Geotechnical Investigation

2411061 Ontario Inc.

Project Name:
Proposed Residential Development
1210 – 1240 Wharncliffe Road South
London, Ontario

Project Number:
LON-23001016-A0

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Geotechnical Report

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1. Introduction and Background

1.1 Introduction

EXP Services Inc. (EXP) was retained by **2411061 Ontario Inc.** to carry out a geotechnical investigation and prepare a geotechnical report relating to the proposed new development at 1210 – 1240 Wharncliffe Road South in London, Ontario, hereinafter referred to as the 'Site'.

It is our understanding that the proposed project will consist of townhouse blocks having a total of 54 units (proposed stacked townhomes) and a stormwater management facility. The residential development is expected to have complete municipal servicing and will have paved access roads.

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 Site development.

1.2 Terms of Reference

The geotechnical investigation was generally completed in accordance with the scope of work outlined in EXP's proposal P22-460 dated January 17, 2023. Authorization to proceed with this investigation was received from Mr. Sam Rattazzi on behalf of **2411061 Ontario Inc.**

The purpose of the investigation was to examine the subsoil and groundwater conditions at the Site by advancing a series of boreholes at the locations chosen by EXP and shown 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, stormwater management facility construction, 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 C** of this report, which contains further information necessary for the proper interpretation and use of this report.

2. Methodology

The fieldwork was carried out between February 16th and 17th, 2023. In general, the geotechnical investigation consisted of the advancement of eight (8) boreholes at the locations shown on **Drawing 1** as BH1 to BH8, inclusive. MW was suffixed to the borehole symbol (BH) where groundwater monitoring wells were installed. A nested well set was installed at BH8A/MW and BH8B/MW.

Prior to drilling, buried service clearances were obtained for the test hole locations by EXP.

The boreholes were completed by a specialist drilling subcontractor under the full-time supervision of EXP geotechnical staff. The boreholes were advanced using 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 and Standard Penetration Test (SPT) methods or auger samples.

During the drilling, the stratigraphy in the test holes 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**. Groundwater monitoring well consisting of 50 mm diameter PVC pipe were installed in Boreholes BH1, BH6, BH7, and BH8. Details of the monitoring well construction are provided on the attached Borehole Logs. Following the drilling, the remaining boreholes were backfilled with the excavated materials and bentonite, to satisfy the requirements of O.Reg. 903

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 of routine moisture content determinations and grain size analysis, with results presented on the borehole logs found in **Appendix A**.

Samples remaining after the classification testing will be stored for a period of three months following the issuance of this report (i.e., until May 2023). 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 a site plan provided by the Client. The ground surface is provided by the Government of Ontario Digital Terrain Model (Lidar-Derived) available on the Ontario GeoHub website (0.5m Resolution).

3. Site and Subsurface Conditions

3.1 Site Description

The subject Site is currently occupied by an existing commercial building and paved parking lots. The Site is bound to the west by Wharncliffe Road South with commercial development beyond. To the south, the Site is a part of adjacent agricultural land. To the east, the Site is bound by a commercial business and residential development. There is an old concrete pad at the central section of the Site, west of the wetland. In general, the Site is relatively flat with a grade difference of about 3.5 m across the borehole locations.

Additionally, a wetland/pond area was identified and located towards the center of the investigation area. The size of the standing water area and depth of standing water will increase and decrease depending on the time of year and rain events. Further information regarding the wetland will be available within the Hydrogeological Assessment report.

The following sections provide a summary of the soil and groundwater conditions.

3.2 Soil Stratigraphy

The detailed stratigraphy encountered in each test hole 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 drilling. These boundaries are intended to reflect transition zones for geotechnical design and should not be interpreted as exact planes of geological change.

3.2.1 Topsoil

Each borehole was surfaced with a layer of topsoil that is about 120 to 620 mm thick.

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

It should be noted that topsoil quantities should not be established from the information provided at the test hole locations only. If required, a more detailed analysis (involving additional shallow test pits) is recommended to accurately quantify the amount of topsoil to be removed for construction purposes.

3.2.2 Fill

Beneath the topsoil, a layer of clayey silt fill was encountered and extended to depths ranging from about 1.5 to 3.1 m below existing grade, corresponding to a range in Elevation of 267.8 to 271.2 m. No fill was encountered within Boreholes BH5 and BH7.

The fill material was generally noted to be brown/grey and contains trace topsoil, trace sand and gravel, with some wet layering, and is soft to stiff (based on SPT N Values ranging between 3 and 10 blows). Based on laboratory testing, the *in situ* moisture content of the fill material ranges between 13 and 29 percent, generally indicative of moist conditions.

3.2.3 Clayey Silt

Underlying the topsoil and fill material, clayey silt was encountered. In general, the clayey silt is brown, becoming grey with depth, with some trace sand and gravel, and is stiff to very stiff, based on SPT N Values ranging between 11 and 20 blows. Based on laboratory testing, the *in situ* moisture content of the clayey silt ranges between 14 and 23 percent, generally indicative of moist conditions.

3.2.4 Glacial Till

With the exception of Borehole BH8, each borehole was terminated in a stratum of glacial till. The till predominantly consists of grey clayey silt. The till contains trace sand and gravel and is typically stiff to very stiff in consistency (typical SPT N Values of 10 to 30). The *in situ* moisture content of the till is 11 to 22 percent, indicating moist conditions.

Table 1 – Grain Size Summary

Borehole	Elevation (m)	Gravel	Sand	Silt	Clay
BH1/MW SA5	~268.2	12%	11%	57%	30%
BH6/MW SA3	~267.4	1%	11%	57%	31%
BH7/MW SA5	~264.6	1%	5%	55%	39%
BH8/MWB SA1	~265.3	2%	9%	52%	37%

3.2.5 Sandy Silt

Borehole BH8 was terminated in a stratum of sandy silt below the glacial till, encountered at 7.9 m bgs. The sandy silt was generally noted to be grey and compact, based on a SPT N Value of 25 blows. The *in situ* moisture content of the sandy silt was 21 percent, indicating wet conditions.

3.3 Groundwater Conditions

To allow for long-term monitoring of the water levels, monitoring wells were installed in BH1/MW, BH6/MW, BH7/MW and BH8/MW. Of these, a nested well set was installed in BH8A/MW and BH8B/MW, with details shown on the Borehole Logs. The wells were installed to depths ranging between about 2.7 and 9.1 m below ground surface (bgs). The summary of well construction details and measured groundwater levels are presented in the tables below.

Table 2 – Monitoring Well Construction Details

Borehole ID	Completion Depth (m bgs)	Screen Length (m)	Ground Surface Elevation (m ASML)	Bottom of Well Elevation (m AMSL)	Screened Strata
BH1/MW	6.1	1.5	272.75	266.52	Clayey Silt Till
BH6/MW	6.1	1.5	269.74	263.66	Clayey Silt Till
BH7/MW	6.1	1.5	269.19	262.79	Clayey Silt Till
BH8A/MW	7.6	1.5	269.90	261.94	Clayey Silt Till
BH8B/MW	5.5	1.5	269.90	264.3	Clayey Silt Till

Table 3 – Monitoring Well and Groundwater Depth and Elevation

Borehole ID	Ground Surface Elevation (m)	Groundwater Depth Below Existing Grade (m) Feb 23, 2023	Groundwater Elevation (m) Feb 23, 2023
BH1/MW	272.75	5.97	266.78
BH6/MW	269.74	0.97	268.77
BH7/MW	269.19	5.85	263.34
BH8A/MW	269.90	5.37	264.53
BH8B/MW	269.90	0.64	269.26

Note: Groundwater levels may not reflect static conditions and groundwater monitoring is on-going as part of the Hydrogeological Assessment.

The monitoring wells have been registered with the Ministry of the Environment, Conservation and Parks (MECP), in accordance with Ontario Regulation 903, and remain intact for the purposes of ongoing monitoring of stabilized groundwater conditions, as required.

Based on the ongoing monitoring of the observation wells, the groundwater levels range from relative Elevation 263.34 to 269.26 m. Further information regarding groundwater conditions will be available within the Hydrogeological Assessment report.

Details of the groundwater conditions observed within the test holes advanced as part of the geotechnical investigation are provided on the attached borehole logs. Upon completion of drilling, the open boreholes were examined for the presence of groundwater and groundwater seepage. Borehole BH5 was dry at completion.

It is noted that insufficient time was available for the measurement of the depth to the stabilized groundwater table prior to backfilling the test holes without monitoring wells installed.

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.

3.4 Methane Gas

No methane gas producing materials or significant organic matter was encountered at the borehole locations, except a thin veneer of topsoil in some of the boreholes.

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 significant methane gas concentration was detected in the boreholes.

4. Discussion and Recommendations

Based on available project drawings, the proposed project will consist of 54 units (proposed stacked townhomes) and a stormwater management facility. The residential development is expected to have complete municipal servicing and will have paved access roads.

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

4.1 Site Preparation

Prior to the placement of foundations, pipe bedding and/or engineered fill, 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. The surficial topsoil may be stockpiled on site for possible reuse for landscaping fill. Any existing wells onsite must be properly decommissioned by a licensed well contractor to protect the aquifer from surface contamination, prevent vertical movement of water between aquifers, or between an aquifer and the ground surface, and eliminate a potential safety hazard. The well decommissioning requirements and standards from Ontario Regulation 903 must be adhered to. EXP can provide this service upon request.

The present condition of the area would require that all surface water, topsoil, trees, organics, cat tails and otherwise unsuitable fill materials to be removed from the pond area prior to construction and/or placement of engineered fill. It should be anticipated that the soil underlying the existing pond, and within the immediate vicinity of the existing pond, is presently in a wet to saturated condition and considered unsuitable for placement of any engineered fill.

As mentioned previously, the pond area should be cleared and the subgrade properly prepared and approved, by a Geotechnical Engineer, prior to fill placement to restore grades in the ponded area. Soil and sediment sampling and testing from the pond area may be required to classify the material for disposal purposes.

Following the removal of the topsoil and unsuitable materials described above and prior to fill placement, the exposed subgrade should be inspected by a Geotechnical Engineer. Any loose or soft zones noted in the inspection should be over-excavated and replaced with approved fill.

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 grades will be raised, the fill material should consist of imported granular or approved onsite (excavated) material. The fill material should be inspected and approved by a Geotechnical Engineer and should be placed in maximum 300 mm (12 inch) thick loose 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 inorganic 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 also consist of on-site excavated soils, or imported granular fill approved by an Engineer. The fill should be placed in maximum 300 mm (12 inch) thick loose lifts and uniformly compacted to 95/98 percent SPMDD, depending on depth, within 3 percent of optimum moisture content 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.

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 Ministry of Environment, Conservation and Parks (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.

4.2 Excavation and Groundwater Control

4.2.1 Excess Soil Management

It should be noted that the Geotechnical Investigation does not include any testing for off-site disposal according to the new Regulation O. Reg. 406/19.

Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) was implemented on January 1, 2021. The new regulation dictates the testing protocol that is required for the management and disposal of Excess Soils. As set forth in the Regulation, specific analytical testing protocols will need to be implemented and followed based on the quality and quantity of soil to be managed. The testing protocols were fully implemented on January 1, 2022, however, effective March 14, 2022, the MECP Policy Branch confirmed the delay of the implementation phase of the regulation. It is understood, however, that effective January 1, 2023, the pause will be lifted and the regulation will resume in full implementation.

The quality of soils is assessed through an Assessment of Past Uses (APU) including the provision of an Ecolog ERIS data base report to determine if there are any Areas of Potential Environmental Concern (APEC). The parameters to be tested will be determined by the APU results.

The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

Soil sampling requirements for Areas of Potential Environmental Concern (APEC) related to the new standard effective January 1, 2022 are provided below.

Table 4 – Ex-Situ (e.g., Stockpiles)

Soil Volume	Sampling Frequency
<130 m^3	
>130 - 220 m^3	4
>220 - 5000 m^3	
>5000 m^3	N = 32 + (Volume – 5000) / 300

*refer to stockpile sampling frequency in O.Reg. 153/04 for specifics. Essentially, one sample for every 150 m^3 after 800 m^3

Table 5 – *In Situ*

Soil Volume	Sampling Frequency
<600 m^3	
>600 m^3 - 10,000 m^3	1 sample per every additional 200 m^3
>10,000 m^3 – 40,000 m^3	
>40,000 m^3	1 sample per every additional 2000 m^3

In areas where no APECs have been identified, the sampling frequency in the tables noted above, do not need to be followed and can be determined at the discretion of the QP.

In addition to the above tables, one field duplicate should be submitted for approximately every 10 samples taken for quality control/quality assurance purposes.

Soil Analytical Testing Requirements:

- Samples to be tested for a minimum of Petroleum Hydrocarbons (PHCs) – Fractions F1-F4, Benzene, Toluene, Ethylbenzene & Xylenes (BTEX), Metals & Inorganics, including Electrical Conductivity (EC) and Sodium Absorption Ration (SAR).
- Any additional potential Contaminant of Concern identified in past uses report (comes into effect January 1, 2022)
- mSPLP Leachate testing (metals and VOCs) (not required for volumes under 350 m^3: between 350 m^3 and 600 m^3 (minimum of 3); greater than 600 m^3 (10 % of samples).

Other components of the new regulation include:

- the Sampling and Analysis Plan (SAP) which follows the APU
- the Soil Characterization Report (SCR) which follows the sampling program
- the Excess Soil Destination Assessment Report (ESDAR) which follows the SCR
- Notice of Project on the Resource Productivity and Recovery Authority (RPRA) which is usually the responsibility of the Contractor during the construction phase
- Tracking Requirements on the RPRA, again, usually the responsibility of the Contractor during the construction phase

4.2.2 Excavations

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. Based on the results of the geotechnical investigation and in accordance with Section 226 of Ontario Regulation 213/91, the very stiff clayey silt till is classified as Type 2 soils, while the fill, clayey silt, sandy silt and stiff till are classified as Type 3 soils.

Temporary excavation sidewalls which extend through and terminate solely 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 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. Should groundwater egress loosen the side slopes of Type 2 or Type 3 soils, slopes of 3H:1V or flatter will be required.

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.

4.2.3 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 silty 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 or as required for space restrictions, 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;

γ = 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 (K_o) 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.

4.2.4 Construction Dewatering

Wet sand and silt lenses were noted within the upper clayey silt and lower clayey silt till deposits at the location of several of the boreholes. Based on the soil texture encountered during the investigation, minor groundwater infiltration may be anticipated within the building and service trench excavations depending on the depth of excavation and stratigraphy encountered.

In areas where excavations extend through and terminate in clayey silt and glacial till soils, only minor groundwater infiltration is expected and 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.

The variance in water levels is attributed to the various wet sand and silt lenses noted within the glacial till deposits at the location of several of the boreholes, causing water to perch at different elevations. Depending on the final grades, a dewatering plan may be required to control the water ingress. Ongoing liaison from this office will be required.

For excavations extending below the groundwater table in sandy soils, suitable groundwater control measures will be required to maintain a dry and stable excavation base and sides.

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.

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 liters per day, an Environmental Activity and Sector Registry (EASR) will be required. Permit to take Water (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.

4.3 Foundations

4.3.1 Conventional Strip and Spread Footings

The low-rise residential buildings can be supported on conventional spread and strip footings founded below the topsoil or unsuitable soils on the natural competent subgrade soils, or on approved engineered fill. Based on the borehole findings, conventional foundations on the natural subgrade or engineered fill are practical for the Site.

Bearing pressures and approximate elevations are provided in the table below for the design of strip and spread footings founded on the natural soils at the borehole locations.

Table 6 – Recommended Bearing Levels at the BH Locations

Test Location	Ground Surface Elevation (m)	Minimum Depth (m bgs)	Elevation (m)	Bearing Resistance at Serviceability Limit States SLS (kPa)	Factored Bearing Resistance at Ultimate Limit State ULS (kPa)
	272.7	1.6	271.1	145	215
BH2	272.4	2.3	270.1	145	215
BH3	269.9	2.3	267.6	145	215
BH4	270.8	3.1	267.7	145	215
BH5	269.8	1.5	268.3	145	215

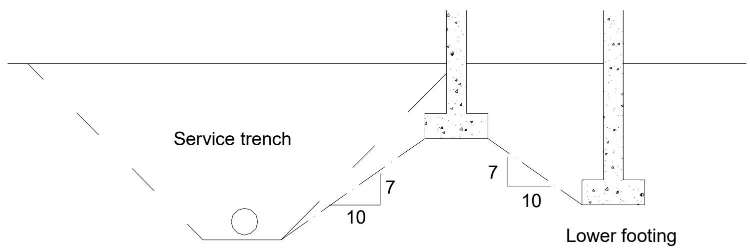
The depths to the competent natural subgrade soils for some boreholes are deeper than others because of the presence of the loose and/or unsuitable materials.

If the grades are to be raised or restored, engineered fill can be used for foundation support. If the thickness of fill is more than 3 m, imported granular soil should be considered, especially if the foundation is constructed immediately after the engineered fill placement. 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. The engineered fill construction should be monitored on a full-time basis by qualified geotechnical personnel to examine and approve fill materials, to evaluate placement operations, and to verify that the specified degree of compaction is being achieved uniformly throughout the fill.

4.3.2 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.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

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.

4.4 Basements

If the development includes buildings with basements, the basement floors can be constructed using cast slab- on- grade techniques provided that the subgrade is stripped of all topsoil, fill, and other obviously objectionable material. The subgrade should then be proof-rolled thoroughly. Any soft zones detected should be dug out and replaced with compactable excavated material placed in accordance with the requirements outlined in the previous Section 5.1.

A 200 mm (8 inch) compacted layer of 19 mm (¾ inch) clear stone should be placed between the prepared subgrade and the floor slabs to serve as a moisture barrier. An alternative option would be to place 300 mm of OPSS Granular 'A' material compacted to 100 percent SPMDD.

The installation and requirement of a vapour barrier under the floor slab, where applicable, should conform to the flooring manufacturer's and designer's requirements. Moisture emission testing is recommended to determine the concrete condition prior to flooring installation.

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 a depth h:
K = earth pressure coefficient, assumed to be 0.4;
γ = unit weight of backfill, a value of 20.4 kN/m^3 may be assumed;
h = depth to point of interest in m and,
q = equivalent value of any surcharge on the ground surface.

If basements are planned, installation of perimeter drains is required. 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 3**.

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 4** 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. An alternative option would be to place 300 mm of OPSS Granular 'A' material compacted to 100 percent SPMDD.

The installation and requirement of a vapour 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 20 MPa/m for the compacted stone layer over the compacted granular subbase.

The water-to-cement ratio and slump of concrete used 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 Foundation Backfill

In general, the existing natural soils excavated from the foundation area should be suitable for re-use as foundation wall backfill exterior to the foundation wall drainage system 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.

4.7 Site Servicing

The subgrade soils beneath the water and sewer pipes which will service the Site are generally expected to consist of clayey silt and glacial till. For services constructed on these natural soils or on 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.

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.

The bases of excavations which cut into and terminate in competent natural soils are expected to remain stable for the short construction period. For bases terminated in wet silty layers, localized improvement will be required. Base improvement may also be required if work is carried out in wet weather seasons. The extent of base improvement or stabilization is best determined in the field during construction, with consultation from a Geotechnical Engineer.

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 metre 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 OPS requirements. A summary of the general recommendations for trench backfill is presented on **Drawings 4** and **5**. 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 natural soil 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.

4.8 Seismic Design Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design using the OBC 2020 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 2020. The subsoils at the Site generally consist of topsoil underlain by layers of clayey silt, sandy silt, and glacial till deposits. It is anticipated that the proposed structures will be founded on the natural deposits, below any loose or soft zones.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2020 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 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 2020. Additional depth drilling or geophysical methods may be advised to determine if the soil conditions below the current depth of exploration can support a higher Site Classification.

4.9 Stormwater Management (SWM) Pond Construction

It is understood that a potential location for the Stormwater Management Facility is in the area of BH7/MW. Borehole BH7/MW was drilled to a depth of 6.6 m bgs and based on the results of the investigation, it is anticipated that clayey silt soils will be encountered through the upper 1.5 m with clayey silt till below this depth and to at least 6.6 m bgs. No information on the depth of the pond has been provided at the time of writing this report.

If the entire pond is constructed within the upper 6.6 m, the subsurface is expected to consist of clayey silt, which should yield sufficiently low permeability characteristics for water retention purposes and should not require a liner. However, during excavation of the pond, attention should be paid to the presence of any localized wet sand seams or silt layering. Where encountered, the localized seams should be sub-excavated an additional 0.3 m and replaced with properly compacted clay (impervious) soils. The excavated clayey silt is generally considered suitable for use as liner material.

Prior to the installation of the liner in any localized sub-excavated areas, the base and/or side slopes of the excavations that terminate in the clayey silt should be scoured and thoroughly proof-rolled. This exercise is carried out in attempts to seal the water producing sand and silt seams naturally occurring in the clayey silt stratum and protect the integrity of the SWM facility.

Where sub-excavation is required, the clay liner should be placed in lifts not exceeding 200 mm in thickness and compacted to a Standard Proctor Maximum Dry Density (SPMDD) of 95 percent within 3 percent on the wet side of the optimum moisture content. Sheepsfoot rollers should be used to compact the liner to reduce the permeability of the clay material. Careful subgrade preparation and stringent control of the clay material and the compaction are required. The finished surface of the natural clayey silt and/or clay fill liner is normally hand rolled and to prevent the development of shrinkage cracks it should be kept moist until the pond is filled. It must also be protected from erosion and/or scouring action of the pond waves.

Sediment build-up will need to be removed from the base of the forebay at regular intervals. It is therefore recommended that a 200 mm Granular 'A' layer be placed over the base and compacted to 100% SPMDD. Following the granular placement, a turf-stone mat should be placed and cover the entire base of the forebay.

It is recommended that the SWM facility slopes be constructed with an inclination of about 3.0 horizontal to 1 vertical or flatter. If the soil is subject to erosion of inundation from water, then the slopes should be lined with concrete or riprap.

Where required, the riprap material should consist of sound limestone, free of inclusions. The limestone should be blasted or crushed, with an average size of 150 to 200 mm. When the source of the riprap is known, then EXP should be notified so that a site visit may be conducted at the quarry, to verify the source and quality of the material.

The slopes of the entire detention facility, after shaping, should be lightly scarified and a 150 mm thick layer of organic topsoil should be placed on the surface to assist in establishing grass-type vegetation which will inhibit erosion. A synthetic erosion blanket can be considered to assist the growth of vegetation. Some routine maintenance of the slope surfaces will likely be required to address minor long-term weathering and erosion.

During the construction of the SWM Pond, it is recommended that inspection and *in situ* density testing be conducted as well as soil sampling, laboratory testing and monitoring of fill placement. Full-time geotechnical supervision is recommended.

If other alternatives are considered and our assumptions are not valid, EXP should be requested to review the proposed system for applicable geotechnical considerations.

4.10 Inlet/Outlet Structures

In the vicinity of the proposed inlet/outlet structures, culverts and/or pipes should be carefully backfilled with excavated clayey silt soils. No bearing problems are anticipated for flexible or rigid pipes founded on the natural deposits or compacted on site soils. The backfill should be in intimate contact with the complete circumference of the pipe.

Any headwall should be embedded sufficiently to permit minimum liner thickness up to the headwall inlet. A vibratory hoe-pack or approved device will likely be required to compact the clayey soils around the headwall. In places where proper compaction may be difficult to achieve, lean concrete backfill should be used.

The support for inlet and outlet structures must be derived from the natural soils or engineered fill. An allowable bearing pressure of 145 kPa (3000 psf) is available in these soils. Any headwalls should be backfilled using free-draining granular material and may be designed using an active earth pressure coefficient of 0.4 and a unit weight of 20.4 kN/m^3. Any footing must be protected with a minimum of 1.2 m (4 ft) of earth cover or equivalent insulation to provide protection against potential frost damage.

If minor grade changes must be accommodated for the footings of the headwall, the levels can be raised by the placement of lean mix concrete on the natural subgrade soils.

During the construction of the SWM Pond and associated infrastructure, it is recommended that inspection and *in situ* density testing be conducted as well as soil sampling, laboratory testing and monitoring of fill placement. Full-time geotechnical supervision is recommended.

4.11 Site Pavement Design

Areas to be paved should be stripped of all topsoil, organics and other obviously unsuitable material. The exposed subgrade must then be thoroughly proof-rolled. Any soft areas 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 that the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated specified classification and subgrade conditions.

Table 7 – Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Local Road
Asphaltic Concrete	92% MRD[1] or 97% BRD[1]	40 mm HL-3 50 mm HL-8
Granular 'A' (Base)	100% SPMDD[1]	150 mm
Granular 'B' (Sub-Base)	100% SPMDD[1]	300 mm

*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. 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' aggregate should be checked for conformance to OPSS 1010 prior to use on Site, and during construction. The Granular 'B' sub-base 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 MUNI 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 6**. 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.

4.12 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 Municipal Standards.

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.

4.13 Methane Gas Testing

No methane gas producing materials or significant organic matter was encountered at the borehole locations, except a thin veneer of topsoil in some of the boreholes.

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 significant methane gas was detected in any of the boreholes.

Based on the present information, no special methane gas abatement measures are indicated at this Site.

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 following removal of existing (if any), fill and organics, prior to foundation installation and engineered fill placement (if required);
- 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;
- Visual examination of concrete reinforcing steel 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 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. 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 test pit 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.

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We trust that this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.