Geotechnical & Hydrogeological Investigation

1732435 Ontario Ltd.

Project Name:
Watson Farm Development
21829 Nissouri Road
Thorndale, Ontario

Project Number:
LON-00017870-GE

Prepared By:
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1. Introduction and Background

1.1 Introduction

EXP Services Inc. (EXP) was retained by **1732435 Ontario Ltd.** to carry out a geotechnical and hydrogeological investigation and prepare a report relating to the proposed development at 21829 Nissouri Road in Thorndale, Ontario, hereinafter referred to as the 'Site'.

Based on a preliminary concept plan provided by Development Engineering (London) Ltd., it is understood that the development will consist of multi-unit condominiums/townhouses. In this regard, the residential subdivision is expected to have complete municipal servicing and will be accessed with paved local roads. Several infiltration galleries are proposed at various locations on the property.

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 by EXP and others, EXP has provided geotechnical engineering guidelines to support the proposed Site development.

1.2 Terms of Reference

The investigation was generally completed in accordance with the scope of work outlined through EXP's Proposal P20-039 dated February 18th, 2020. Authorization to proceed with this investigation was received from Mrs. Jane Elliot of 1732435 Ontario Ltd**.** through email correspondence.

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 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 advanced at the site, EXP Services Inc. has provided engineering guidelines for the geotechnical design and construction of the proposed development. More specifically, this report provides comments on site preparation, excavations, dewatering, foundations, slab-on-grade and basement construction, site servicing, trenchless installation options, low impact development, seismic design considerations, pavement recommendations, and impact assessment.

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 and hydrogeological analyses, or if any questions arise concerning geotechnical and hydrogeological 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 E** of this report, which contains further information necessary for the proper interpretation and use of this report.

2. Methodology

The fieldwork was carried out on March 24th, 2020. In general, the geotechnical and hydrogeological investigation consisted of the advancement of six (6) boreholes at the locations denoted on **Drawing 1** as BH1 to BH6. MW was suffixed to the borehole identification where monitoring wells were installed.

Prior to the 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 to depths of 5.0 m to 6.6 m below ground surface (bgs) utilizing a track-mounted drill rig equipped with continuous flight 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 and stabilized groundwater levels in the monitoring wells are recorded in the borehole logs found in **Appendix A**. Following the drilling, the boreholes without monitoring wells 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 four (4) grain size analyses with results presented in **Appendix B** and routine moisture content determinations, 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 the final version of this report. After this time, they will be discarded unless prior arrangements have been made for longer storage.

The boreholes and monitoring wells were installed for the purpose of providing insight on potential impacts of development on local water users, and how groundwater conditions may impact the progress of construction activities such as excavations for basement construction and site servicing. Ground surface elevations at each test hole location were surveyed to the top of spindle of hydrant at the southeast corner of Nissouri Road and Elliot Trail (Benchmark #2). The benchmark has a geodetic elevation of 287.094 m.

A twelve (12) month groundwater level monitoring program was requested for the three (3) monitoring wells. The monitoring began on March 26, 2020 to determine the seasonal high groundwater elevation.

3. Site Description and Geologic Setting

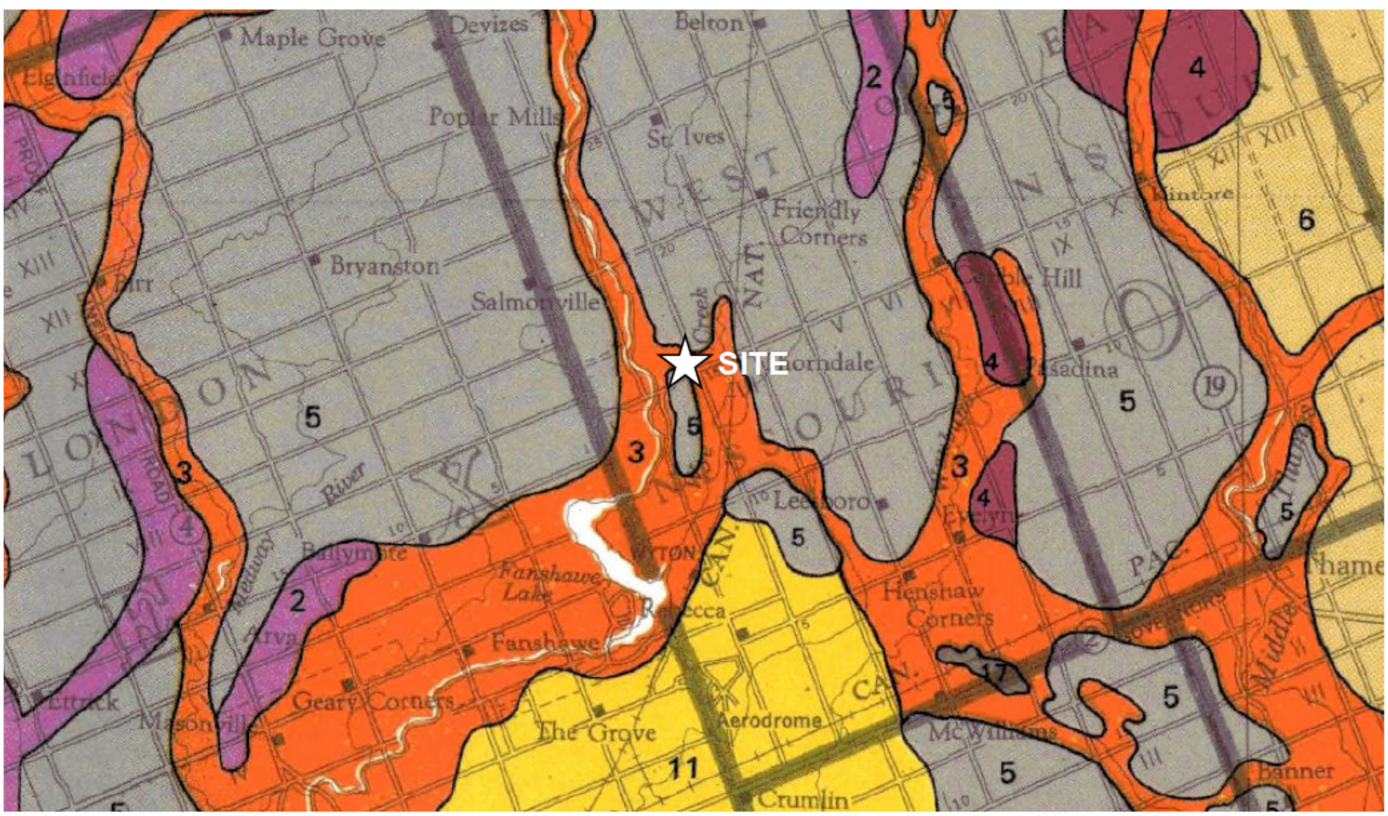
3.1 Site Description

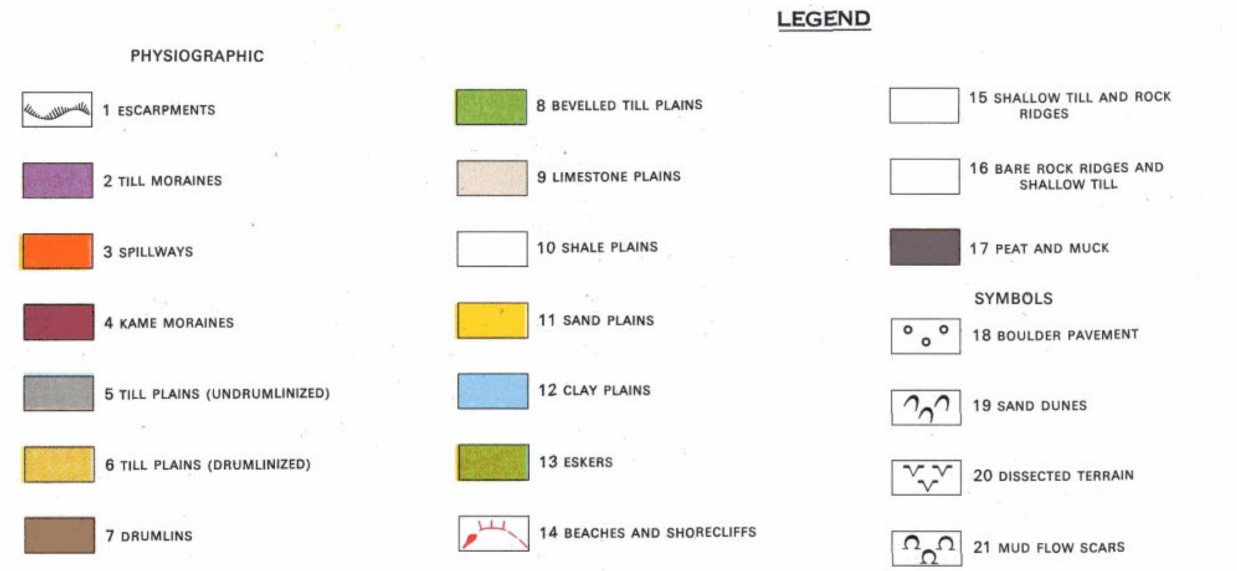
The subject area is currently in use as an agricultural field with a garage and driveway on the east side. The Site is generally bounded by agricultural fields to the north and west, Thorndale Road to the south, and Nissouri Road to the east. Residential housing is present on the southeast Site boundary.

3.2 Site Physiography, Quaternary and Surficial Geology

The physiography of Southwestern Ontario was altered significantly by the glacial and interglacial periods that took place throughout the Quaternary period. The overburden deposits which are present in the study area were formed by numerous glacial events during the late Wisconsinan glacial stage approximately 10,000 to 23,000 years before present. There were two distinct glacial lobes present in Southwestern Ontario during this period. The Huron Lobe advanced from Lake Huron southwards, and the Erie Lobe advanced from the northeast, receding to the east.

During the advancement of the glacial ice sheets, bedrock and unconsolidated sediments were eroded. During the recession of the glaciers, the eroded materials were deposited in lakes, rivers and along spillways, contributing to the present configuration of moraines, abandoned spillways, drumlins, eskers, abandoned shorelines, and various still-water sediment deposits.





The surficial deposits were mapped and categorized into a number of physiographic regions by Chapman and Putnam (1984). The physiographic regional mapping for the area indicates that the site is situated within the Stratford Till Plain (Chapman and Putnam, 1984).

Review of physiographic landform mapping, above, indicates that the Site is located within spillways. Quaternary geology mapping indicates the Site is located in an area characterized by Tavistock Till: sandy silt to silt matrix, silty clay matrix in the south and north.

Surficial geology mapping shows the surficial Site soils to be predominantly gravelly glaciofluvial deposits with sandy silt to silty sand textured till in the northeastern border (**Drawing 2**). Fluvial terraces are mapped to the north and south of the Site and extend into the site as shown in **Drawing 2**.

3.3 Bedrock Geology

The Site is underlain by limestone, dolostone and shale of the Dundee Formation (OGS, 2011). This formation consists of 60 to 160 feet (18 to 49 m) of light brown, medium-grained with some minor chert (Hewitt, 1972), and is part of the Algonquin Arch, which forms a ridge along the southwestern Ontario peninsula between the Michigan Basin (to the northwest) and the Appalachian Basin (to the southwest). Bedrock is generally not exposed in the area.

Review of bedrock topography mapping (OGS, 1978) indicates the bedrock surface at an elevation in the range of 259 m to 267 m (850 to 875 feet) above mean sea level (amsl). The bedrock surface generally slopes to the southwest in this area. Review of Ministry of Environment, Conservation and Parks (MECP) Water Well Records (WWR) for the area indicates 16 wells within 500 m of the Site intersect bedrock at depths of approximately 19 m to 27 m below ground surface (bgs). Based on ground surface elevations detailed in the MECP WWR, this equates to a bedrock elevation of about 258 to 270 m, which is generally consistent with the bedrock topography mapping. Bedrock was not encountered during the drilling work completed as part of this investigation.

4. Site and Subsurface Conditions

4.1 Soil Stratigraphy

The detailed stratigraphy encountered in the boreholes 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 test hole 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.

4.1.1 Topsoil

Topsoil was observed at the surface of each borehole. The topsoil ranged in thickness from 250 mm to 300 mm.

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.

4.1.2 Clayey Silt/Sandy Silt

Beneath the topsoil and extending to between 0.6 m and 1.2 m below ground surface (bgs) in Boreholes BH2, BH3 and BH6 was a layer of clayey silt/sandy silt.

The clayey silt was generally described as brown, weathered with trace sand and moist (based on tactile examination).

The sandy silt in Borehole BH6 was brown in colour, weathered, and contained some clay. It was loose in compactness (based on a Standard Penetration Test (SPT) N Value of 8 blows per 300 mm split spoon sampler penetration) and very moist (based on tactile examination and an *in situ* moisture content of 17 percent).

4.1.3 Sand and Gravel

Underlying the topsoil or clayey silt/sandy silt in each borehole was a stratum of sand and gravel. The brown sand and gravel extended to depths of between 1.7 m and 5.8 m bgs. Borehole BH3 was terminated in the layer. In general, the sand and gravel contained trace to some silt and was compact to very dense (SPT N Values of 17 to 60). Laboratory testing of the sand and gravel yielded *in situ* moisture contents of 4 to 20 percent, indicative of moist to wet conditions. Cobbles were encountered in the sand and gravel layer in Boreholes BH2, BH3 and BH4/MW near 1.0 m depth.

Three (3) grain size analyses were carried out on recovered samples of the sand and gravel, with results presented in **Appendix B**.

4.1.4 Sand

At varying depths and thicknesses in Boreholes BH2 and BH3 was a layer of sand. The sand was brown in colour, fine to medium grained, and contained trace to some silt and occasionally trace gravel. It was loose to compact in relative density (SPT N Values of 9 to 18) and wet (tactile examination and *in situ* moisture contents of 22 to 25 percent).

One (1) grain size analysis of the sand was carried out with results presented in **Appendix B**.

4.1.5 Glacial Till

With the exception of Borehole BH3, each borehole was terminated in a stratum of glacial till. The till varied in composition between silt, clayey silt, and sandy silt. The till was generally brown becoming grey in colour with depth and contained trace to some gravel. It was compact/very stiff to very dense/hard (SPT N Values of 17 blows per 300 mm to greater than 50 blows per 150 mm split spoon sampler penetration) and typically moist (tactile examination and typical *in situ* moisture contents of 7 to 18 percent). The sandy silt till near the interface with the wet sand in Boreholes BH2 and BH6 was described as very moist to wet.

Possible cobbles and boulders were encountered within the till in Borehole BH6 near 5.2 m depth.

4.2 Groundwater Conditions

Three (3) monitoring wells were installed during the drilling on March 24th, 2020. The wells were advanced to depths of between 2.4 m and 5.8 m bgs. The summary of well construction details is presented in the table below.

Table 1 – Monitoring Well Construction Details

Well ID	Ground Surface Elevation (m)	Completion Depth (m bgs)	Screen Length (m)
BH1/MW	285.96	5.79	1.52
BH4/MW	284.03	2.44	1.52
BH5/MW	284.60	3.35	1.52

The stabilized groundwater levels in the monitoring wells were generally measured monthly on 10 occasions between March and November 2020. A summary table of the stabilized groundwater levels is attached in **Appendix C**.

The EXP 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. The measurements provided in **Appendix C** indicate a variation in the shallow overburden groundwater table between elevations 283.12 m and 279.09 m over the monitored period. The general direction of groundwater flow is to the south and is generally expected to be influenced by the Thames River and a fluvial terrace nearby the Site (See **Drawing 2**). Further interpretation will be carried out as ongoing water level measurements are collected.

Details of the groundwater conditions observed within the test holes advanced as part of the geotechnical and hydrogeological 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. Groundwater was measured near depths of 1.8 m and 3.0 m bgs in Boreholes BH6 and BH3 upon completion of drilling, respectively. Borehole BH2 had caved in about the level of groundwater seepage following the drilling.

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.

4.3 Potable Groundwater

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 2 – Summary of MECP Well Records

Well ID	Well Type	Date Completed	Depth (m)	Water Use	Water Status	Screened Lithology	Water Found at Depth (m)	Static Water Level (m)
4104066	Overburden	4-Oct-64	4.6	Domestic	Water Supply	Gravel	---	3.7
4104067	Bedrock	2-Mar-59	20.4	Domestic	Water Supply	Rock	19.8	4.3
4104114	Bedrock	12-Sep-61	32.0	Domestic	Water Supply	Limestone	19.2	2.1
4104123	Bedrock	5-Nov-59	34.1	Domestic	Water Supply	Limestone	25.9	6.1
4104126	Bedrock	19-Nov-65	24.4	Livestock	Water Supply	Limestone	24.1	4.9
4104136	Bedrock	6-Dec-60	39.9	Domestic	Water Supply	Rock	39.6	3.0
4105503	Overburden	14-Aug-71	11.0	Domestic	Water Supply	Sand	6.7	6.7
4107158	Bedrock	2-Apr-75	24.1	Industrial	Water Supply	Limestone	24.1	1.8
4107278	Bedrock	30-Jul-75	47.5	Industrial	Water Supply	Limestone	47.5	10.1
4108825	Bedrock	11-May-79	29.3	Domestic	Water Supply	Limestone	28.0	5.5
4108826	Bedrock	5-May-79	39.6	Domestic	Water Supply	Limestone	38.4	8.2
4109219	Bedrock	26-May-80	20.7	Domestic	Water Supply	Limestone	20.7	3.7
4109814	Bedrock	3-Nov-82	22.3	Commercial	Water Supply	Limestone	22.3	2.4
4110016	Bedrock	26-Mar-84	21.3	Domestic	Water Supply	Limestone	21.3	4.0
4111677	Bedrock	16-Jun-89	34.4	Domestic	Water Supply	Limestone	34.4	4.6
4114153	Overburden	30-Jun-99	10.7	Domestic	Water Supply	Gravel	9.1	5.5
4114695	Bedrock	18-Apr-01	25.9	Domestic	Water Supply	Limestone	25.9	11.3
4114788	Bedrock	15-Oct-01	48.8	Domestic	Water Supply	Limestone	33.5	12.8
4115854	Bedrock	22-Nov-04	23.1	Domestic	Water Supply	Limestone	23.0	5.4
7114507	---	30-Sep-08	10.1	Abandoned	---	---	---	---
7201840	---	13-May-13	29.3	Abandoned	---	---	---	---
7233087	---	2-Dec-14	39.9	Abandoned	---	---	---	---
7281174	---	16-Jan-17	---	---	---	---	---	---

The potable wells are typically set into bedrock aquifers. The bedrock was generally encountered at depths of 19 m to 27 m below existing grade. Two wells (MECP Well No. 4104066 and 4105503) were found to be drawing from a shallow aquifer, while MECP Well No. 4114153 was installed in an intermediate confined gravel aquifer. Overburden soils noted in the MECP WWR were generally described as clay with intermediate and deep sand layers. Some wells noted unconfined shallow sand and gravel layers.

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.

4.4 Methane Gas

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

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.

5. Discussion and Recommendations

Based on a preliminary information provided by Development Engineering (London) Ltd., it is understood that the development will consist of low-density residential housing. In this regard, the residential subdivision is expected to have complete municipal servicing and will be accessed with paved local roads.

The following sections of this report provide geotechnical comments and recommendations regarding site preparation, excavations, dewatering, foundations, slab-on-grade and basement construction, site servicing, trenchless installation options, low impact development, seismic design considerations, pavement recommendations, and impact assessment.

5.1 Site Preparation

Prior to placement foundations and/or engineered fill, all surficial topsoil, vegetation and/or otherwise deleterious materials should be stripped. It is anticipated that the surficial topsoil may be stockpiled on site for possible reuse as landscaping fill.

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 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, 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 3**.

The natural and inorganic fill 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 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 in order to provide adequate stability for the new pavements. Refer to **Drawings 6** and **7** for specified compaction levels.

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 utilized 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.

5.2 Excavations and Groundwater Control

5.2.1 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. Based on the results of the geotechnical investigation and in accordance with Section 226 of Ontario Regulation 213/91, the very stiff to hard clayey silt till and dense to very dense sandy silt/silt till soils encountered at the site are classified as Type 2 soils, while the clayey silt/sandy silt, sand/sand and gravel, and compact sandy silt/silt till soils 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 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.

5.2.2 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, 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/m3 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.

5.2.3 Construction Dewatering

As noted in **Appendix C**, groundwater was measured between 1.47 m and 5.11 m below ground surface (bgs) (Elevations 283.12 m and 279.09 m) in the monitoring wells over the monitored period. For excavations extending below the groundwater table, suitable groundwater control measures will be required to maintain a dry and stable excavation base and sides. Based on the results of the current investigation, significant groundwater infiltration is generally anticipated for excavations that extend below an elevation of about 282.5 m. Depending on the timing of construction works, groundwater may be encountered at shallower depths.

To ensure the stability of excavations, it is recommended that the base of any excavations on Site be set a minimum of 0.5 m higher than the above-mentioned elevations. If the above recommendation is followed, it is expected that any minor groundwater infiltration can be accommodated using conventional sump pumping techniques. In the event groundwater infiltration persists, positive groundwater control may be required.

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 liters 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.

5.3 Foundations

5.3.1 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 approved engineered fill.

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

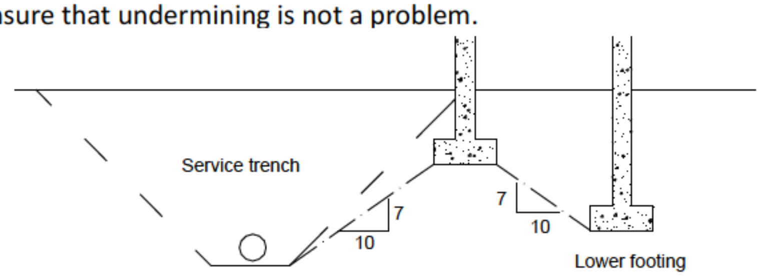
Bearing Resistance at Serviceability Limit States (SLS)	145 kPa (3,000 psf)
Factored Bearing Resistance at Ultimate Limit States (ULS)	215 kPa (4,500 psf)

As discussed in Section 5.2.3, groundwater was measured between 1.47 m and 5.11 m bgs (Elevation 283.12 m to 279.09 m) in the monitoring wells over the monitored period. It is recommended that the footing depths of any permanent structures be founded at a maximum depth of 0.6 m above the groundwater table.

If the 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 3**, 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.

5.3.2 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.

5.4 Basements

If the development includes buildings with basements, the basement floors can be constructed using cast slab-on-grade techniques provided the subgrade is stripped of all topsoil 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 slab to serve as a moisture barrier. A less desirable, 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 4**. Due to the presence of groundwater in the sand/sand and gravel soils and expected fluctuations in the level of the groundwater table, water proofing measures will be required to minimize the ingress of water seepage. An underfloor drainage system will be required for all buildings with basements planned at the Site. If the founding level is 0.6 m above the high groundwater level, water proofing measures may not be necessary.

5.5 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 loose 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 5** 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. A less desirable, 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 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.

5.6 Foundation Backfill

In general, the existing natural 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.

5.7 Site Servicing

The subgrade soils beneath the water and sewer pipes which will service the site are generally expected to comprise sand and gravel, sand, and glacial till. For services constructed on the natural soils or engineered fill, the bedding should conform to 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. Localized improvement may be required in areas where wet silty soils are present, and 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.

Groundwater was measured between 1.47 m and 5.11 m bgs in the monitoring wells over the monitored period. If excavation penetrates below these levels, positive groundwater control and base stabilization will be required. Ongoing liaison from this office will be required.

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 loose 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 **Drawing 6** and **7**. 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 soils 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.

5.8 Trenchless Installation

5.8.1 General

It is understood that trenchless pipe installation is being considered for the sewer and watermain installation under Nissouri Road at the intersection with Thorndale Road. The construction under Nissouri Road may require a trenchless technique to minimize disturbance to daily traffic volumes and maintain existing conditions of the roadway.

If trenchless pipe installation is considered, it is anticipated that the invert levels for the crossing will be set about 2 to 4 m below the existing road surface which would be within moist to wet sand/sand and gravel or moist glacial till deposits. It should be noted that the boreholes advanced as part of this investigation were away from the intersection of Nissouri Road and Thorndale Road. In this regard, the soil stratigraphy and groundwater conditions may vary from those encountered on Site and should be confirmed through test pit advancement and witnessed by prospective contractors prior to the bidding phase.

The work should be carried out by a specialist contractor and should be continually monitored for levelness and alignment. If the work is carried out properly, and no significant obstructions are encountered, only nominal settlement would be expected in the overlying soils.

The use of a liner will assist in controlling minor groundwater seepage and will provide stability during the installation process.

The liner should be structurally designed to resist the jacking load as well as any loads imposed by the overlying soil and/or traffic. The liner should advance before the auger to prevent loss of ground.

Excavation recommendations for the proposed entry and receiving pits were provided in Section 5.2.

The municipality will have to be notified and all applicable requirements in regards to settlement and monitoring of the road surface will have to be stipulated, if warranted. Further review of the applicable requirements should be undertaken with the specialist tunnelling contractor at the time of construction.

A general description of the possible trenchless methods is presented in the following paragraphs.

5.8.2 Pipe Ramming

Pipe ramming installation is analogous to driving an open-ended tube pile directed horizontally. Impact force from a percussive hammer is used to advance a conduit pipe from an entry pit to a receiving pit. During the advancement, most of the soil being penetrated fills the conduit rather than being excavated. The rammed conduit is advanced to a receiving pit at which point the soil contained in the pipe is removed. The vibrations and cyclic forces may make this option not feasible depending on the invert level of the service.

Minimal groundwater control should be needed along the installation path because the soil within the pipe is not removed until after the crossing has been completed. The retained soil will tend to act as a plug, reducing the potential for ground water seepage and running of soil into the pipe.

Reference is given to the excavation section of this report for recommendations pertaining to the construction of entry and receiving pits. Groundwater was measured between 1.47 m and 5.11 m below ground surface (bgs)

(Elevations 283.12 m and 279.09 m) in the monitoring wells over the monitored period. Dewatering measures may be required at the entry and receiving pits, depending on the soil and groundwater conditions.

5.8.3 Jack and Bore

Jack and bore typically involves the simultaneous advancement of continuous flight auger and conduit pipe. The auger is used to excavate soil from within the conduit pipe and transport cuttings back to the receiving pit where they are removed. Rotary power to auger and pushing force is provided by a drill rig located within a jacking pit.

Jack and bore installation(s) should be conducted in accordance with OPSS 416, Construction Specifications for Pipe Line and Utility Installation by Jacking and Boring.

As stated above in Section 5.8.2, groundwater was measured between 1.47 m and 5.11 m below ground surface (bgs) (Elevations 283.12 m and 279.09 m) in the monitoring wells over the monitored period. Dewatering measures may be required at the entry and receiving pits.

5.8.4 Directional Drilling

The typical technique for installing pipe by directional drilling uses a surface mounted drill rig that launches drill rods creating a string of rods (called the drill string) below the ground guided by a drill head that is directionally controlled by the drill operator. The direction of the drill is along a pre-determined path (called the drill path) based upon the above ground and below ground, pre-construction investigations of the site. A locating device is used during the drill to track the location of the drill head so that the operator may make adjustments as necessary.

A small diameter pilot hole is drilled from the entrance point (typically in a sending pit) to the desired exit point (receiving pit). It is necessary to use a drilling fluid during drilling to lubricate and protect the pipe, and to maintain the size of the hole being opened. Following the exit of the pilot drill bit, the hole is then enlarged by the use of a back reamer attached to the end of the drill string which is pulled back through the pilot hole. As the back reaming takes place, the pipe being installed is also pulled into the hole.

Directional drilling may require the use of a casing pipe to protect the carrier pipe where required and/or dictated by an outside agency. As mentioned in the previous sections, the groundwater conditions would apply.

5.8.5 Installation Methods Comparison

The following table summarizes some of the possible alternatives for the pipe installation:

Table 3 – Installation Methods Comparison

Installation Method	Advantages	Disadvantages
Pipe Ramming	• Well suited for steel pipes and casings over distances usually up to 30m (150 ft.) up long and up to 1,500mm (60-inches) in diameter • Suitable for a wide variety of soil conditions (soils containing cobbles and boulders) • Minimal groundwater control required along the installation route • Can be driven at almost any angle • Accommodates obstructions well • No traffic interruption or requirement for detour route • In some soil conditions, little surface settlement can be anticipated	• Installation utilizes pneumatic percussive blows to advance the pipe, creating a lot of noise • Pipe can be difficult to steer/direct • Requires staging pits • Groundwater control is required for staging pits • Vibration from ramming operation can consolidate soils around the pipe • Slower than other techniques • Potential for ground heave
Jack and Bore	• Handles wide variety of ground conditions • Very accurate (slopes of 0.2% easily achieved) • Relatively common use in Ontario • No traffic interruption or requirement for detour route • suitable for steel pipes up to 1.8 m in diameter	• Requires large area for jacking shaft and support equipment • Requires short and long term settlement monitoring • Fluid to support annular space • Groundwater control is required
Directional Drilling	• Handles wide variety of ground conditions • Steerable both horizontally and vertically to maintain and adjust alignment • Does not require staging pits • Minimal groundwater control required • Alignment can be adjusted to avoid obstruction • Ramped drilling • Small settlement, if fluid well controlled • No traffic interruption or requirement for detour route	• Potential for inadvertent drilling returns • Requires drilling fluid to maintain the bore which could allow subsidence • Site grades may require longer bore or staging pits • Annular space filling (fluid or grouting) • Large diameter boring (> 24 inch.) not practical through cohesionless soil conditions.

5.9 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.

Based on the information collected at the borehole locations, and the above cited criteria, the near surface soils encountered at some of the test hole locations have potential for use in LID stormwater management design. The following table summarizes the elevations where the upper surface of the LID material was encountered and the elevation of the underlying impervious soil.

Table 4 – Low Impact Development Potential

Borehole No.	Ground Elevation (m)	Elevation of Top of LID Material (m)	Elevation of Underlying Impervious Soil (m)	Highest Measured Groundwater Elevation (m)	Total Depth of LID Material (m)
BH1/MW	284.20	283.90	278.41	280.63	5.49
BH2	284.35	283.74	278.10	-	5.64
BH3	283.57	282.96	278.54[2]	-	4.42[2]
BH4/MW	284.03	283.73	282.35	282.55	1.38
BH5/MW	284.60	284.30	282.47	283.12	1.83
BH6	283.70	282.48	280.50	-	1.98

Notes: 1. Thickness of LID material available for design is typically taken as 1 m above the impervious strata or seasonal high groundwater table.
2. Borehole BH3 was terminated in the sand and gravel (LID material). Actual depth to impervious stratum may be greater.

Four (4) grain size analyses were carried out samples of the natural sand/sand and gravel soils obtained from the boreholes advanced at the site. The gradations are generally representative of the LID soils available at the site. The results are provided in **Appendix B**.

For consideration in design, based on the grain size distribution, the estimated hydraulic conductivity (K) of the natural sand was approximately 4.6×10^{-3} cm/s, while the sand and gravel ranged from 7.3×10^{-4} to 5.8×10^{-3} cm/s. This corresponds with estimated infiltration rates of 130 mm/hr in the sand and 80 mm/hr to 135 mm/hr for the sand and gravel.

It is understood that recommended factors of safety will be applied to the estimated parameters cited above for use in design. Further discussion regarding LID techniques and opportunities is provided in Section 6.3.

5.10 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 topsoil, clayey silt/sandy silt, sand and gravel/sand over 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 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 advanced to a maximum depth of 6.6 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.

5.11 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 the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated specified classification and anticipated subgrade conditions.

Table 5 – Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Local Road	Arterial Road (Thornedale Road/Nissouri Road Tie-In)
Asphaltic Concrete	92% MRD[1] or 97% BRD[1]	40 mm HL-3 50 mm HL-8	50 mm HL-3 80 mm HL-8
Granular 'A' (Base)	100% SPMDD[1]	150 mm	150 mm
Granular 'B' (Sub-base)	100% SPMDD[1]	300 mm	450 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.

It is noted that an extensive sand and gravel unit was noted in each of the boreholes advanced at the site. Depending on final site grading, the existing sand and gravel may be able to be used in place of the Granular 'B' sub-base material, provided that sieve testing is carried out at the intervals detailed in **Appendix D** and meeting the requirements of OPSS 1010.

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 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 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 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.

Additional comments on the construction of roadways are as follows:

1. The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavourable weather.

2. It is recommended that EXP be retained to review the final pavement structure designs and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

5.12 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 and OPSS 1350 Specifications.

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.

5.13 Methane Gas Testing

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

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.

5.14 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 the removal of existing services (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 and visual examination of concrete reinforcing steel placement in footings set on engineered fill;
- 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.

6. Hydrogeological Comments

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

6.1 Potable Wells

The Ministry of Environment, Conservation and Parks (MECP) Water Well Records (WWR) for this area are summarized in Section 4.3. The stratigraphy encountered in the wells generally consists of an extensive clay unit separated by intermediate and deep sand layers, with an unconfined shallow sand and gravel layers in some of the wells. Groundwater information provided by MECP WWR indicates that groundwater in the area is typically sourced from bedrock aquifers (19 m to 27 m depth). MECP Well No. 4104066 and 4105503 were found to be drawing from the shallow aquifer and MECP Well No. 4114153 was installed in an intermediate confined gravel aquifer.

Shallow groundwater flow across the Site is typically affected by the soil permeability, topography and drainage. Deep aquifers are significantly less affected by surface conditions. The extensive clay strata noted in the MECP WWR will effectively limit both the vertical and horizontal zone of influence impacting the intermediate, deep and bedrock wells due to the low permeability of the soils. No significant long-term impact to these wells is anticipated either quantitatively or qualitatively since the inverts of excavations are not expected to be deep enough to penetrate into the underlying aquifers. Any temporary dewatering operations which may be required to deal with groundwater seepage from the overburden soils are not expected to cause any long-term impacts to the intermediate to deep aquifers supplying the water supply wells near the Site.

However, as mentioned above, two (2) MECP WWR within 500 m of the Site were noted to be drawing from a shallow sand and gravel aquifer. It is possible that construction and potential dewatering activities on Site may temporarily influence the shallow groundwater regime in the area. In this regard, it is recommended that a door-to-door water well survey be carried out to determine in any potable wells in the area are still in use.

6.2 Construction Dewatering Impacts

At the time of reporting, no firm development plan was available for the Site. Based on the test hole information for the Site, the proposed development is generally expected to be set within the sand, sand and gravel, and glacial till strata.

Based on the results of the current investigation, no significant short- or long-term impact is anticipated on the nearby intermediate to deep wells, either quantitatively and qualitatively since the typical inverts of residential excavations (generally 2 m to 5 m below grade) are typically not deep enough to penetrate into the underlying intermediate or deep aquifers. Any temporary dewatering operations which may be required are not expected to cause any long-term impacts to the aquifers which supply the nearby potable intermediate, deep, and bedrock wells.

As stated in Section 6.1, two (2) potable wells noted in the MECP WWR were set into a shallow overburden sand and gravel aquifer. A door-to-door water well survey within 500 m of the Site should be carried out to determine if any shallow potable wells in the area are still active. Further discussion in this regard will be provided at a later time.

6.3 Secondary Infiltration Opportunities

Due to the increased impermeable surfaces (such as roof-tops, roadways, sidewalks), the proposed development is expected to result in a reduction in the post-development infiltration level, and a corresponding increase in the estimated run-off. The use of secondary infiltration opportunities is recommended to reduce the variation between pre-development and post-development conditions.

Mitigative measures that could be considered may include reducing the amount of impervious surface areas, which is not always practical to implement on an effective scale. The shallow sand/sand and gravel soils observed in the study area are conducive to shallow groundwater recharge by infiltrated surface water.

For residential developments, some examples of on-site stormwater management practices include:

- Routing pavement runoff to grassed areas;
- Planting of trees and bushes;
- Installing pervious pavement;
- Installing soakaway areas;
- Infiltrating rooftop runoff onto grassed areas;
- Implementing rainwater harvesting (i.e. to re-use in toilet flushing and irrigation, etc.);
- Installing green roof technologies;
- Using filters/bio-retention (i.e. islands, parking areas, etc.);
- Installing absorbent landscaping; and,
- Installing oil/grit separators.

In terms of maintaining infiltration rates in post-development, the most effective stormwater management practices include installing infiltration trenches, lot grading, roof leader discharge to soakaway pits/pervious areas, using pervious pipes, and installing pervious catch-basins.

It is recommended that some of these practices be utilized in site planning and design in order to mitigate the impact of increased runoff and stormwater pollution. By implementing Low Impact Development (LID) practices during development, infiltration volumes can be effectively stored and returned to the natural environment by various development technologies and methods described above.

Based on a concept plan provided by Development Engineering Ltd., it is understood that several infiltration galleries are planned as part of the development. The following text is reference from the *Low Impact Development Stormwater Management Planning and Design Guide* developed by the Credit Valley Conservation Authority (CVC) and Toronto and Region Conservation Authority (TRCA).

There are several common concerns associated with the use of soakaways, infiltration trenches and infiltration chambers:

- Risk of Groundwater Contamination: Most pollutants in urban runoff are well retained by infiltration practices and soils and therefore, have a low to moderate potential for groundwater contamination (Pitt et al., 1999). Chloride and sodium Low Impact Development Stormwater Management Planning and Design Guide Version 1.0 4-47 from de-icing salts applied to roads and parking areas during winter are not well attenuated in soil and can easily travel to shallow groundwater.

To minimize risk of groundwater contamination the following management approaches are recommended (Pitt et al., 1999; TRCA, 2009b):

- Stormwater infiltration practices should not receive runoff from high traffic areas where large amounts of de-icing salts are applied (e.g., busy highways), nor from pollution hot spots (e.g., source areas where land uses or activities have the potential to generate highly contaminated runoff such as vehicle fueling, servicing or demolition areas, outdoor storage or handling areas for hazardous materials and some heavy industry sites);
- Prioritize infiltration of runoff from source areas that are comparatively less contaminated such as roofs, low traffic roads and parking areas; and,
- Apply sedimentation pretreatment practices (e.g., oil and grit separators) before infiltration of road or parking area runoff.

In addition, a layer of geotextile fabric should be placed at the surface of the native sand/sand and gravel layer to limit the migration of fines and to provide an extra barrier from potential contaminants entering the groundwater.

The use of sedimentation pre-treatment practices before infiltration of road and parking area runoff will be imperative to ensure the quality of groundwater is maintained post-development.

7. General Comments

The information presented in this report is based on the interpretation of geotechnical and hydrogeological information provided to EXP and a limited investigation carried out by EXP designed to provide information to support an assessment of the current geotechnical and hydrogeological 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, hydrogeological 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 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 Google Earth Pro and should be read in conjunction with EXP Geotechnical Report LON-00017870-GE.

Geotechnical Investigation

Watson Farm Development

21829 Nissouri Road, Thorndale, Ontario

CLIENT 1732435 Ontario Ltd.	
TITLE Borehole Location Plan	
Prepared By: E.B.	Reviewed By: D.S.
exp	EXP Services Inc. 15701 Robin's Hill Road, London, ON, N5V 0A5

DATE	APPROXIMATE SCALE	PROJECT NO.	DWG.
APRIL 2020	1:2000	LON-00017870-GE	1

-LEGEND-

Approximate Site Boundary

7b **Glaciofluvial deposits:** river deposits and delta topset facies
7b Gravelly deposits

5b **5b Till:** Stone-poor, sandy silt to silty sand-textured till

Fluvial Terrace

Image Source: The Ontario Geological Survey. 2003. Surficial Geology of Southern Ontario.

Geotechnical Investigation

Watson Farm Development

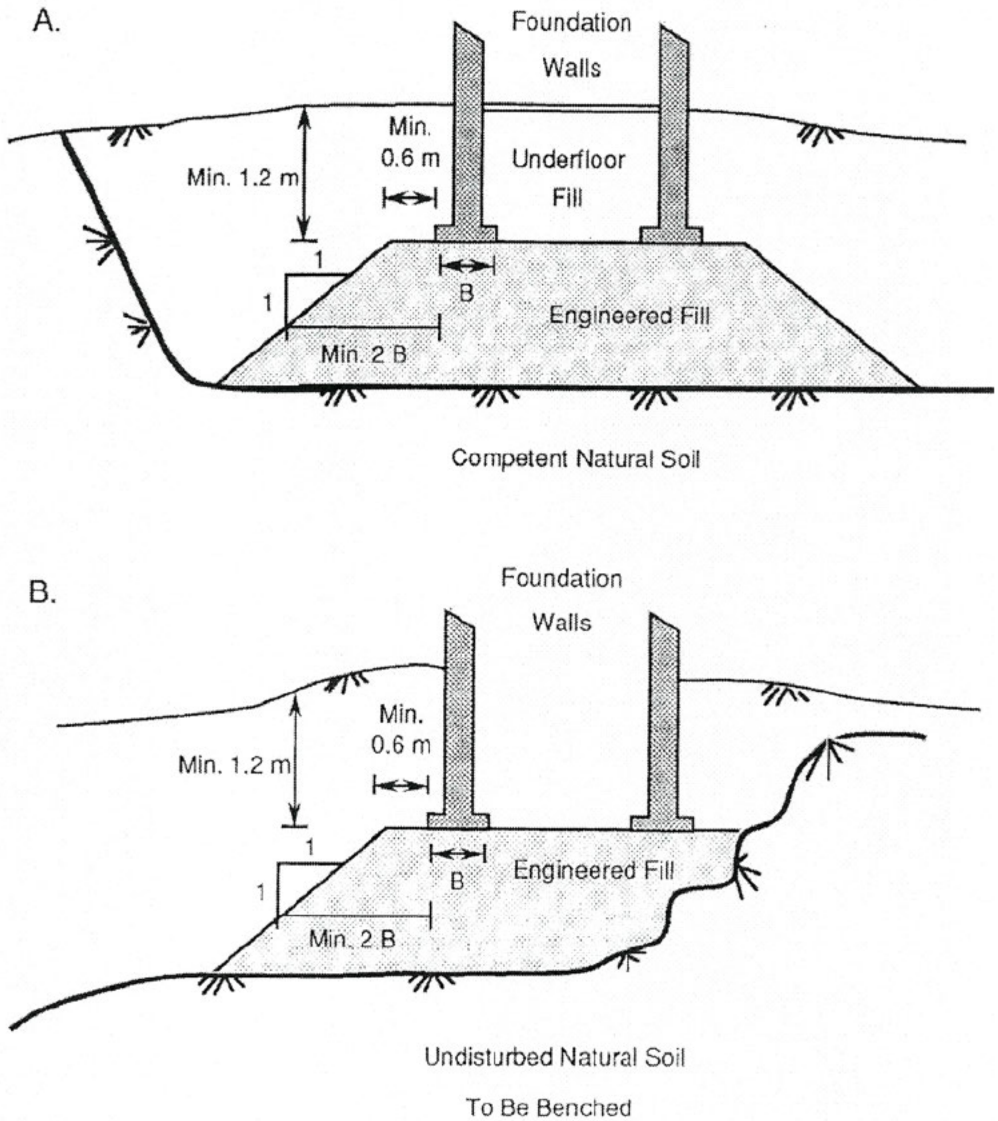
21829 Nissouri Road, Thorndale, Ontario

CLIENT 1732435 Ontario Ltd.	
TITLE Surficial Geology	
Prepared By: E.B.	Reviewed By: D.S.
exp EXP Services Inc. 15701 Robin's Hill Road, London, ON, N5V 0A5	

DATE	APPROXIMATE SCALE	PROJECT NO.	DWG.
APRIL 2020	1:5000	LON-00017870-GE	2

DRAWING 3 – GEOMETRIC REQUIREMENTS FOR FOUNDATIONS ON ENGINEERED FILL

Schematic (Not to Scale)



SECTION VIEW

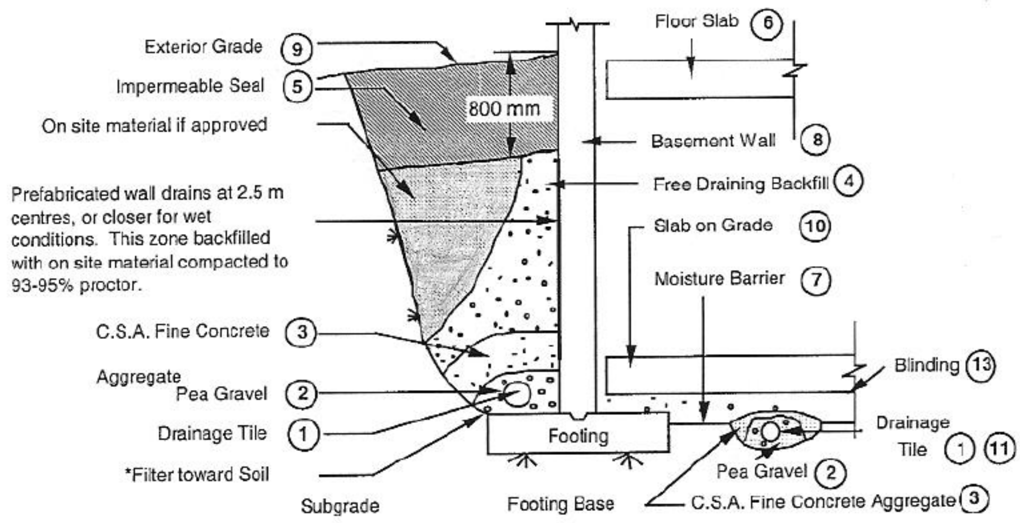
Section A – Typical Section of Slab-on-Grade Building
Section B – Typical Section of Building with Basement

Refer to Detailed Notes on following page.

NOTES FOR ENGINEERED FILL PLACMENT:

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-00017870-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.

DRAWING 4 – BACKFILL AND BASEMENT DRAINAGE DETAIL (NOT TO SCALE)



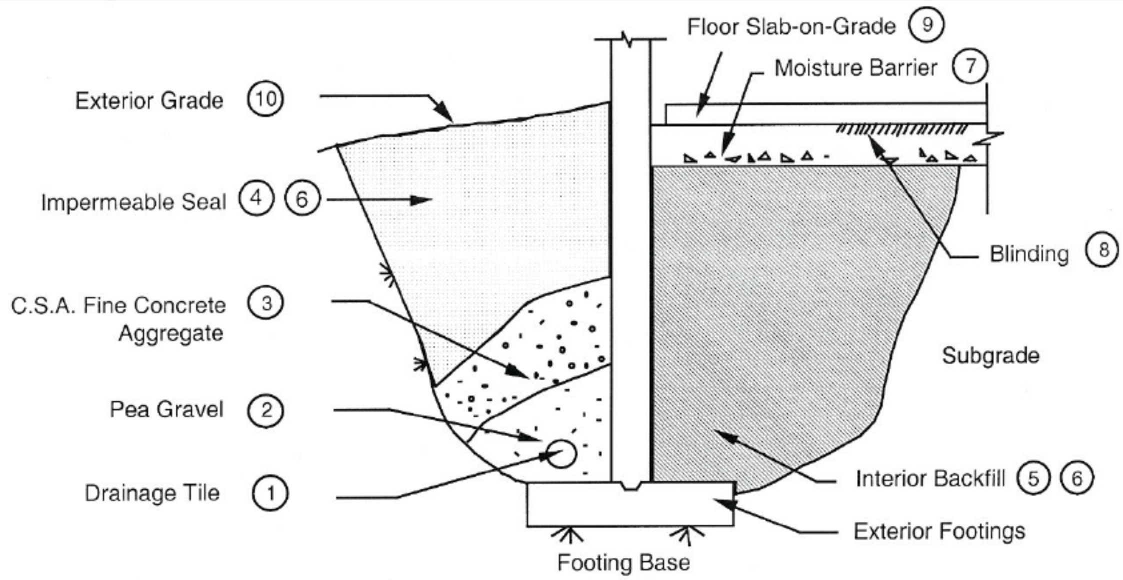
NOTES:

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).
8. 4. Free-draining backfill - OPSS Granular B or equivalent compacted to 93 to 95 (maximum) percent Standard Proctor density. Do not compact closer than l.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.

DRAWING 5 – DRAINAGE AND BACKFILL RECOMMENDATIONS (NOT TO SCALE)

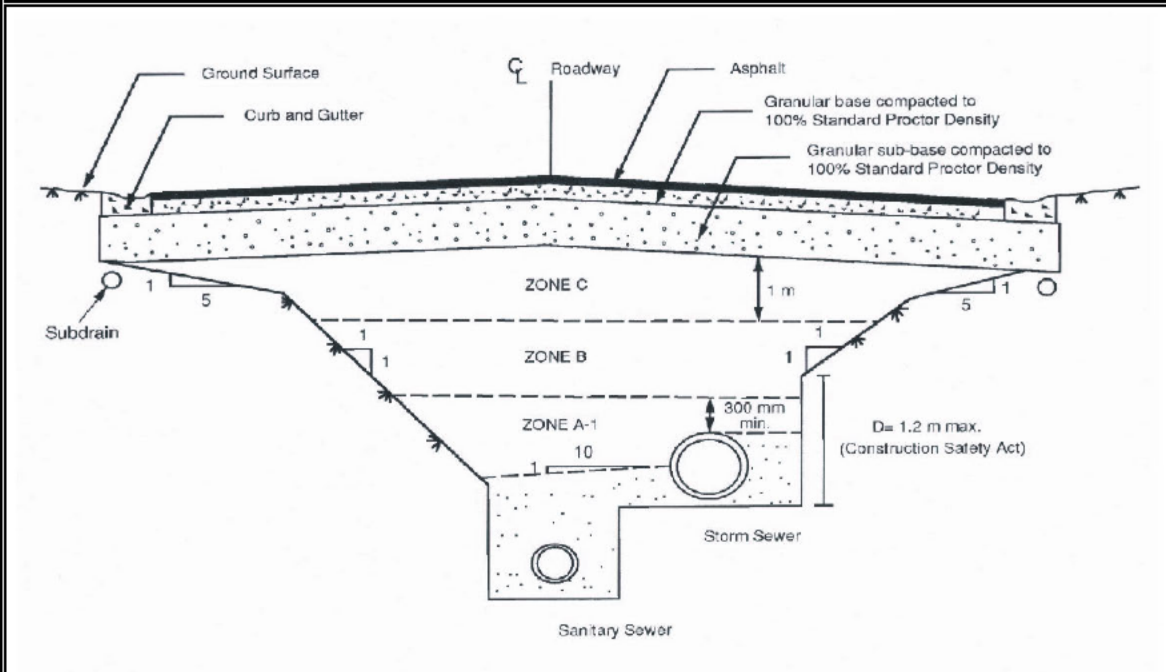


NOTES:

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.

DRAWING 6 – TYPICAL BACKFILL DETAIL
STORM AND SANITARY SEWER (COMMON TRENCH)



SECTION VIEW

NOTES:

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

ZONE A-l
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).

DRAWING 7 – TRENCH BACKFILL REQUIREMENTS

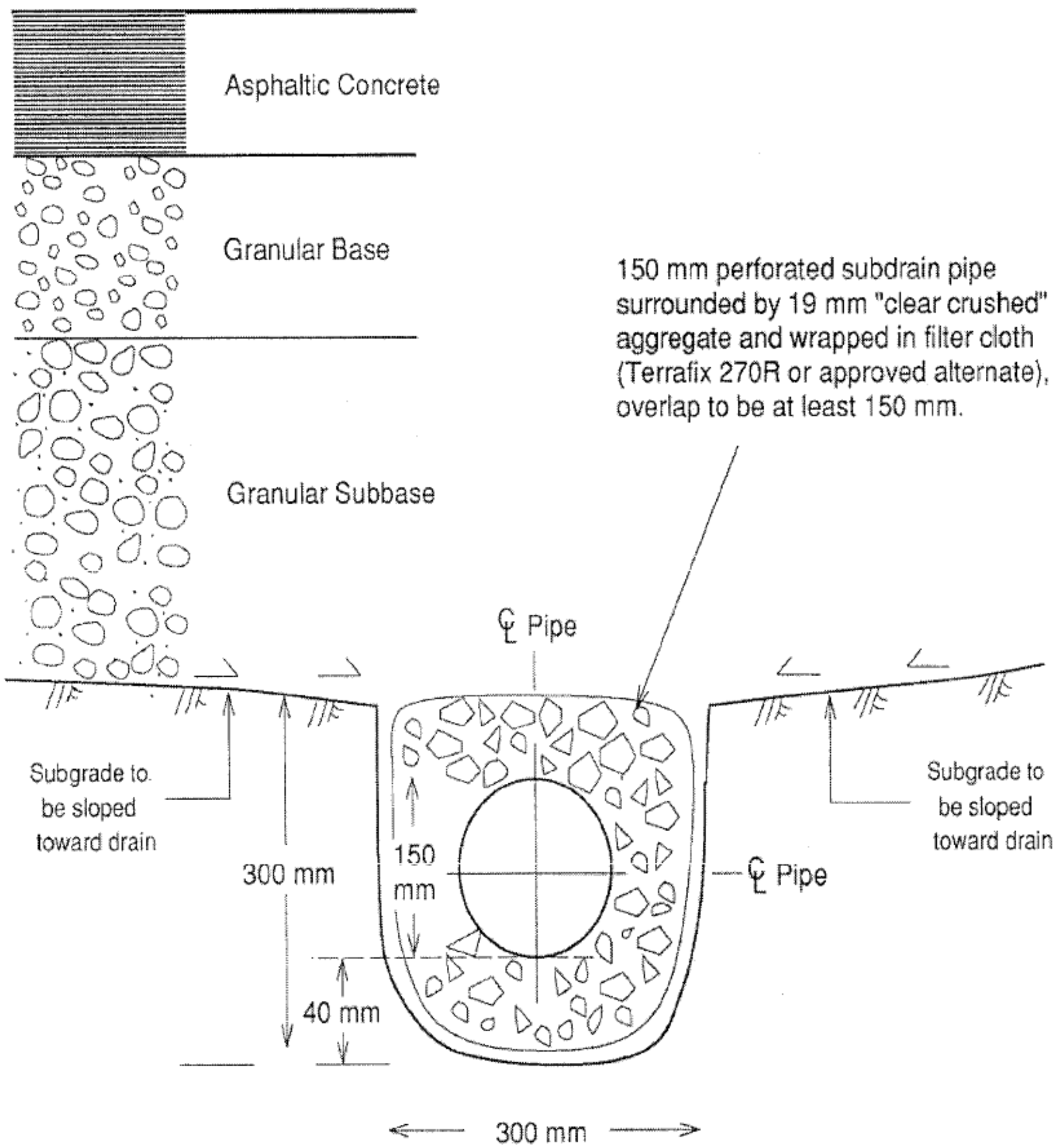
Requirements for backfill in service trenches, etc. should conform to current OPSS requirements. A summary of the general recommendations for trench backfill is presented on **Drawing 6**.

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 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.

DRAWING 8 – PAVEMENT SUBDRAIN DETAIL



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

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.

UNIFIED SOIL CLASSIFICATION	Fines (silt and clay)		Sand			Gravel		Cobbles	
			Fine	Medium	Coarse	Fine	Coarse		
M.I.T. SOIL CLASSIFICATION	Clay	Silt	Sand			Gravel			
			Fine	Medium	Coarse				
	Sieve Sizes		200	40	10	4	3/4		
	Particle Size (mm)	0.002	0.06	0.075	0.2	0.6	2.0	5.0	20	80

2. **Fill**: Where fill is designated on the test hole 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 test hole 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 test hole. Despite the use of test holes, 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 test hole 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 test hole 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 test hole 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

BH1/MW
Sheet 1 of 1

CLIENT **1732435 Ontario Ltd.** PROJECT NO. **LON-00017870-GE**
PROJECT **Watson Farm Development** DATUM **Geodetic**
LOCATION **21829 Nissouri Road, Thorndale, ON** DATES: Boring **March 24, 2020** Water Level **Apr 10/20**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	284.2	**TOPSOIL** - 300 mm					
	283.9	**SAND AND GRAVEL** - brown, trace to some silt, compact to dense, moist					
1			SS	S1	300	18	5
			SS	S2	200	30	5
2			SS	S3	250	28	4
3			SS	S4	200	29	4
		- becoming wet near 3.7 m bgs					
4			SS	S5	200	32	6
		- clayey silt layering encountered near 4.6 m bgs	SS	S6	250	30	
5	278.4						
6		**CLAYEY SILT TILL** - grey, some sand, some gravel, hard, moist	SS	S7	250	50*	11
	277.7	**End of Borehole at 6.6 m bgs.**					
7							

SHEAR STRENGTH: ✚ S Field Vane Test (#=Sensitivity); ▲ Penetrometer; ■ Torvane; 100, 200 kPa
Atterberg Limits and Moisture: W_P W W_L
● SPT N Value × Dynamic Cone: 10, 20, 30, 40

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.

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 C Consolidation
H Hydrometer CD Consolidated Drained Triaxial
S Sieve Analysis CU Consolidated Undrained Triaxial
γ Unit Weight UU Unconsolidated Undrained Triaxial
P Field Permeability UC Unconfined Compression
K Lab Permeability DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

BOREHOLE LOG

BH2
Sheet 1 of 1

CLIENT **1732435 Ontario Ltd.** PROJECT NO. **LON-00017870-GE**
PROJECT **Watson Farm Development** DATUM **Geodetic**
LOCATION **21829 Nissouri Road, Thorndale, ON** DATES: Boring **March 24, 2020** Water Level

DEPTH (m bgs)	ELEVATION (~m) 284.4	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	284.1	**TOPSOIL** - 250 mm					
	283.7	**CLAYEY SILT** - brown, weathered, trace sand, moist					
1		**SAND AND GRAVEL** - brown, trace silt, compact to dense, moist - cobble encountered near 1.0 m bgs	SS	S1	300	40	5
			SS	S2	250	42	5
2–3			SS	S3	200	28	7
3			SS	S4	300	17	4
4	280.3	**SAND** - brown, fine to medium grained, trace to some silt, compact, wet					
4–5			SS	S5	300	18	22
6	278.1	**SANDY SILT TILL** - grey, trace clay, some gravel, compact, wet	SS	S6	300	23	19
	277.8	**End of Borehole at 6.6 m bgs.**					
7							

SHEAR STRENGTH: ✚ S Field Vane Test (#=Sensitivity) ▲ Penetrometer ■ Torvane (100, 200 kPa)
Atterberg Limits and Moisture: W_P W W_L
● SPT N Value × Dynamic Cone (10, 20, 30, 40)

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) Borehole open to 2.4 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.

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
- H Hydrometer
- S Sieve Analysis
- γ Unit Weight
- P Field Permeability
- K Lab Permeability
- C Consolidation
- CD Consolidated Drained Triaxial
- CU Consolidated Undrained Triaxial
- UU Unconsolidated Undrained Triaxial
- UC Unconfined Compression
- DS Direct Shear

WATER LEVELS
- ⊻ Apparent
- ▼ Measured
- ▲ Artesian (see Notes)

BOREHOLE LOG

BH3
Sheet 1 of 1

CLIENT	**1732435 Ontario Ltd.**	PROJECT NO.	**LON-00017870-GE**
PROJECT	**Watson Farm Development**	DATUM	**Geodetic**
LOCATION	**21829 Nissouri Road, Thorndale, ON**	DATES: Boring **March 24, 2020**	Water Level

SHEAR STRENGTH: ✚ S Field Vane Test (#=Sensitivity); ▲ Penetrometer; ■ Torvane (100, 200 kPa)
Atterberg Limits and Moisture: W_P W W_L
● SPT N Value × Dynamic Cone (10, 20, 30, 40)

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	283.6	**TOPSOIL** - 300 mm					
	283.3	**CLAYEY SILT** - brown, weathered, trace sand, moist					
	283.0	**SAND AND GRAVEL** - brown, trace to some silt, compact, moist - silty in upper levels - cobble encountered near 1.0 m bgs					
1			SS	S1	150	23	8
			SS	S2	150	17	5
2	281.4	- becoming wet near 2.0 m bgs					
		SAND - brown, fine to medium grained, trace to some silt, trace gravel, loose to compact, wet	SS	S3	300	10	22
3			SS	S4	400	9	25
4	279.5	**SAND AND GRAVEL** - brown, trace silt, compact, wet					
			SS	S5	300	25	20
5	278.5	**End of Borehole at 5.0 m bgs.**					
6							
7							

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) Borehole open to 3.7 m bgs and groundwater measured near 3.0 m bgs upon completion of drilling.
3) bgs denotes below ground surface.
4) No significant methane gas concentration was detected upon completion of drilling.

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 — C Consolidation
H Hydrometer — CD Consolidated Drained Triaxial
S Sieve Analysis — CU Consolidated Undrained Triaxial
γ Unit Weight — UU Unconsolidated Undrained Triaxial
P Field Permeability — UC Unconfined Compression
K Lab Permeability — DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

BOREHOLE LOG

BH4/MW
Sheet 1 of 1

CLIENT **1732435 Ontario Ltd.** PROJECT NO. **LON-00017870-GE**
PROJECT **Watson Farm Development** DATUM **Geodetic**
LOCATION **21829 Nissouri Road, Thorndale, ON** DATES: Boring **March 24, 2020** Water Level **Apr 10/20**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	284.0	**TOPSOIL** - 300 mm					
	283.7	**SAND AND GRAVEL** - brown, trace silt, compact to very dense, moist - cobble encountered near 1.0 m bgs					
1			SS	S1	200	52	5
	282.3	- becoming wet near 1.7 m bgs					
		CLAYEY SILT TILL - brown, trace sand, trace gravel, very stiff, moist	SS	S2	300	21	7
2			SS	S3	400	17	15
3		- becoming grey near 2.9 m bgs	SS	S4	450	17	16
4	280.0	**SANDY SILT TILL** - grey, trace to some clay, trace gravel, very dense, moist					
5			SS	S5	200	50*	7
6			SS	S6	200	50*	6
	277.5	**End of Borehole at 6.6 m bgs.**					
7							

SHEAR STRENGTH
✚ S Field Vane Test (#=Sensitivity)
▲ Penetrometer ■ Torvane
100 200 kPa
Atterberg Limits and Moisture
W_P W W_L
● SPT N Value × Dynamic Cone
10 20 30 40

52

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.

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 C Consolidation
H Hydrometer CD Consolidated Drained Triaxial
S Sieve Analysis CU Consolidated Undrained Triaxial
γ Unit Weight UU Unconsolidated Undrained Triaxial
P Field Permeability UC Unconfined Compression
K Lab Permeability DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

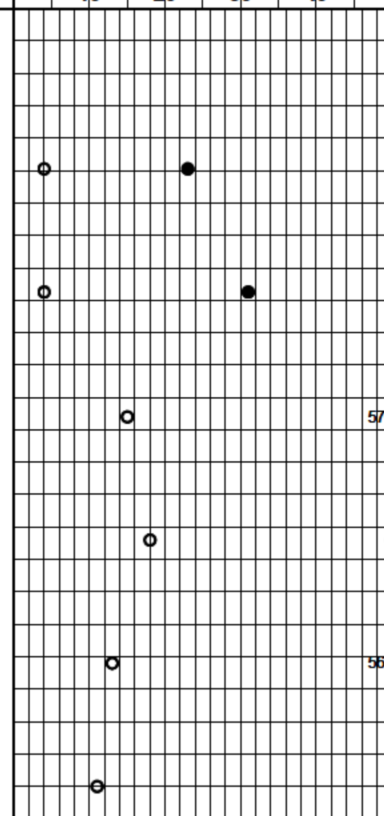
BOREHOLE LOG

BH5/MW
Sheet 1 of 1

CLIENT **1732435 Ontario Ltd.** PROJECT NO. **LON-00017870-GE**
PROJECT **Watson Farm Development** DATUM **Geodetic**
LOCATION **21829 Nissouri Road, Thorndale, ON** DATES: Boring **March 24, 2020** Water Level **Apr 10/20**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	284.6	**TOPSOIL** - 300 mm					
	284.3	**SAND AND GRAVEL** - brown, trace silt, compact to dense, moist					
1			SS	S1	200	23	4
			SS	S2	200	31	4
2	282.5	- becoming wet near 2.1 m bgs					
		SILT TILL - brown, trace to some clay, trace to some sand, trace gravel, very dense, moist	SS	S3	300	57	15
3			SS	S4	200	50*	18
		- becoming grey near 3.7 m bgs					
4			SS	S5	300	56	13
			SS	S6	300	50*	11
5	279.6	**End of Borehole at 5.0 m bgs.**					
6							
7							

SHEAR STRENGTH: ✚ S Field Vane Test (#=Sensitivity); ▲ Penetrometer; ■ Torvane; 100, 200 kPa
Atterberg Limits and Moisture: W_P W W_L
● SPT N Value × Dynamic Cone: 10 20 30 40



NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.

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
- H Hydrometer
- S Sieve Analysis
- γ Unit Weight
- P Field Permeability
- K Lab Permeability
- C Consolidation
- CD Consolidated Drained Triaxial
- CU Consolidated Undrained Triaxial
- UU Unconsolidated Undrained Triaxial
- UC Unconfined Compression
- DS Direct Shear

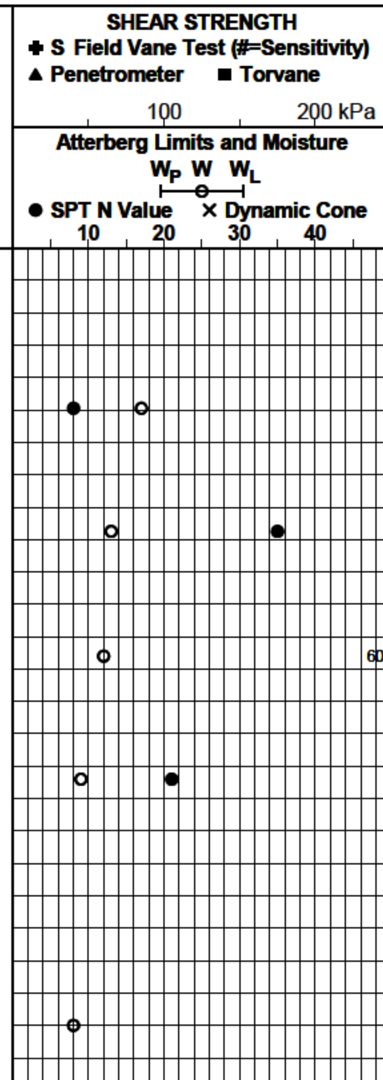
WATER LEVELS
- Apparent
- Measured
- Artesian (see Notes)

BOREHOLE LOG

BH6
Sheet 1 of 1

CLIENT **1732435 Ontario Ltd.** PROJECT NO. **LON-00017870-GE**
PROJECT **Watson Farm Development** DATUM **Geodetic**
LOCATION **21829 Nissouri Road, Thorndale, ON** DATES: Boring **March 24, 2020** Water Level

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	WELL LOG	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	283.7	**TOPSOIL** - 300 mm						
	283.4	**SANDY SILT** - brown, weathered, some clay, loose, very moist						
1				SS	S1	150	8	17
	282.5	**SAND AND GRAVEL** - brown, trace silt, dense to very dense, moist						
			∇	SS	S2	150	35	13
2		- becoming wet near 2.1 m bgs						
		- silt layering encountered near 2.5 m bgs		SS	S3	150	60	12
3	280.5	**SANDY SILT TILL** - grey, trace clay, trace gravel, very dense, moist - very moist to wet in upper 0.6 m		SS	S4	300	21	9
4								
5				SS	S5	150	50*	8
	278.5	- possible cobble/boulder encountered near 5.2 m bgs						
		End of Borehole at 5.2 m bgs due to auger refusal.						
6								
7								



NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON-00017870-GE.
2) Borehole open to 2.1 m bgs and groundwater measured near 1.8 m bgs upon completion of drilling.
3) bgs denotes below ground surface.
4) No significant methane gas concentration was detected upon completion of drilling.

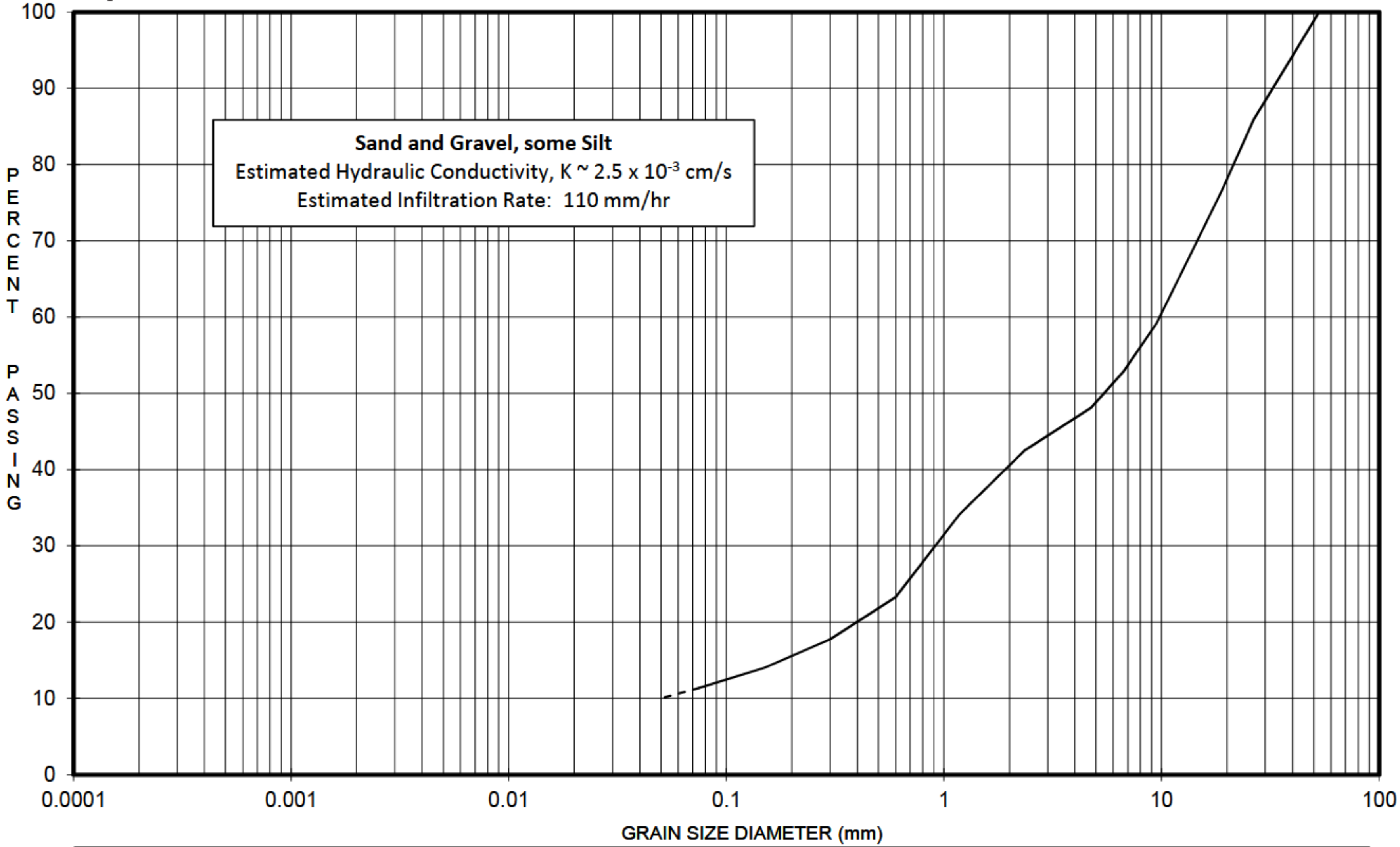
SAMPLE LEGEND
⊠ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

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

WATER LEVELS
∇ Apparent ▼ Measured ▲ Artesian (see Notes)

Appendix B – Grain Size Distribution Analyses

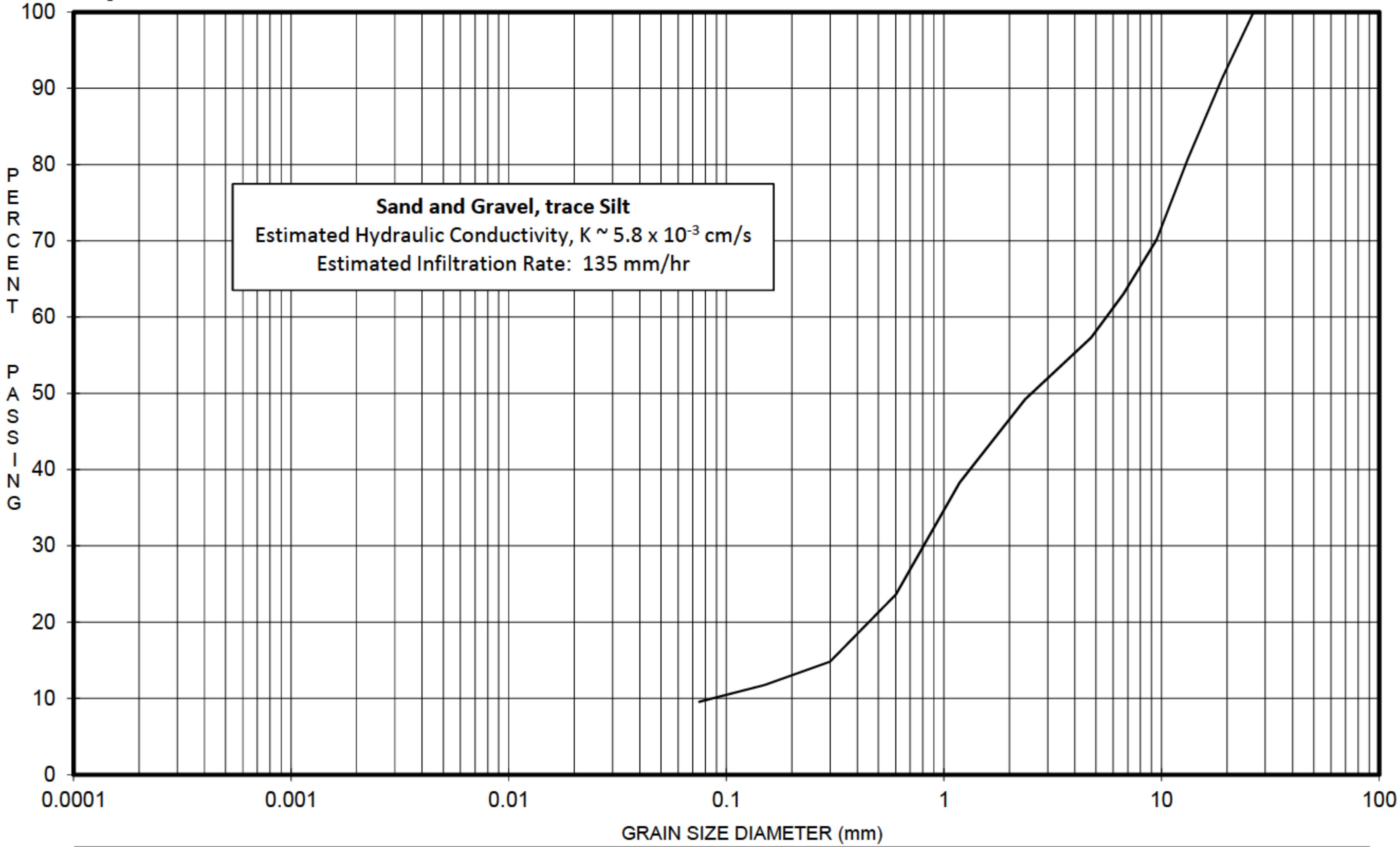
MECHANICAL GRAIN SIZE ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

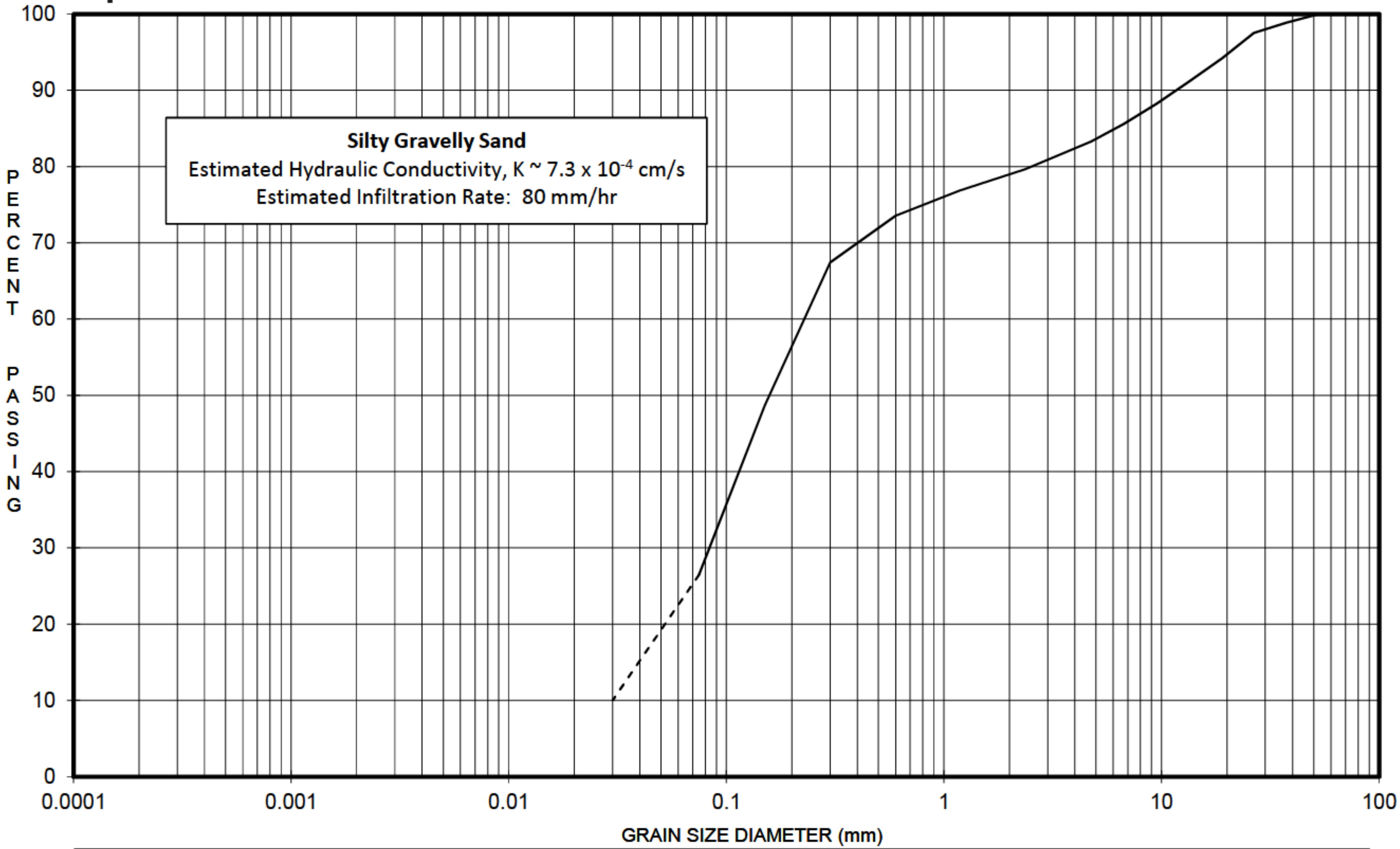
MODIFIED M.I.T. CLASSIFICATION	**Sample Description:** **Sand and Gravel (Borehole BH1, 0.3 to 0.6 m depth)**	**Watson Farm Development** **Project: LON-00017870-GE**	**Figure 1**

MECHANICAL GRAIN SIZE ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		
MODIFIED M.I.T. CLASSIFICATION	**Sample Description:** **Sand and Gravel (BH2 S2&S3, 1.5 to 3.0 m depth)**				**Watson Farm Development** **Project: LON-00017870-GE**			**Figure 2**	

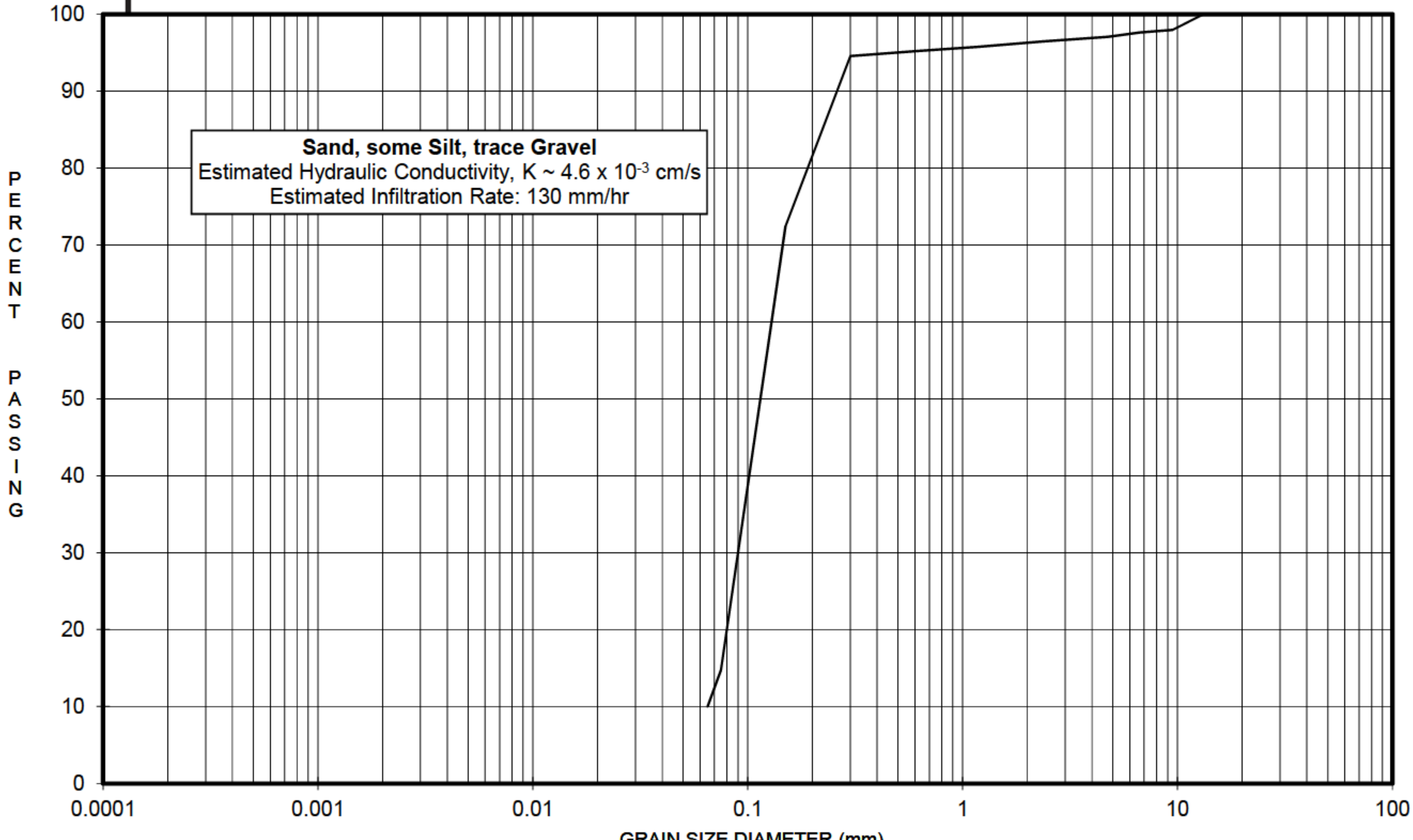
MECHANICAL GRAIN SIZE ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION		
Sample Description: **Silty Gravelly Sand (Borehole BH3, 0.6 to 0.8 m depth)**	**Watson Farm Development** **Project: LON-00017870-GE**	**Figure 3**

MECHANICAL GRAIN SIZE ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION	**Sample Description:** **Sand (BH3 S3, 2.3 to 2.7 m depth)**	**Watson Farm Development** **Project: LON-00017870-GE**	**Figure 4**

Appendix C – Stabilized Groundwater Measurements

Appendix C - Stabilized Groundwater Measurements

Well ID	Ground Surface Elevation (m)	Top of Pipe Elevation (m)	Groundwater Elevation (m)									
			26-Mar-20	10-Apr-20	15-Apr-20	19-May-20	15-Jun-20	31-Jul-20	31-Aug-20	30-Sep-20	22-Oct-20	25-Nov-20
BH1/MW	284.20	284.98	280.59	280.63	280.57	280.29	280.17	279.78	279.38	279.17	279.13	279.09
BH4/MW	284.03	284.65	282.51	282.55	282.53	282.42	282.14	281.75	281.59	281.59	281.55	281.57
BH5/MW	284.59	285.49	283.12	282.51	282.46	282.32	282.37	282.08	282.04	282.04	282.02	Dry

Appendix D – Inspection and Testing Schedule

INSPECTION & 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 in situ density test per 50 linear metres of road
- 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 E – 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 test pit results contained in the Report. The number of test pits 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 envelopment 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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