
<td>BH5/MW</td>
<td>257.6</td>
<td>6.25</td>
<td>1.5</td>
</tr>
<tr>
<td>BH7/MW</td>
<td>254.7</td>
<td>6.50</td>
<td>1.5</td>
</tr>
</tbody>
</table>

**Table 2 – Stabilized Groundwater Level**

<table>
<thead>
<tr>
<th>Borehole ID</th>
<th>Ground Surface Elevation (m)</th>
<th>Depth to Groundwater, m bgs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>August 28, 2019</td>
</tr>
<tr>
<td>BH3/MW</td>
<td>270.3</td>
<td>Dry</td>
</tr>
<tr>
<td>BH5/MW</td>
<td>257.6</td>
<td>2.85 (254.75)</td>
</tr>
<tr>
<td>BH7/MW</td>
<td>254.7</td>
<td>3.34 (251.36)</td>
</tr>
</tbody>
</table>

Upon completion of drilling, the open borehole excavations were examined for the presence of groundwater and groundwater seepage. Groundwater was measured in Borehole BH2 at 4.0 m bgs upon completion of drilling. All other boreholes were dry following drilling. The observed groundwater is not representative of stabilized conditions. It is believed to be perched in the sandy silt layer above the less permeable glacial till.

It is also noted that the depth to the groundwater table may vary in response to climatic or seasonal conditions, and, as such, may differ at the time of construction, with higher levels in wet seasons. Capillary rise effects should also be anticipated in fine-grained soil deposits.
To identify the depth of the potable groundwater aquifer for the area, a review of the local Ministry of Environment, Conservation and Parks (MECP) water well records (WWR) was carried out within close proximity (500 m or less) to the investigation area. The findings are summarized in the following table:

<table>
<thead>
<tr>
<th>MECP Well No.</th>
<th>Depth of Well (m bgs)</th>
<th>Water Use</th>
<th>Well Status</th>
<th>Bottom Lithology</th>
</tr>
</thead>
<tbody>
<tr>
<td>2100163</td>
<td>27.7</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Sand</td>
</tr>
<tr>
<td>2100169</td>
<td>32.9</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2100170</td>
<td>34.7</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2100171</td>
<td>32.9</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Gravel</td>
</tr>
<tr>
<td>2100176</td>
<td>37.8</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2100177</td>
<td>35.1</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2100178</td>
<td>33.5</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Clay</td>
</tr>
<tr>
<td>2100288</td>
<td>35.1</td>
<td>Public</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103308</td>
<td>33.2</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Shale</td>
</tr>
<tr>
<td>2103330</td>
<td>34.1</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103367</td>
<td>31.7</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Clay Till</td>
</tr>
<tr>
<td>2103391</td>
<td>35.7</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Shale</td>
</tr>
<tr>
<td>2103428</td>
<td>34.1</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103434</td>
<td>26.5</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Clay Till</td>
</tr>
<tr>
<td>2103544</td>
<td>34.1</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Shale</td>
</tr>
<tr>
<td>2103574</td>
<td>30.2</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Clay Till</td>
</tr>
<tr>
<td>2103632</td>
<td>31.7</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Sand</td>
</tr>
<tr>
<td>2103649</td>
<td>30.5</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103747</td>
<td>30.8</td>
<td>Livestock</td>
<td>Water Supply</td>
<td>Sand</td>
</tr>
<tr>
<td>2103846</td>
<td>33.5</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103879</td>
<td>31.1</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103880</td>
<td>31.7</td>
<td>Irrigation</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103887</td>
<td>37.2</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Limestone</td>
</tr>
<tr>
<td>2103939</td>
<td>30.5</td>
<td>Domestic</td>
<td>Water Supply</td>
<td>Sand</td>
</tr>
</tbody>
</table>
The wells are generally set into bedrock at depths ranging between 27 m and 41 m below existing grades. MECP Well records indicate eleven wells are set into the deep aquifer of clay or sand. Overburden soils noted in the MECP WWR were generally described as clay and sand layering.

Groundwater flow across the Site is affected by the soil permeability, topography and drainage. The wells in the area indicate that potable water is generally found in bedrock aquifers.

### 3.6 Methane Gas

An RKI Gx-2003 Gas Detector was used in the upper levels of the open boreholes. The unit measures LEL combustibles, methane gas, oxygen content, carbon monoxide and hydrogen sulfide in standard confined space gases. No methane gas was detected in the boreholes.
4. Discussion and Recommendations

4.1 General

It is understood that the development will consist of a condo development including single family residential lots with basements, 4-plex townhomes, storm sewers and a sanitary pump station. Additionally, the development includes a watermain and sanitary forcemain extension from the Site Commissioners Road East.

Other associated features of the development may include access roads, new site services, and landscaped areas.

The following sections of this report provide geotechnical comments and recommendations regarding site preparation, excavations and dewatering, foundations, slab-on-grade design, bedding and backfill, earthquake design considerations, pavement design and curbs and sidewalks.

4.2 Site Preparation

General

Prior to placement of foundations, engineered fill, pipe bedding and/or road subgrade, all surficial topsoil, vegetation and/or otherwise deleterious materials should be stripped. Thicker areas of topsoil may be anticipated in areas with trees and/or heavy vegetative cover. It is anticipated that the surficial topsoil may be stockpiled on site for possible reuse as landscaping fill.

Following the removal of topsoil, fill and other unsuitable soils, the exposed subgrade surfaces at subgrade design level should be thoroughly proof-rolled with a heavy roller and examined by a Geotechnical Engineer. Any soft or loose areas detected during the proof-rolling process should be sub-excavated and replaced with approved material.

It is recommended that construction traffic be minimized on the finished subgrade, and that the subgrade be sloped to promote surface drainage and runoff.

In the building areas where the grade will be raised, the fill material should comprise imported granular or approved onsite (excavated) material. The fill material should be inspected and approved by a Geotechnical Engineer, should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD) within 3 percent of optimum moisture content. The geometric requirements for engineered fill are provided on Drawing 2.

The natural and inorganic sandy silt and clayey silt materials on site would be suitable for reuse as engineered fill. The material should be examined and approved by a Geotechnical Engineer prior to reuse.

In areas along the proposed roadways, fill material used to raise grades may comprise onsite excavated soils or imported granular fill approved by the engineer. The fill should be placed in maximum 300 mm (12 inch) thick loose lifts and uniformly compacted to 95/98 percent SPMDD within 3 percent of optimum moisture content in order to provide adequate stability for the new pavements.

In situ compaction testing should be carried out during the fill placement to ensure that the specified compaction is being achieved.

Excess materials should be removed from the site and disposed of in accordance with Ministry of the Environment, Conservation and Parks (MECP) guidelines and requirements. Analytical sampling and testing may be required in accordance with O. Reg. 153 for transportation and off-site disposal of excavated material.
If imported fill material is used at the site, verification of the suitability of the fill may be required from an environmental standpoint. Conventional geotechnical testing will not determine the suitability of the material in this regard. Analytical testing and environmental site assessment may be required at the source. This will best be assessed prior to the selection of the material source. A quality assurance program should be implemented to ensure that the fill material will comply with the current (MECP) standards for placement and transportation.

The disposal of excavated materials must also conform to the MECP Guidelines and requirements. EXP can be of assistance if an assessment of the materials is required.

4.3 Excavation and Groundwater Control

General

All work associated with design and construction relative to excavations must be carried out in accordance with Part III of Ontario Regulation 213/91 under the Occupational Health and Safety Act (OHSA). OHSA specifies that where workmen must enter a trench or excavation carried deeper than 1.2 m, the trench or excavation must be suitably sloped and/or braced in accordance with the OHSA regulations.

The natural very stiff clayey silt till encountered at the site is generally classified as Type 2 soils while the sandy silt and clayey silt and sandy silt materials are classified as Type 3 soils. Temporary excavation sidewalls which extend through and terminate within Type 2 soil may be cut vertical in the bottom 1.2 m (4 ft), and cut back at an inclination of 1 horizontal to 1 vertical or flatter above that level. Where excavations extend into or through Type 3 soil, excavation side slopes must be cut back at a maximum inclination of about 1H:1V from the base of the excavation. When excavations extend through Type 2 and Type 3 soils, the excavation should be cut as a Type 3 soil. In the event groundwater egress loosens the sidewalls, flatter slopes of 3H:1V will be required and the soils should be considered Type 4.

Geotechnical inspection at the time of excavation can confirm the soil type present.

It should be noted that the presence of cobbles and boulders in natural glacial deposits may influence the progress of excavation and construction.

Excavation Support

The recommendations for side slopes given in the above section would apply to most of the conventional excavations expected for the proposed development. However, in areas adjacent to buried services that are located above the base of the excavations, side slopes may require support to prevent possible disturbance or distress to these structures. This concept also applies to connections to existing services. In granular soils above the groundwater and in cohesive natural soils, bracing will not normally be required if the structures are behind a 45-degree line drawn up from the toe of the excavation. In wet sandy or silt soils, the setback should be about 3H to 1V if bracing is to be avoided.

For support of excavations such as for any deep manholes, shoring such as sheeting or soldier piles and lagging can be considered. The design and use of the support system should conform to the requirements set out in the most recent version of the Occupational Health and Safety Act for Construction Projects and approved by the Ministry of Labour. Excavations should conform to the guidelines set out in the proceeding section and the Safety Act.

The shoring should also be designed in accordance with the guidelines set out in the Canadian Foundation Engineering Manual, 4th Edition. Soil-related parameters considered appropriate for a soldier pile and lagging system are shown below.
Where applicable, the lateral earth pressure acting on the excavation shoring walls may be calculated from the following equation:

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

where, 
- \( P \) = lateral earth pressure in kPa acting at depth \( h \);
- \( \gamma \) = natural unit weight, a value of 20.4 kN/m\(^3\) may be assumed;
- \( h \) = depth of point of interest in m;
- \( q \) = equivalent value of any surcharge on the ground surface in kPa.

The earth pressure coefficient \( (K) \) may be taken as 0.25 where small movements are acceptable and adjacent footing or movement sensitive services are not above a line extending at 45 degrees from the bottom edge of the excavation; 0.35 where utilities, roads, sidewalks must be protected from significant movement; and 0.45 where adjacent building footings or movement sensitive services (gas and water mains) are above a line of 60 degrees from the horizontal extending from the bottom edge of the excavation.

For long term design, a \( K \) at rest \( (Ko) \) of a minimum of 0.5 should be considered.

The above expression assumes that no hydrostatic pressure will be applied against the shoring system. It should be recognized that the final shoring design will be prepared by the shoring contractor. It is not possible to comment further on specific design details until this design is completed.

If the shoring is exposed to freezing temperatures, appropriate insulation may be provided to prevent outward movement.

The performance of the shoring must be checked through monitoring for lateral movement of the walls of the excavation to ensure that the shoring movements remain within design limits. The most effective method for monitoring the shoring movements can best be devised by this office when the shoring plans become available. The shoring designer should however assess the specific site requirements and submit the shoring plans to the engineer for review and comment.

**Construction Dewatering**

Groundwater was measured in Boreholes BH2, BH5 and BH7 between 2.7 and 6.1 m bgs upon completion of drilling. All other boreholes were dry following drilling. The observed groundwater is likely travelling through the sandy silt layer perched above a less permeable till deposit. Based on the soil texture encountered during the investigation, significant groundwater infiltration should not be anticipated within the building and service trench excavations. Where minor groundwater infiltration is encountered it can most likely be accommodated using conventional sump pumping techniques; provided that the sump pits are lined with a suitable geotextile filter fabric and pump inlet is set in a clear stone, which must fill the sump pit completely. Use of a filtered system will result in migration of sandy soil particles that will loosen the soil deposits.
However, if groundwater infiltration persists, more extensive dewatering measures may be required. EXP would be pleased to provide further information in this regard, upon request.

The collected water should be discharged a sufficient distance away from the excavated area to prevent the discharge water from returning to the excavation. Sediment control measures should be provided at the discharge point of the dewatering system. Caution should also be taken to avoid any adverse impacts to the environment.

Although not anticipated for foundation excavations to conventional depths, it is important to mention that for any projects requiring positive groundwater control with a removal rate of 50,000 litres to less than 400,000 litres per day, an Environmental Activity and Sector Registry (EASR) or Permit to Take Water (PTTW) will be required. PTTW applications are required for removal rates more than 400,000 L per day and will need to be approved by the MECP per Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and the Water Taking and Transfer Regulation O. Reg. 387/04. It is noted that a standard geotechnical investigation will not determine all the groundwater parameters which may be required to support the application. Accordingly, a detailed hydrogeological assessment from a quantitative point of view may be required to estimate the quantity of water to be removed. EXP can assist if the need arises.

4.4 Foundation Construction

The proposed residential units can be supported on conventional spread and strip footings founded below the topsoil or unsuitable soils on the natural competent subgrade soils or engineered fill.

The following allowable bearing pressures (net stress increase) can be used on the natural, undisturbed sandy silt, silty sand or clayey silt till soils below a typical depth of approximately 1.2 m below existing grade throughout the site:

| Bearing Resistance at Serviceability Limit States (SLS) | 145 kPa (3,000 psf) |
| Factored Bearing Resistance at Ultimate Limit States (ULS) | 215 kPa (4,500 psf) |

Loose soil was encountered in Boreholes BH6 and BH7 near 1.2 m depth. If conventional footings are considered in the area of these boreholes, a founding depth of 3.0 m bgs should be considered.

Where soil is removed and grades are to be raised or restored, engineered fill can be used for foundation support. The geometric requirements for the fill placement are shown on Drawing 2, appended. The available SLS bearing capacity for the engineered fill is 145 kPa (3,000 psf). For footings placed on engineered fill, it is recommended that the strip footings be widened to 500 mm (20 inches) and contain nominal concrete reinforcing steel.

Verification of the soil conditions and the extent of reinforcement are best determined by the Geotechnical Engineer at the time of excavation.
4.4.1 Foundations - General

Footings at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing. This concept should also be applied to service excavation, etc. to ensure that undermining is not a problem.

Provided that the footing bases are not disturbed due to construction activity, precipitation, freezing and thawing action, etc., and the aforementioned bearing pressures are not exceeded, the total and differential settlements of footings designed in accordance with the recommendations of this report and with careful attention to construction detail are expected to be less than 25 mm and 20 mm (1 and ¾ inch) respectively.

All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m (4 ft) of soil cover or equivalent insulation.

It should be noted that the recommended bearing capacities have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, if more specific information becomes available with respect to conditions between boreholes when foundation construction is underway. The interpretation between the boreholes and the recommendations of this report must therefore be checked through field inspections provided by EXP to validate the information for use during the construction stage.

4.5 Slab-On-Grade Construction

Preparation of the subgrade should include the removal of all topsoil and/or deleterious material from the proposed building area. The entire floor slab area should then be thoroughly proof-rolled with a heavy roller and examined by a Geotechnical Engineer. Any excessively soft or loose areas should be sub-excavated and replaced with suitable compacted fill. Where the exposed subgrade requires reconstruction to achieve the design elevations, structural fill should be used. It is recommended that structural fill comprises granular material, such as OPSS Granular ‘B’, or approved alternative material. The fill should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). For best compaction results, the in-situ moisture content of the fill should be within about three percent of optimum, as determined by Standard Proctor density testing.
No special underfloor drains are required provided that the exterior grades are lower than the floor slab, and positively sloped away from the slab. It is recommended that an impermeable soil seal such as clay, asphalt or concrete be provided on the surface to minimize water infiltration from the exterior of the building. See **Drawing 3** for Drainage and Backfill recommendations for slab-on-grade construction.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear stone, should be then placed between the prepared granular sub-base and the floor slab.

The installation and requirement of a vapor barrier under a concrete slab should conform to the flooring manufacturer’s and designer’s requirements. Moisture emission testing will be required to determine the concrete condition prior to flooring installation. In order to minimize the potential for excess moisture in the floor slab at the time of the flooring installation, a concrete mixture with a low water-to-cement ratio (i.e., 0.45 to 0.55) should be used. Chemical additives may be required at the time of placement to make the concrete workable, and should be used in place of additional water at the point of placement. Ongoing liaison from this office will be required.

For slab on grade design, the modulus of subgrade reaction \( k \) can be taken as 25 MPa/m for the compacted stone layer over the compacted granular subbase.

The water-to-cement ratio and slump of concrete utilized in the floor slabs should be strictly controlled to minimize shrinkage of the slabs. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.

### 4.6 Basement Construction

The main floor slab for the proposed basements may be constructed using conventional concrete slab-on-grade techniques. The floor subgrade area should be stripped of any fill. The exposed area should be thoroughly proof rolled with a heavy roller and any soft spots detected by this or any other means should be dug out and made good with compactable fill, following the guidelines set out in Section 4.2.

Care should be taken to protect the subgrade below the floor slabs during construction, by limiting construction traffic on the prepared subgrade soils. In addition, if the exposed subgrade soils are exposed to inclement weather conditions (i.e. rain, snow, freezing conditions), some remedial works may be required to remove wet, soft, or disturbed soils prior to stone and concrete placement.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear, crushed stone, should be placed between the prepared subgrade and the floor slab. For design, the modulus of subgrade reaction \( k \) can be taken as 25 MPa/m for the compacted stone layer over the natural subgrade soils.

The water-to-cement ratio and slump of concrete utilized in the floor slab should be strictly controlled to minimize shrinkage of the slab. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.
All basement walls should be damp-proofed and must be designed to resist a horizontal earth pressure ‘P’ at any depth ‘h’ below the surface as given by the following expression:

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

where,
- \( P \) = lateral earth pressure in kPa acting at depth \( h \);
- \( \gamma \) = natural unit weight, a value of 20.4 kN/m³ may be assumed;
- \( h \) = depth of point of interest in m;
- \( q \) = equivalent value of any surcharge on the ground surface in kPa.
- \( K \) = earth pressure coefficient, assumed to be 0.4

Installation of perimeter drains is required for the basements at the Site. The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall. Suggestions for permanent perimeter drainage are given on Drawing 4. Consideration should be given to the installation of an under floor drainage system to collect and remove any water buildup beneath the structure.

4.7 Foundation Backfill

In general, the existing natural inorganic sandy silt and clayey silt till soils excavated from the foundation area should be suitable for re-use as foundation wall backfill if the work is carried out during relatively dry weather. The materials to be re-used should be within three percent of optimum moisture for best compaction results. Materials should be stockpiled per their composition; i.e. sandy soils should not be mixed with clayey soils.

If the weather conditions are very wet during construction, then imported granular material such as OPSS Granular ‘B’ should be used. Site review by the geotechnical consultant may be advised.

The backfill must be brought up evenly on both sides of walls not designed to resist lateral earth pressures.

During construction, the fill surface around the perimeter of structures should be sloped in such a way that the surface runoff water does not accumulate around the structure.
4.8 Site Servicing

The subgrade soils beneath the water and sewer pipes which will service the Site are generally expected to comprise sandy silt or clayey silt till. Where soils are removed, and grade is to be restored, granular should be used. For services constructed on the natural soils or engineered fill, the bedding should conform to City of London and OPS Standards. The bedding course may be thickened if portions of the subgrade become wet during excavation. Bedding aggregate should be placed around the pipe to at least 300 mm (12 inch) above the pipe and be compacted to a minimum 95 percent SPMDD.

The bases of excavations which cut into and terminate in competent till are expected to remain stable for the short construction period. Clear stone or crushed stone bedding may be used in the service trenches as bedding below spring line of the pipe if necessary to assist groundwater control and provide stabilization to the excavation base in wet silty soils. Geotextile should be wrapped around the stone bedding to minimize migration of fines.

The potential locations for use of stone bedding should be identified during construction and are expected to vary across the site due to seasonal conditions and variations in the event of perched groundwater.

Water and sewer lines installed outside of heated areas should be provided with a minimum 1.2 m (4 ft.) of soil cover for frost protection.

To minimize disturbance to the base, pipe laying should be carried out in short sections, with backfilling following closely after laying and no section of trench should be left open overnight.

The trenches above the specified pipe bedding should be backfilled with inorganic on-site soils, placed in 300 mm thick lifts and uniformly compacted to at least 95% SPMDD. For trench backfill within 1 meter below the roadway subbase, the fill should be uniformly compacted to at least 98% SPMDD. A program of in situ density testing should be set up to ensure that satisfactory levels of compaction are achieved.

Requirements for backfill in service trenches, etc. should also have regard for City of London requirements. A summary of the general recommendations for trench backfill is presented on Drawing 5 and 6. A program of in situ density testing should be set up to ensure that satisfactory levels of compaction are achieved.

Based on the results of this investigation, the majority of the excavated clayey silt till material may be used for construction backfill provided that reasonable care is exercised in handling. In this regard, the material should be within 3 percent of the optimum moisture as determined in the Standard Proctor density test and stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet or otherwise adverse weather.

Soils excavated from below the stabilized groundwater table may be too wet for reuse as backfill unless adequate time is allowed for drying, or if the material is blended with approved dry fill; otherwise, it may be stockpiled onsite for reuse as landscape fill.

As noted previously, disposal of excavated materials off site should conform to current MECP guidelines.
4.9 Pumping Station

The proposed sanitary pump station will be sited at the north end of the site. Once the surface topsoil is removed and subgrade is exposed below the frost disturbed zone, the underlying soils consisted of compact, silt, silty sand and sand and gravel underlain by generally stiff to very stiff silty clay.

The pump station building can be constructed on conventional strip and spread footings with a slab-on-grade floor, in accordance with the general comments provided in the following paragraphs. The anticipated minor groundwater ingress may require temporary control to facilitate construction.

For foundations set on the natural soils at depths of approximately 1.2 m below existing grades or approved structural (granular) fill, the following allowable bearing pressures (net stress increase) can be used:

- **Bearing Resistance at Serviceability Limit States (SLS)**: 145 kPa (3,000 psf)
- **Factored Bearing Resistance at Ultimate Limit States (ULS)**: 215 kPa (4,500 psf)

Verification of the footing base soil conditions should be undertaken by the geotechnical engineer at the time of excavation. It may be necessary to compact the sand soils prior to footing construction.

**Flow Split Chamber / Valve Chambers**

Details of the pumping station were not available at the time of preparing this report. The flow split chamber and other valve chambers can be supported on the natural sandy silt up to about 4.0 m depth using a design SLS bearing capacity of 145 kPa. The geotechnical resistance at Ultimate Limit States (ULS) is 215 kPa. If there are deeper structures involved other than the wet well, EXP should be contacted to review the proposed soil bearing. Based on the groundwater conditions observed in Borehole BH5 the sandy silt subgrade may be very moist to wet, and likely sensitive to disturbance. Groundwater control with conventional sumps and shoring may be required. A mud-mat over the exposed subgrade may be advised to ensure that the base remains in a stable condition.

**The Wet Well**

At the time of this investigation the exact depth of the wet well was unknown and will be determined through detail design. However, shoring will likely be required during the wet well excavation. In addition to excavation support through the compact upper materials, it will also assist with groundwater control. Shoring will function as a cut-off wall for the water-bearing silty sand soils as well as limiting seepage into the excavation from possible wet seams. Shoring comments have been provided in Section 4.3.
Below Grade Structures

All below grade structure walls should be damp-proofed and must be designed to resist a horizontal earth pressure ‘p’ at any depth ‘h’ below the surface as given by the following expression:

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

where, \( P \) = lateral earth pressure in kPa acting at depth \( h \);
\( \gamma \) = natural unit weight, a value of 20.4 kN/m\(^3\) may be assumed;
\( h \) = depth of point of interest in m;
\( q \) = equivalent value of any surcharge on the ground surface in kPa.
\( K \) = earth pressure coefficient, assumed to be 0.4

All surface loads must be considered in the calculation of the lateral earth pressures.

Installation of perimeter drains is required for below grade structures. The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall.

Where hydrostatic pressures cannot be relieved, they must be added to the lateral earth pressures. This is discussed further in the section below.

Waterproofing and Buoyancy Effects

Damp proofing of below-grade structures will be required. In addition, a waterstop membrane should be installed between the footing pad and the foundation wall in order to prevent the flow of water between the base of the foundation wall and the top of the footing.

For below grade structures, the installation of a perimeter drain is recommended to alleviate the build-up of any hydrostatic pressure behind the wall. In the event perimeter drainage is not provided around the wet well and valve chamber structures, waterproofing will also be required, and the structure must be designed to withstand uplift forces due to buoyancy effects. For design purposes, the long term static groundwater table should be assumed at a maximum depth of 1.5 m below the existing grades. The groundwater table could rise to near surface around the deeper structures in wetter seasons. To resist the uplift forces when empty, the weight of the below grade structure can be considered. Additional resistance to uplift can be achieved by extending the footings laterally around the perimeter of the structure, such that the weight of the backfill around the structure overlying the footing extension can also be used to resist the effects of uplift. Alternatively, the net weight of the structure can be increased. Further comments pertaining to uplift calculations are provided on Drawing 7.
Any portion of the structure which extends below the high water level should be designed to resist uplift due to hydrostatic pressures. The magnitude of the hydrostatic uplift may be calculated using the following expression:

\[ P = \gamma \times d \]

where:

- \( P \) = hydrostatic uplift pressure acting on the base of the structure (kPa);
- \( \gamma \) = unit weight of water - use 9.8 kN/m\(^3\);
- \( d \) = depth of base of the structure below the design high water level (m).

Where joints or connections in the structure occur, consideration should be given to providing suitable waterproofing to restrict groundwater into the structure.

As discussed in the previous section, the lateral earth pressures are based on adequate drainage being provided to prevent build-up of hydrostatic pressure. For structures below the ground water level, such as the wet well, lateral earth pressures can be calculated using the following formula:

\[ P = K (q + \gamma_s d_1 + \gamma_{sub} d_2) + \gamma_w d_2 \]

where, \( P \) = lateral earth pressure in kPa acting at depth \( d_2 \);

- \( \gamma_s \) = natural unit weight of moist soil, a value of 21.0 kN/m\(^3\) may be assumed;
- \( \gamma_{sub} \) = submerged soil, 11 kN/m\(^3\);
- \( \gamma_w \) = unit weight of water, 9.81 kN/m\(^3\);
- \( d_1 \) = depth to the groundwater level in m;
- \( d_2 \) = depth below the groundwater level in m;
- \( q \) = equivalent value of any surcharge on the ground surface in kPa;
- \( K \) = earth pressure coefficient, assumed to be 0.4.

All surface loads must be considered in the calculation of the lateral earth pressures.
Slab-On-Grade Floor

The floor slab for the Pumping Station may be constructed using conventional concrete slab-on-grade techniques. The floor slab subgrade should be prepared in accordance with the recommendations provided in Section 4.2.

No special underfloor drains are required provided that the exterior grades are lower than the floor slab, and positively sloped away from the slab.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear stone, should be placed between the prepared subgrade and the floor slab. The installation and requirement of a vapor barrier under the slab should conform to the flooring manufacturer’s and designer’s requirements. Moisture emission testing will be required to determine the concrete condition prior to flooring installation. On-going liaison from this office will be required.

For design, the modulus of subgrade reaction (k) can be taken as 30 MPa/m for the compacted stone layer over the natural subgrade soils.

The floor slab of below grade structures can be cast directly on the natural soils. It is recommended to keep the floor slab a minimum of 0.5 m above the groundwater table. Prior to placement of the slab, the exposed subgrade should be assessed by a Geotechnical Engineer to ensure the absence of any soft soils. Any soft spots encountered will require subexcavation and replacement with compacted fill. In order to minimize moisture flux through the below grade structure floor slab, a minimum 10 mil thick poly barrier should be provided beneath the concrete slab.

The water-to-cement ratio and slump of concrete utilized in the floor slabs should be strictly controlled to minimize shrinkage of the slabs. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.

Foundation Backfill

In general, the natural soils excavated from the foundation and service trench areas should be suitable for reuse as foundation wall backfill provided the work is carried out during relatively dry weather. Any excavated soils proposed for re-use as backfill should be examined by a Geotechnical Engineer. The materials to be re-used should be within three percent of optimum moisture for best compaction results. If the weather conditions are very wet during construction, then imported granular material such as OPSS Granular 'B' should be used.

The backfill must be brought up evenly on both sides of walls not designed to resist lateral earth pressures. Drainage and backfill recommendations are given in **Drawing 4**.

The fill surface around the perimeter of structures should be sloped in such a way that the surface runoff water does not accumulate around the structure. It is recommended that an impermeable soil seal such as clay, asphalt or concrete be provided on the surface to minimize water infiltration.
4.10 Low Impact Development (LID)

It is understood that LID stormwater management design requires the practical availability of unsaturated, sufficiently pervious soil with depth and aerial extent to accommodate the infiltration of stormwater run-off created by land development.

Three (3) locations were selected for in situ testing of the LID capacity of the existing soils. The three locations are as follows:

- Test Pit 1 (TP 1) – located near BH5 and BH6
- Test Pit 2 (TP 2) – located near BH7
- Test Pit 3 (TP 3) – located near BH3

Based on the information collected at the nearby borehole locations and information during preparation for in situ testing, and the above cited criteria, the existing material encountered at the in situ test locations has some potential for use in LID stormwater management design. The LID area will be limited by the sidewalk, trees and driveways of the surrounding area within the boulevard. The following table summarizes the depths where the upper surface of potential LID material was encountered, the depth to the water table and the approximate depth of the underlying impervious soil.

<table>
<thead>
<tr>
<th>Test Hole ID</th>
<th>Depth of Top of LID Soil (m bgs)</th>
<th>Groundwater Depth (m bgs)</th>
<th>Depth of Underlying Impervious Soils (m bgs)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP1</td>
<td>0.35</td>
<td>Unknown</td>
<td>&gt; 6.6</td>
<td>No groundwater was encountered in the test hole</td>
</tr>
<tr>
<td>TP2</td>
<td>0.35</td>
<td>Unknown</td>
<td>&gt; 6.6</td>
<td>No groundwater was encountered in the test hole</td>
</tr>
<tr>
<td>TP3</td>
<td>0.30</td>
<td>Unknown</td>
<td>3.6</td>
<td>No groundwater was encountered in the test hole</td>
</tr>
<tr>
<td>BH1</td>
<td>0.10</td>
<td>Dry</td>
<td>0.60</td>
<td>Low permeable clayey silt till encounter below 0.60, limited potential</td>
</tr>
<tr>
<td>BH2</td>
<td>0.075</td>
<td>4.0</td>
<td>0.90</td>
<td>Low permeable clayey silt till encounter below 0.90, limited potential</td>
</tr>
<tr>
<td>BH3/MW</td>
<td>0.30</td>
<td>Dry</td>
<td>3.7</td>
<td>3.7 m of sandy silt above impervious clayey silt</td>
</tr>
<tr>
<td>BH4</td>
<td>0.075</td>
<td>Dry</td>
<td>0.075</td>
<td>No LID material available</td>
</tr>
<tr>
<td>BH5/MW</td>
<td>0.40</td>
<td>3.17</td>
<td>&gt; 6.6</td>
<td>Sandy silt with groundwater measured at 3.17 m bgs</td>
</tr>
<tr>
<td>BH6</td>
<td>0.30</td>
<td>Dry</td>
<td>&gt; 9.6</td>
<td>Sandy Silt</td>
</tr>
<tr>
<td>BH7/MW</td>
<td>0.20</td>
<td>3.47</td>
<td>&gt; 6.6</td>
<td>Sandy silt with groundwater measured at 3.47 m bgs</td>
</tr>
</tbody>
</table>

Note 1: Depth to impervious soils based on borehole information closest to the in situ location
Infiltration tests were carried out using Saturo Dual Head Infiltrometer system to obtain field saturated hydraulic conductivity values of the near surface soils. Testing was complete in manual dug holes and after removing the topsoil at each location.

The Saturo Dual Head Infiltrometer has the ability to measure infiltration rates between $1.1 \times 10^{-6}$ to $3.2 \times 10^{-2}$ cm/s (Meter Group, 2018). Four (4) grain size analyses were conducted on soil samples retrieved from BH3, BH4, BH and BH7. Hydraulic conductivity values were determined using the Hazen and Puckett methods based on laboratory results. The results of the mechanical grain size analyses are described in the Table below and included in Appendix B.

<table>
<thead>
<tr>
<th>Test Hole ID (depth)</th>
<th>Method</th>
<th>Field Saturated Hydraulic Conductivity ($K_s$)</th>
<th>Estimated T-Time (min/cm)</th>
<th>Infiltration Rate (mm/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP1 (0.5 m)</td>
<td>Dual Head Infiltrometer</td>
<td>$1.0 \times 10^{-3}$ cm/s</td>
<td>8</td>
<td>75</td>
</tr>
<tr>
<td>TP2 (0.7 m)</td>
<td>Dual Head Infiltrometer</td>
<td>$2.7 \times 10^{-3}$ cm/s</td>
<td>7</td>
<td>90</td>
</tr>
<tr>
<td>TP3 (0.6 m)</td>
<td>Dual Head Infiltrometer</td>
<td>$8.1 \times 10^{-3}$ cm/s</td>
<td>5</td>
<td>130</td>
</tr>
<tr>
<td>BH3/MW (0.8-2.3 m)</td>
<td>Hazen</td>
<td>$4.9 \times 10^{-5}$ cm/s</td>
<td>16</td>
<td>40</td>
</tr>
<tr>
<td>BH4 (0.8 m)</td>
<td>Puckett</td>
<td>$3.3 \times 10^{-7}$ cm/s</td>
<td>&gt; 50</td>
<td>&lt;12</td>
</tr>
<tr>
<td>BH5/MW (3.8-4.6 m)</td>
<td>Hazen</td>
<td>$2.5 \times 10^{-4}$ cm/s</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>BH7/MW (0.8-2.3 m)</td>
<td>Hazen</td>
<td>$1.0 \times 10^{-4}$ cm/s</td>
<td>11</td>
<td>50</td>
</tr>
</tbody>
</table>

It is understood that recommended factors of safety will be applied to the estimated parameters cited above for use in design.

4.11 Earthquake Design Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design using the OBC 2012 are presented below.

The subsoil and groundwater information at this Site have been examined in relation to Section 4.1.8.4 of the OBC 2012. The subsoils at the Site generally consist of loose to dense sandy silt and firm to very stiff clayey silt till deposits. It is anticipated that the proposed structures will be founded on the natural, inorganic sandy silt and clayey silt till deposits.
Earth and Environmental Division - Geotechnical

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m (below the lowest basement level) are to be used. The boreholes advanced at this Site were excavated to a maximum depth of 9.6 m below existing grade. Therefore, the Site Classification recommendation would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the area.

Based on the above assumptions, interpretations in combination with the known local geological conditions, the Site Class for the proposed development is “D” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. Additional depth drilling may be advised to determine if the soil conditions below the current depth of exploration can support a higher Site Classification.

4.12 Site Pavement Design

Areas to be paved should be stripped of all topsoil, fill, organics and other obviously unsuitable material. The exposed subgrade must then be thoroughly proof-rolled. Any soft spots revealed by this or any other observations must be over-excavated and backfilled with approved material. All fill required to backfill service trenches or to raise the subgrade to design levels must conform to requirements outlined previously. Preferably, the natural inorganic excavated soils should be used to maintain uniform subgrade conditions, provided adequate compaction can be achieved.

Provided the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated specified classification (local roads internal to the site) and anticipated subgrade conditions.

### Recommended Pavement Structure Thickneses

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Compaction Requirements</th>
<th>Neighborhood Street</th>
<th>Neighborhood Connector</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Asphaltic Concrete</strong></td>
<td>92% MRD(^1) or 97% BRD(^1)</td>
<td>40 mm HL-3 or SP12.5</td>
<td>40 mm HL-3 or SP12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 mm HL-8 or SP19</td>
<td>2 x 45 mm HL-8 or SP19</td>
</tr>
<tr>
<td><strong>Granular ‘A’ (Base)</strong></td>
<td>100% SPMDD(^1)</td>
<td>150 mm</td>
<td>150 mm</td>
</tr>
<tr>
<td><strong>Granular ‘B’ (Base)</strong></td>
<td>100% SPMDD(^1)</td>
<td>300 mm</td>
<td>450 mm</td>
</tr>
</tbody>
</table>

*Notes:  
1) SPMDD denotes Standard Proctor Maximum Dry Density, MRD denotes Maximum Relative Density, BRD denotes Bulk Relative Density.  
2) The subgrade must be compacted to 98% SPMDD.  
3) The above recommendations are minimum requirements.*

The recommended pavement structures provided in the above table are based on the existing subgrade soil properties determined from visual examination and textural classification of the soil samples. Consequently, the recommended pavement structures should be considered for preliminary design purposes only. Other granular configurations may also be possible provided the granular base equivalency (GBE) thickness is maintained. These recommendations on thickness design are not intended to support heavy and concentrated construction traffic, particularly where only a portion of the pavement section is installed.
If construction is undertaken under adverse weather conditions (i.e., wet or freezing conditions) subgrade preparation and granular sub-base requirements should be reviewed by the geotechnical engineer. If the sub-base is set on wet or dilatant silty soils, a geotextile will be required. A woven type geotextile such as Terrafix 200W or equivalent would be suitable for this application.

If only a portion of the pavement will be in place during construction, the granular subbase may have to be thickened, and/or the subgrade improved with a geotextile separator or geogrid stabilizing layer. This is best determined in the field during the site servicing stage of construction, prior to road construction.

Samples of both the Granular 'A' and Granular 'B' aggregates should be checked for conformance to OPSS 1010 and City of London standards prior to utilization on Site, and during construction. The Granular 'B' subbase and the Granular 'A' base courses must be compacted to 100 percent SPMDD.

The asphaltic concrete paving materials should conform to the requirements of OPSS 1150. The asphalt should be placed in accordance with OPSS 310 and compacted to at least 97 percent of the Marshall mix design bulk relative density or 92% of maximum relative density. A tack coat should be applied between the surface and binder asphalt courses.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. In low areas, sub-drains should be installed to intercept excess subsurface moisture and prevent subgrade softening, as shown on Drawing 8. This is particularly important in heavier traffic areas at the site entrances. The locations and extent of sub-drainage required within the paved areas should be reviewed by this office in conjunction with the proposed grading.

A program of in situ density testing must be carried out to verify that satisfactory levels of compaction are being achieved.

To minimize the effects of differential settlements of service trench fill, it is recommended that wherever practical, placement of binder asphalt be delayed for approximately six months after the granular sub-base is put down. The surface course asphalt should be delayed for a further one year. Prior to the surface asphalt being placed, it is recommended that a pavement evaluation be carried out on the base asphalt to identify repair areas or areas requiring remedial works prior to surface asphalt being placed.

4.13 Curbs and Sidewalks

It is recommended that the concrete for curb and gutter and sidewalks should be proportioned, mixed, placed, and cured in accordance with the requirements of OPSS 353, OPSS 1350 and City of London Specifications for Curbs and Sidewalks (refer to current City of London Drawings).

During cold weather, the freshly placed concrete must be covered with insulating blankets to protect against freezing. Three cylinders from each day’s pour should be taken for compressive strength testing. Air entrainment, temperature, and slump tests should be made from the same batch of concrete from which test cylinders are made.

The subgrade for the sidewalks should comprise undisturbed natural competent soil of well-compacted fill. A minimum 150 mm thick layer of compacted Granular 'A' type aggregate should be placed beneath the sidewalk slabs.
It is recommended that the Granular 'A' be compacted to a minimum 100 percent SPMDD, to provide adequate support for the concrete sidewalk. Construction traffic should be kept off the placed curbs and sidewalks as they are not designed to withstand heavy traffic load.

4.14 Inspection and Testing Requirements

An effective inspection and testing program is an essential part of construction monitoring. The Inspection and Testing Program typically includes the following items:

- Subgrade examination prior to engineered fill placement;
- Inspection and Materials testing during engineered fill placement (full-time supervision is recommended) and site servicing works, including soil sampling, laboratory testing (moisture contents and Standard Proctor density test on the pipe bedding, trench backfill and engineered fill material), monitoring of fill placement, and in situ density testing;
- Footing Base Examinations to confirm suitability to support the design bearing pressures; and, visual examination of concrete reinforcing steel placement in footings set on engineered fill.
- Inspection and testing for underfloor subgrade and granular placement.
- Materials testing for concrete foundations, floor slab, curbs and sidewalks.
- Inspection and Materials testing during paved area construction, including subgrade examination of the paved area subgrade soils following site servicing, laboratory testing (grain size analyses and Standard Proctor density tests on the Granular A and B material placed on site roadways), and in situ density testing;
- Inspection and Materials testing for base and surface asphalt, including laboratory testing on asphalt sampling and Benkelman beam deflection testing to confirm conformance to project specifications and standards;

EXP would be pleased to prepare an inspection and testing work program prior to construction, incorporating the above items.
5. Hydrogeological Comments

Based on our understanding of the proposed development and the results of the current investigation, the following paragraphs provide hydrogeological comments and discussion pertaining to the proposed development.

The Ministry of Environment, Conservation and Parks (MECP) Well Records for this area are summarized in Section 3.5. Groundwater information provided by MECP Records indicate that groundwater in the area is generally sourced from bedrock aquifers (27 m to 41 m depth).

It is noted that the site is located within the community of Jackson (see map below) and municipal water will be supplied to the development.
Shallow groundwater flow across the site is typically affected by the soil permeability, topography and drainage. Intermediate and deep aquifers are significantly less affected by surface conditions.

Based on the test holes information for the site, the proposed sewers and water mains are generally expected to be set within the sandy silt or clayey silt till strata. Localized discontinuous lenses of wet silt and sand are typical within the glaciolacustrine deposits, and could be contacted in the base of the service trench excavations along the servicing alignment.

The texture of the predominant soils in the area is described as sandy silt or clayey silt till. The sandy silt or clayey silt till deposits generally have a very low permeability and act as an effective barrier to minimize vertical and horizontal groundwater movement.

Based on the results of the current investigation, no significant short- or long-term impact is anticipated on the nearby wells, either quantitatively and qualitatively since the proposed inverts of the sewers are typically not deep enough to penetrate into the underlying intermediate or deep aquifers, if any. Any temporary dewatering operations which may be required to deal with minor seepage from localized pockets of sand and silt are not expected to cause any long-term impacts to the aquifers which supply the nearby potable wells.

In any event, native backfill should be used where possible to minimize the change in hydraulic conductivity within the service trenches. In the event the sewer excavations encounter any wet sandy soils, and for those areas where the excavations extend below the stabilized shallow groundwater table, clay collars may be installed at strategic locations, if necessary, as part of the contingency plan. This can be best assessed during the early stage of construction by a Geotechnical Engineer.

The actual design and location of collars should be assessed at the final design phase and confirmed during construction. However, the following comments are provided for guidance:

- Clay collar/plug shall be constructed of plastic clayey silt with moisture content about 3% above optimum moisture as determined by the standard Proctor density test.
- Excavation shall be carried out to the specified dimensions indicated on the sketches.
- The excavation shall be backfilled with compacted silty clay to at least 98 percent SPMDD to minimize settlement.
A schematic diagram for the design and use of clay collars is provided below for your review and consideration.

The use of conventional water-tight gaskets are considered suitable to prevent groundwater migration into the structures and pipes.

**UTRCA Regulated Lands**

A review of the UTRCA Regulated Area Screening Map indicates that the subject Site is located on the border of a Regulated Area (O.Reg 157/06 under O.Reg 97/04) to the west between the Site and Highbury Road North. Therefore, it is expected that permission is not required from the UTRCA in regards to development/site alteration as the area is outside of the regulated area.
6. General Comments

The information presented in this report is based on a limited investigation designed to provide information to support an assessment of the current geotechnical conditions within the subject property. The conclusions and recommendations presented in this report reflect site conditions existing at the time of the investigation. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent. Should this occur, EXP Services Inc. should be contacted to assess the situation, and the need for additional testing and reporting. EXP has qualified personnel to provide assistance in regards to any future geotechnical and environmental issues related to this property.

Our undertaking at EXP, therefore, is to perform our work within limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession.

The comments given in this report are intended only for the guidance of design engineers. The number of test holes required to determine the localized underground conditions between test holes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. 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 results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

EXP Services Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not afforded the privilege of making this review, EXP Services Inc. will assume no responsibility for interpretation of the recommendations in this report.

This report was prepared for the exclusive use of Dillon Consulting Limited and may not be reproduced in whole or in part, without the prior written consent of EXP, or used or relied upon in whole or in part by other parties for any purposes whatsoever. Any use which a third party makes of this report, or any part thereof, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. 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 trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.
Drawings
-NOTES-

1. The boundaries and soil types have been established only at test hole locations. Between test holes they are assumed and may be subject to considerable error.
2. Soil samples will be retained in storage for 3 months and then destroyed unless client advises that an extended time period is required.
3. Topsoil quantities should not be established from the information provided at the test hole locations.
4. The site plan was reproduced from City of London Mapping and should be read in conjunction with EXP Geotechnical Report LON-00017363-GE.

Geotechnical Investigation
Forest View Development
101 Meadowlily Road South, London, Ontario
DRAWING 2 – GEOMETRIC REQUIREMENTS FOR FOUNDATIONS ON ENGINEERED FILL

Schematic (Not to Scale)

A. Foundation Walls

Min. 1.2 m

Min. 0.6 m

Underfloor Fill

Min. 2B

Engineered Fill

Competent Natural Soil

B. Foundation Walls

Min. 1.2 m

Min. 0.6 m

Engineered Fill

Undisturbed Natural Soil

To Be Benched

SECTION VIEW

Section A – Typical Section of Slab-on-Grade Building
Section B – Typical Section of Building with Basement

Refer to Detailed Notes on following page.
NOTES FOR ENGINEERED FILL PLACEMENT:

1. The area must be stripped of all topsoil contaminated fill material, and other unsuitable soils, and proof rolled. Soft spots must be dug out. The stripped natural subgrade must be examined and approved by an EXP Engineer prior to placement of engineered fill.

2. In areas where engineered fill is placed on a slope, the fill should be benched into the approved subgrade soils. EXP would be pleased to provide additional comments and recommendations in this regard, if required.

3. All excavations must be carried out in accordance with the Occupational Health and Safety Regulation of Ontario (Construction Projects - O.Reg. 213.91)

4. Material used for engineered fill must be free of topsoil, organics, frost and frozen material, and otherwise unsuitable or compressible soils, as determined by a Geotechnical Engineer. Any material proposed for use as engineered fill must be examined and approved by EXP, prior to use onsite. Clean compactable granular fill is preferred.

5. Approved engineered fill should be placed in maximum 300 mm thick lifts, and uniformly compacted to 100% Standard Proctor dry density throughout. For best compaction results, engineered fill should be within 3 percent of its optimum moisture content, as determined by the Standard Proctor density test. Imported fill should satisfy the MECP regulations and requirements.

6. Full time geotechnical monitoring, inspection and in situ density (compaction) testing by EXP is required during placement of the engineered fill.

7. Site grades should be maintained during area grading activities to promote drainage, and to minimize ponding of surface water on the engineered fill mat. Rutting by construction equipment should be kept to a minimum, where possible. Additional work to ensure suitability of engineered fill may be required if fill is placed in extreme (hot/cold) weather.

8. The fill must be placed such that the specified geometry is achieved. Refer to sketches (previous page) for minimum requirements. Proper environmental protection will be required, such as providing frost penetration during construction, and after the completion of the engineered fill mat.

9. An allowable bearing pressure of 145 kPa (3000 psf) may be used provided that all conditions outlined above, and in the Geotechnical Report are adhered to.

10. These guidelines are to be read in conjunction with the attached Geotechnical Report. (EXP Project No. LON-00017363-GE)

11. For foundations set on engineered fill, footing enhancement and/or concrete reinforcing steel placement is recommended. The footing geometry and extent of concrete reinforcing steel will depend on site specific conditions. In general, consideration may be given to having a minimum strip footing width of 500 mm (20 inches), containing nominal steel reinforcement. Alternatively, concrete reinforcement may be recommended in the top and bottom of the foundation wall strip. The final footing geometry and extent of reinforcement is best determined in the field, by a Geotechnical Engineer.
NOTES:

1. Drainage tile to consist of 100 mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150 mm (6 in.) below underside of interior floor slab.
2. Pea gravel 150 mm (6 in.) top and sides of drain. If drain is not on footing, place 100 mm (4 in.) of pea gravel below drain. 20 mm (3/4 in.) clear stone may be used provided if it is covered by an approved porous geotextile fabric membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12 in.) top and side of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted. Compact backfill to 95 percent Standard Proctor Maximum Dry Density.
5. The interior fill may be any clean, inorganic soil which may be compacted to at least 95 percent Standard Proctor density in this confined space.
6. Do not use heavy compaction equipment within 450 mm (18 in.) of the wall. Do not fill or compact within 1.8 m (6 ft) of wall unless fill is placed on both sides simultaneously.
7. Moisture barrier to be at least 200 mm (8 in.) of compacted 20 mm (3/4 in.) clear, crushed stone or equivalent free-draining material.
8. If the 20 mm (3/4 in.) clear stone requires surface binding, use 60 mm (1/4 in.) clear stone chips.
9. Slab on grade should not be structurally connected to wall or footing.
10. Exterior grade to slope away from building.

This system is not normally required if the floor is at least 300 mm (1 ft.) above exterior grade.
NOTES:

1. Drainage tile to consist of 100 mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150 mm (6 in.) below underside of floor slab.

2. Pea gravel 150 mm (6 in.) top and sides of drain. If drain is not on footing, place 100 mm (4 in.) of pea gravel below drain. 20 mm (3/4 in.) clear stone may be used provided if it is covered by an approved porous geotextile fabric membrane (Terrafix 270R or equivalent).

3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12 in.) top and side of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).

4. Free-draining backfill - OPSS Granular B or equivalent compacted to 93 to 95% (maximum) percent Standard Proctor density. Do not compact closer than 1.8 m (6 ft) from wall with heavy equipment. Use hand controlled light compaction equipment within 1.8 m (6 ft) of wall.

5. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted.

6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.

7. Moisture barrier to consist of compacted 20 mm (3/4 in.) clear, crushed stone or equivalent free-draining material. Layer to be 200 mm (8 in.) minimum thickness.

8. Basement walls to be damp-proofed.

9. Exterior grade to slope away from wall.

10. Slab on grade should not be structurally connected to wall or footing.

11. Underfloor drain invert to be at least 300 mm (12 in.) below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25 ft.) centres one way. Place drain on 100 mm (4 in.) of pea gravel with 150 mm (6 in.) of pea gravel top and sides. CSA fine concrete aggregate to be provided as filter material or an approved porous geotextile membrane (as in 2 above) may be used.

12. Do not connect the underfloor drains to perimeter drains.

13. If the 20 mm (3/4 in.) clear stone requires surface binding, use 6 mm (1/4 in.) clear stone chips.

Note:

a) Underfloor drainage can be deleted where not required (see report).

b) Free draining backfill, item 4 may be replaced by wall drains, as indicated, if more economical.
NOTES:

ZONE A
Granular bedding satisfying current City of London Standards compacted to 95% Standard Proctor maximum dry density.

ZONE A-I
To be compacted to 95% Standard Proctor maximum dry density.

ZONE B
To be compacted to 95% Standard Proctor maximum dry density.

ZONE C
To be compacted to 98% Standard Proctor maximum dry density.

The excavations shown above are for Type 1 or 2 soils. Where excavations extend through Type 3 soils, the side walls should be sloped back at a maximum inclination of 1 horizontal to 1 vertical from the base (Reference O.Reg 219/31).
Requirements for backfill in service trenches, etc. should conform to current City of London and OPS requirements. A summary of the general recommendations for trench backfill is presented on Drawing 4.

The bedding materials for the services designated as Zone A on the attached drawings should consist of approved granular material satisfying the current OPS minimum standards and specifications. (Class B bedding should provide adequate support for the pipes). These materials should be uniformly compacted to 95 percent of standard Proctor dry density. Some problems may be encountered in maintaining alignment when bedding pipes in wet sandy soil. If Granular ‘A’ or other sandy material is used for bedding, they may become ‘spongy’ when saturated. If significant amounts of clear stone are used to stabilize the base, a geotextile should be incorporated to avoid problems with migration of fine grained materials and differential settlement under the pipes as the groundwater rises after backfilling. For minor local use of crushed stone without a geotextile filter, a graded HL3 stone is preferable.

The backfill in Zone B will consist of the native material. This material should be placed in loose lifts not exceeding 300 mm (12 inches) and be uniformly compacted to at least 95 percent of the standard Proctor maximum dry density. Material wetter than 5 percent above optimum must be allowed to dry sufficiently or should be discarded or used in landscaped areas.

The upper 1 meter of the general backfill (i.e. Zone C) should be placed in loose lifts not exceeding 300 mm (12 inches) and be uniformly compacted to at least 98 percent of the standard Proctor maximum dry density. To achieve satisfactory compaction, the fill material should be within 3 percent of standard Proctor optimum moisture content at placement.
NOTES:

Factor of safety against uplift, $F_{Su}$, is given by

$$F_{Su} = \frac{W_w}{W_a}$$

Where

$W_w = $ total weight of water displaced (hatched zone)

$W_a = $ total weight acting downward, (assume plane A)

$W_{so} = $ weight of concrete structure

$W_s = $ weight of soil = 2 \{B_1 h_2 (\Sigma b - \Sigma w) + B_1 h_2 \Sigma a\}

F = $ friction = average value of lateral earth pressure $x$

$$\text{coeff of friction} \times 2H = K_0 \Sigma H^2 f$$

A = $ adhesion a (if backfill is cohesive) \times 2H 2aH$

In most cases granular backfill is used and the following parameters apply:

$$K_0 = 0.4, \Sigma b = 19 \text{ kN/m}^3, f = 0.4, a = 0$$

For cohesive backfill, values of “a” range from 10 to 20 kPa

Suggested value of $F_{Su}$:

$$F_{Su}$$

- 1.5 if using plane B
- 1.0 if using plane A
NOTES:

1. All dimensions in millimetres.
2. All sub drains to be set on at least 1% grade draining to a positive outlet.
3. Subgrade soil conditions should be verified onsite, during subgrade preparation works, following site servicing installations.

Scale: NTS
Appendix A – Borehole Logs and Gradation Analysis Results
NOTES ON SAMPLE DESCRIPTIONS

1. All descriptions included in this report follow the 'modified' Massachusetts Institute of Technology (M.I.T.) soil classification system. The laboratory grain-size analysis also follows this classification system. Others may designate the Unified Classification System as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain size analysis has been carried out, all samples are classified visually and the accuracy of the visual examination is not sufficient to differentiate between the classification systems or exact grain sizing. The M.I.T. system has been modified and the EXP classification includes a designation for cobbles above the 75 mm size and boulders above the 200 mm size.

<table>
<thead>
<tr>
<th>UNIFIED SOIL CLASSIFICATION</th>
<th>M.I.T. SOIL CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fines (silt and clay)</td>
<td>Clay</td>
</tr>
<tr>
<td>Sand</td>
<td>Silt</td>
</tr>
<tr>
<td>Gravel</td>
<td>Sand</td>
</tr>
<tr>
<td>Cobbles</td>
<td>Gravel</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sieve Sizes</th>
<th>0.002</th>
<th>0.06</th>
<th>0.075</th>
<th>0.12</th>
<th>0.4</th>
<th>0.66</th>
<th>2.0</th>
<th>5.0</th>
<th>20</th>
<th>35/4</th>
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<tr>
<td>Particle Size (mm)</td>
<td></td>
<td>200</td>
<td>400</td>
<td>80</td>
<td>10</td>
<td>5.0</td>
<td>4</td>
<td>20</td>
<td>35/4</td>
<td>80</td>
</tr>
</tbody>
</table>

2. **Fill:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description therefore, may not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces or subsurface basements, floors, tanks, even though none of these obstructions may have been encountered in the borehole. Despite the use of boreholes, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. The fill at this site has been monitored for the presence of methane gas and the results are recorded on the borehole logs. The monitoring process neither indicates the volume of gas that can be potentially generated or pinpoints the source of the gas. These readings are to advise of a potential or existing problem (if they exist) and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic waste that renders the material unacceptable for deposition in any but designated land fill sites; unless specifically stated, the fill on the site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common, but not detectable using conventional geotechnical procedures.

3. **Glacial Till:** The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process, the till must be considered heterogeneous in composition and as such, may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm in diameter) or boulders (greater than 200 mm diameter) and therefore, contractors may encounter them during excavation, even if they are not indicated on the borehole logs. It should be appreciated that normal sampling equipment can not differentiate the size or type of obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited area; therefore, caution is essential when dealing with sensitive excavations or dewatering programs in till material.
**BOREHOLE LOG**

**CLIENT**  Dillon Consulting Limited  
**PROJECT**  Forest View Development  
**LOCATION**  101 Meadowlily Rd. S., London, ON  
**DATING**  Boring August 21, 2019  

<table>
<thead>
<tr>
<th>STRATA DESCRIPTION</th>
<th>DEPTH (m bgs)</th>
<th>SAMPLES</th>
<th>MOISTURE (% blow)</th>
<th>RECOVERY (%)</th>
<th>SPT N VALUE</th>
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</thead>
<tbody>
<tr>
<td>ASPHALT - 100 mm</td>
<td>281.6</td>
<td>SS S1</td>
<td>9</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>SAND AND GRAVEL - brown, compact, moist</td>
<td>281.0</td>
<td>SS S2</td>
<td>17</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>CLAYEY SILT TILL - brown, trace sand, trace gravel, stiff to very stiff, damp</td>
<td>-1</td>
<td>SS S3</td>
<td>15</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-2</td>
<td>SS S4</td>
<td>21</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-3</td>
<td>SS S5</td>
<td>19</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-4</td>
<td>SS S6</td>
<td>12</td>
<td>16</td>
<td></td>
</tr>
</tbody>
</table>

End of Borehole at 5.0 m bgs.

**NOTES**
1) Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
2) Borehole was open to 4.3 m bgs and dry upon completion of drilling.
3) bgs denotes below ground surface.
4) No significant methane gas concentration was detected upon completion of drilling.
**BOREHOLE LOG**

**CLIENT** Dillon Consulting Limited  
**PROJECT** Forest View Development  
**LOCATION** 101 Meadowlily Rd. S., London, ON  
**DATE:** August 21, 2019

<table>
<thead>
<tr>
<th>DEPTH (m bgs)</th>
<th>STRATA DESCRIPTION</th>
<th>SAMPLES</th>
<th>SPT N Value</th>
<th>MOISTURE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>271.5</td>
<td>ASPHALT - 75 mm</td>
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<td>11</td>
<td>6</td>
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<tr>
<td>270.6</td>
<td>SAND AND GRAVEL - brown, compact, moist</td>
<td>SS S2</td>
<td>100</td>
<td>19</td>
</tr>
<tr>
<td>269.1</td>
<td>CLAYEY SILT TILL - brown, trace gravel, stiff to very stiff, damp</td>
<td>SS S3</td>
<td>100</td>
<td>17</td>
</tr>
<tr>
<td>266.5</td>
<td>SANDY SILT - brown, dilatant, compact, wet</td>
<td>SS S4</td>
<td>100</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>- trace to some clay beginning near 4.4 m bgs</td>
<td>SS S5</td>
<td>100</td>
<td>12</td>
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<tr>
<td></td>
<td>End of Borehole at 5.0 m bgs.</td>
<td>SS S6</td>
<td>100</td>
<td>26</td>
</tr>
</tbody>
</table>

**NOTES**
1) Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
2) Borehole was open to 4.0 m bgs and groundwater encountered near 4.0 m bgs upon completion of drilling.
3) bgs denotes below ground surface.
4) No significant methane gas concentration was detected upon completion of drilling.

**SHEAR STRENGTH**
- S Field Vane Test (#=Sensitivity)
- Penetrometer
- Torvane

**WATER LEVELS**
- Measured
- Artesian (see Notes)
BOREHOLE LOG

CLIENT: Dillon Consulting Limited
PROJECT: Forest View Development
LOCATION: 101 Meadowlily Rd. S., London, ON

DATE: Boring August 20, 2019
DATUM: Geodetic

TOPSOIL - 300 mm
- SANDY SILT - brown, weathered in upper limits, dilatant layering, loose to compact, moist
  - becoming wet near 2.1 m bgs
  - becoming grey near 3.1 m bgs
- CLAYEY SILT - grey, firm to stiff, moist
- SILTY SAND - grey, very dense, moist

End of Borehole at 9.6 m bgs.

NOTES
1) Borehole Log interpretation requires assistance by EXP before use by others.
   Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) * denotes 51 blows for 150 mm split spoon sampler penetration.
5) Water Level Measurements

SAMPLE LEGEND
- AS Auger Sample SS Split Spoon ST Shelby Tube
- Rock Core (eg. BQ, NQ, etc.) VN Vane Sample

OTHER TESTS
- G Specific Gravity CD Consolidated Drained Triaxial
- H Hydrometer CS Consolidated Undrained Triaxial
- S Sieve Analysis CU Consolidated Undrained Triaxial
- Unit Weight UU Unconsolidated Undrained Triaxial
- Field Permeability UC Unconfined Compression
- K Lab Permeability DS Direct Shear

WATER LEVELS
- % Apparent Measured Artesian (see Notes)

Dillon Consulting Limited
Sheet 1 of 1
### BOREHOLE LOG

**CLIENT** Dillon Consulting Limited  
**PROJECT** Forest View Development  
**LOCATION** 101 Meadowlily Rd. S., London, ON  
**PROJECT NO.** LON-00017363-GE  
**DATE** August 21, 2019  
**_DATUM_** Geodetic  

#### STRATA DESCRIPTION

<table>
<thead>
<tr>
<th>Depth (m bgs)</th>
<th>Description</th>
<th>MOISTURE CONTENT (%)</th>
<th>Atterberg Limits (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>272.4</td>
<td>TOPSOIL - 75 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>272.4</td>
<td>CLAYEY SILT TILL - brown, trace sand, trace clay, firm to very stiff, damp</td>
<td></td>
<td></td>
</tr>
<tr>
<td>270.0</td>
<td>SANDY SILT - brown, weathered, loose to dense, moist</td>
<td></td>
<td></td>
</tr>
<tr>
<td>267.4</td>
<td>- becoming wet and dilatant near 4.5 m bgs</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### End of Borehole at 5.0 m bgs.

#### NOTES

1. Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
2. Borehole was open to 4.0 m bgs and dry upon completion of drilling.
3. bgs denotes below ground surface.
4. No significant methane gas concentration was detected upon completion of drilling.
TOPSOIL - 400 mm

SANDY SILT - brown, weathered, dilatant layering, trace clay, loose to compact, moist becoming wet near 1.5 m bgs

Grain Size
Silt 67%
Sand 30%
Clay 3%

- 150 mm grey clayey silt layer encountered near 6.4 m bgs

End of Borehole at 6.6 m bgs.

NOTES
1) Borehole Log interpretation requires assistance by EXP before use by others.
2) Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
3) No significant methane gas concentration was detected upon completion of drilling.
4) Water Level Measurements
   August 28, 2019 2.85 254.7
   September 20, 2019 3.17 254.4
BH6

BOREHOLE LOG

CLIENT: Dillon Consulting Limited
PROJECT: Forest View Development
LOCATION: 101 Meadowlily Rd. S., London, ON
DATING: Boring August 20, 2019

1) Borehole Log interpretation requires assistance by EXP before use by others.
Borehole Log must be read in conjunction with EXP Report LON00017363-GE.
2) Borehole was open 4.9 m bgs and dry upon completion of drilling.
bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.

DEPTH

- 257.8
- 257.5
- 257.5
- 248.2

TOPSOIL - 300 mm

257.5 - SANDY SILT - brown, weathered in upper limits, dilatant layering, loose to compact, moist
- becoming wet near 1.5 m bgs

- some clay beginning near 7.1 m bgs

End of Borehole at 9.6 m bgs.

Notes:
1) Borehole Log interpretation requires assistance by EXP before use by others.

Atterberg Limits and Moisture

<table>
<thead>
<tr>
<th>Wp</th>
<th>Wl</th>
<th>Ww</th>
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</thead>
<tbody>
<tr>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
</tbody>
</table>

 Other Tests:
- G Specific Gravity
- H Hydrometer
- S Sieve Analysis
- U Unit Weight
- P Field Permeability
- K Lab Permeability
- C Consolidation
- CD Consolidated Drained Triaxial
- CU Consolidated Undrained Triaxial
- UC Unconfined Compression
- DS Direct Shear

WATER LEVELS
- Measured
- Artesian (see Notes)
TOPSOIL - 200 mm

SILTY SAND - brown, weathered in upper limits, dilatant layering, loose to compact, moist

- becoming wet near 1.5 m bgs

- thin clayey silt layering encountered near 3.8 m bgs

End of Borehole at 6.6 m bgs.
Silty Clay, trace sand
Gravel - 2%
Sand - 9%
Silt - 41%
Clay - 48%
Estimated Permeability~3.3x10^{-7} cm/sec
MECHANICAL GRAIN SIZE ANALYSIS

**Sandy Silt, trace clay**
- Sand - 37%
- Silt - 58%
- Clay - 5%

Estimated Permeability ~ $4.9 \times 10^{-5}$ cm/sec

**Sample Description:** BH3 S1-S3 2.5'-7.5'

**Project:** LON00017363

**Meadowlilly**
Sandy Silt, trace clay
Sand - 30%
Silt - 67%
Clay - 3%
Estimated Permeability ~ $1.8 \times 10^{-4}$ cm/sec

<table>
<thead>
<tr>
<th>CLAY</th>
<th>FINE</th>
<th>MEDIUM</th>
<th>COARSE</th>
<th>FINE</th>
<th>MEDIUM</th>
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<tr>
<td>SILT</td>
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</tbody>
</table>

Sample Description: Lab No: 19-571
BHS S5 & S6

Project: LON00017363
Meadowlily

Figure 1
MECHANICAL GRAIN SIZE ANALYSIS

Sandy Silt, trace clay
- Sand: 22%
- Silt: 74%
- Clay: 4%

Estimated Permeability ~ 1.0 x 10^-4 cm/sec

Sample Description: BH7 S1-S3 2.5'-7.5'
Project: LON00017363 Meadowlily

Figure 1
Appendix B – Inspecting and Testing Schedule
The following program outlines suggested minimum testing requirements during backfilling of service trenches and construction of pavements. In adverse weather conditions (wet/freezing), increased testing will be required. The testing frequencies are general requirements and may be adjusted at the discretion of the engineer based on test results and prevailing construction conditions.

## I TRENCH BACKFILL

**ZONE A**
- One in situ density test per 100 cubic meters or 50 linear metres of trench whichever is less
- One laboratory grain size and Proctor density test per 50 density tests or 4000 cubic metres or on change of material (source, visual)

**ZONE A1**
- One in situ density test per 75 cubic metres of material or 25 linear metres of each lift of fill
- One laboratory grain size and Proctor density test per each 50 density tests or 4000 cubic metres of material placed or as directed by the engineer

**ZONES B & C**
- One in situ density test per 150 cubic metres of material or 50 linear metres or each lift whichever is less
- One laboratory grain size and Proctor density test per 50 density tests or 4000 cubic metres of material placed or as directed by the engineer

## II PAVEMENT MATERIALS

**GRANULAR SUBBASE**
- One laboratory grain size and standard Proctor test per 50 density tests or 4000 cubic metres or each change of material (visual, source), as determined by the engineer

**GRANULAR BASE**
- One in situ density test per 50 linear metres of road
- One laboratory grain size and Proctor per 50 density tests or 8000 cubic metres or change in material (visual, source), as determined by the engineer
- Benkelman beam testing at 10 metre intervals per lane, after final grading and compaction. Asphaltic concrete should not be placed until rebound criteria have been satisfied.

**ASPHALTIC CONCRETE**
- One in situ density test per 25 linear metres of roadway
- One complete Marshall Compliance test including stability flow, etc. for each mix type to check mix acceptability. One extraction and gradation test per each day of paving to be compared to job mix formula

**NOTES:** Where testing indicates inadequate compaction, additional fill should not be placed until the area is recompacted and retested at the discretion of the engineer.
Appendix C – Limitations and Use of Report
LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

This report ("Report") is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP's recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelope assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.
RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

STANDARD OF CARE

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

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