Craigholme Estates Ltd.

Geotechnical Investigation

Project Name
Craigholme Subdivision – Phase 6

Project Location
Seventh Avenue, Belmont, Ontario

Project Number
LON-00016106-GE

Prepared By
EXP Services Inc.
15701 Robin's Hill Road
London, ON   N5V 0A5
Canada

Date Submitted
March 18, 2019

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Date Submitted:
March 18, 2019

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1. Introduction

As requested, EXP Services Inc. (EXP) has conducted a preliminary geotechnical investigation in conjunction with the proposed residential development to be located at Seventh Line, Belmont, Ontario. It is understood that the development will have full municipal servicing and will be accessed by local roadways. This report summarizes the results of the investigation, and provides geotechnical engineering guidelines to assist with the design and construction of the proposed development.

1.1 Terms of Reference

The geotechnical investigation was generally done in accordance with details discussed in email correspondence between EXP and the client. Authorization to proceed with the investigation was received from Mr. Don Leahy of Craigholme Estates Ltd.

The purpose of the investigation was to examine the subsoil and groundwater conditions at the Site by advancing a series of sampled boreholes at the locations shown on the attached Borehole Location Plan (**Drawing 1**).

Based on an interpretation of the factual borehole data, and a review of soil and groundwater information from test holes advanced at the site, EXP Services Inc. has provided engineering guidelines to assist with the geotechnical design and construction of the proposed residential subdivision. More specifically, this report provides comments on site preparation, excavations, dewatering, foundations, pipe bedding, backfill, and pavement design. Comments are also provided on groundwater infiltration and Stormwater Management Facilities.

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

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 conducted on March 28 and 29, 2018 and consisted of drilling a total of six (6) boreholes at the approximate locations shown on **Drawing 1**. The boreholes within the site are designated as BH1 to BH6 inclusive. The borehole labels were suffixed with "MW" where monitoring wells were installed.

The boreholes were advanced using a locally subcontracted drill rig equipped with continuous flight solid stem augers, soil sampling and soil testing equipment. The boreholes within the site were each terminated at depths varying between 6.6 and 9.6 m below ground surface (bgs).

Within the boreholes, Standard Penetration Tests (SPTs) were performed to assess the compactness or consistency of the underlying soils and to obtain representative samples. During the drilling, the stratigraphy in the boreholes was examined and logged in the field by EXP geotechnical personnel. Short-term groundwater level observations within the open boreholes and the natural moisture contents of recovered soil samples are recorded on the borehole logs found in **Appendix A**.

Following the drilling, Borehole BH6 was backfilled with the excavated materials and bentonite, to satisfy the requirements of O.Reg. 903. Monitoring wells were installed in all other boreholes. The details of the monitoring wells are provided on the Borehole Logs in **Appendix A**.

Representative samples of the various soil strata encountered at the borehole locations were taken to our laboratory in London for further examination by a geotechnical engineer and laboratory classification testing. Laboratory testing for this investigation comprised of routine moisture content determinations, with results presented on the borehole logs found in **Appendix A**. Two grain size analyses were conducted on samples of the till and sandy silt. The results of the grain size analyses are summarized in Section 3.2 and the graphs provided in **Appendix B.**

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

The ground elevations of the boreholes were surveyed by EXP to the top of spindle of the fire hydrant on the south east corner of Snyders Avenue and Street B as shown on the topographic plan of survey prepared by MTE OLS Ltd. dated March 9, 2018 (Geodetic Elevation 262.229 m).

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.

3. Site and Subsurface Conditions

3.1 Site Description

The site is located at Part of Lot 2, Concession 7, Municipality of Central Elgin, formerly the Village of Belmont in the County of Elgin. It fronts onto Seventh Avenue which runs west off of Main Street in Belmont.

The Site currently is utilized for agricultural purposes. It is approximately 17.4 Ha in size. There is a wetlands area in the south portion of the property. Field tiles were encountered at two of the boreholes during the drilling program.

3.2 Soil Stratigraphy

The detailed stratigraphy encountered in each borehole and the results of routine laboratory tests carried out on representative samples of the subsoils are presented on the attached borehole logs. It must be noted that boundaries of soil indicated on the logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purposes of geotechnical design and should not be interpreted as exact planes of geological change.

The subsurface soil conditions encountered in the boreholes are detailed on the Borehole Logs provided in **Appendix A**, and summarized as follows.

3.2.1 Topsoil

Topsoil was at surface at all boreholes. The topsoil varied in thickness between about 230 mm and 300 mm.

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

3.2.2 Clayey Silt Till

Clayey silt till was encountered below the topsoil at all boreholes. All boreholes except for Borehole BH1/MW were terminated in the till. The till was weathered in the upper levels and contained trace sand and trace gravel. The relative density of the till was firm to hard based on SPT N Values of 7 to 50 for 75 mm penetration of the split spoon sampler and was described as moist with laboratory *in situ* moisture contents of 10 to 28 percent. Shear strengths of the till were obtained using a pocket penetrometer with readings varying from 100 kPa to greater than 225kPa. The till generally became grey between 2.1 m bgs and 2.9 m bgs. Wet sand seams were observed in Boreholes BH1/MW and BH2/MW near 7.9 m bgs, and BH3/MW near 1.8 m bgs, 6.3 m bgs and 7.9 m bgs.

Borehole BH5/MW was terminated at 6.1 m bgs due to auger refusal. It is likely that the auger encountered a cobble.

Field tiles were found at Boreholes BH4/MW and BH6 near 1.2 m bgs.

A grain size distribution of the till is provided in **Appendix B, Figure 1** and summarized below.

Table 1 – Grain Size Analysis Results

Sample	Sample Depth m bgs	Composition, %			
		Gravel	Sand	Silt	Clay
BH2/MW, S8	7.6 – 8.1	3	13	51	33

3.2.3 Sandy Silt

Borehole BH1/MW encountered sandy silt near 8.6 m bgs. The sandy silt was grey in colour with some clayey layering, dilatant, and compact (SPT N Value of 22). Borehole BH1/MW was terminated in this layer. A grain size distribution of the sandy silt is provided in **Appendix B, Figure 2** and summarized below.

Table 2 – Grain Size Analysis Results

Sample	Sample Depth m bgs	Composition, %			
		Gravel	Sand	Silt	Clay
BH1/MW, S9	9.1 – 9.6	0	35	59	6

3.3 Groundwater Conditions

Monitoring wells were installed in five of the six boreholes. A summary of the groundwater measurements is provided below.

Table 3 – Summary of Groundwater Findings

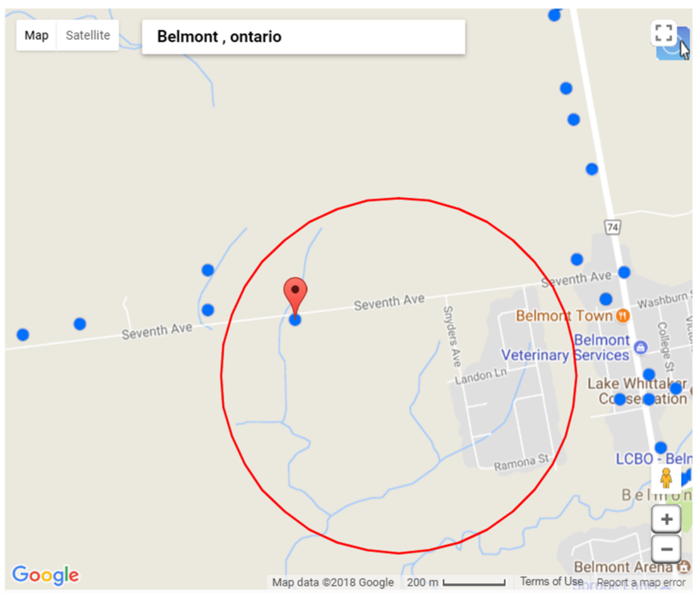
Location	Ground Surface Elevation m	Depth Monitoring Well Installed To, m bgs	Water Levels and Elevations							
			Apr 2/18		Apr 18/18		Apr 24/18		May 11/18	
			Depth m bgs	Elev m	Depth m bgs	Elev m	Depth m bgs	Elev m	Depth m bgs	Elev m
BH1/MW	261.3	9.1	1.4	259.9	1.1	260.2	0.9	260.4	1.0	260.3
BH2/MW	259.6	7.6	2.8	256.8	0.6	259.0	0.7	258.9	0.7	258.9
BH3/MW	259.0	7.6	1.7	257.3	1.6	257.4	1.5	257.5	0.6	258.4
BH4/MW	256.3	9.1	1.5	254.8	0.4	255.9	0.5	255.8	0.6	255.7
BH5/MW	257.5	6.1	1.5	256.0	1.3	256.2	1.3	256.2	1.4	256.1

Based on observations during and upon completion of drilling the boreholes, groundwater appeared to be located in sand and silt seam within the till and varies between elevations 255.8 to 260.4 m.

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 high levels in wet seasons. Capillary rise effects should also be anticipated in fine-grained soil deposits.

3.4 Potable Groundwater (Wells)

A review of the local Ministry of Environment and Climate Change (MOECC) well records was carried out for the area surrounding the site (see picture below) to identify the depth of the potable groundwater aquifer for the area.



Latitude:42.89415, Longitude:-81.11312 (UTM Zone:17, Easting:490764, Northing:4749066)

Only one well was located within the area. The well record for this well is provided in **Appendix C** and summarized below in Table 4.

Table 4 – MOECC Well Record No. 41-11588

Well Location Concession Lot	MOECC Well No.	Elev. (ft)	Depth of Well (ft)	Depth Water Found (ft)	Static Water Level Depth (ft)	Static Water Level Elev. (ft)	Rate of Pumping gal./min.
Conc. 7 / Lot 2	41-11588	N/R	238	238	33	- -	12

Groundwater flow across the site is affected by the soil permeability, topography and drainage. The potable well recorded in the MOECC Well Records is set at a depth of 238 ft in water bearing gravels. The well in the area indicates that potable water is generally found in deep aquifers.

4. Discussion and Recommendations

4.1 General

Two hundred single family lots are proposed for the development which will also include full municipal servicing and paved roadways. The following sections of this report provide geotechnical recommendations regarding site preparation, excavations, dewatering, foundations, bedding, backfill, infiltration potential, earthquake design considerations, pavement design and stormwater management design considerations.

4.2 Site Preparation

Prior to placement of foundations and/or engineered fill, any existing fill, loose soils, topsoil and/or otherwise deleterious materials should be stripped from the footing area of the proposed buildings.

Following the removal of the deleterious material and unsuitable soil described above, and prior to structural fill placement, the exposed subgrade should be inspected by a geotechnical engineer. Any loose or soft zones noted during the inspection should be further excavated and replaced with approved structural fill.

In proposed building areas where the subgrade requires reconstruction to achieve the design elevations, engineered fill should be used. The geometric requirements for engineered fill are provided on **Drawing 2**.

The engineered fill should consist of clean (i.e., free of organics and/or debris), compactable, inorganic soils with a moisture content within about 3 percent of optimum, as determined by standard Proctor testing. Where engineered fill thicknesses of more than 3 metres are required to achieve the design underside of footing (USF) elevations, the engineered fill mat should consist of granular fill instead of fine-grained soils. The granular fill material (>3 m) is recommended in the building areas to reduce risk of settlements. Ideally, a sand and gravel blend (such as OPSS 1010 Granular B) is recommended; however, there are also options of using sand fill, subject to review and approval by the geotechnical consultant onsite.

Within the roadway, fill materials may comprise of other on-site excavate soils or imported granular fill materials approved by an engineer. Where the two types of fill material meet, it is recommended that the granular fill be benched into the other fill. Regardless of the type of material used, full time inspection and testing is required to monitor and certify the engineered fill.

The engineered fill material should be inspected and approved by a geotechnical engineer and should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD). *In situ* compaction testing should be carried out during the fill placement to ensure that the specified compaction is being achieved.

If imported fill material is used at the Site, verification of the suitability of the fill may be required from an environmental standpoint. Conventional geotechnical testing will not determine the suitability of the material in this regard. Analytical testing and environmental site assessment may be required at the source. This will best be assessed prior to the selection of the material source. A quality assurance program should be implemented to ensure that the fill material will comply with the current MOECC standards for placement and transportation.

As indicated previously, the disposal of any excess excavated materials must conform to the MOECC Guidelines and requirements. EXP can be of assistance if an assessment of the materials is required.

4.3 Excavation and Construction Dewatering

4.3.1 General

Side slopes of temporary excavations must conform to Regulation 213/91 of the Occupational Health and Safety Act of Ontario. The stiff to hard clayey silt till would generally be classified as Type 2 soil. For Type 2 soils, the sides of excavations may be cut vertically within the bottom 1.2 m and then with a slope having a minimum inclination of 1H:1V above that level.

Where excavations encounter clayey silt till with wet sand seams and water seepage, the soil would be classified as Type 3 soil and excavation side walls should be sloped from the base of the excavation with a slope having a maximum inclination of 1H:1V

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

4.3.2 Excavation Support

The recommendations for side slopes given in Section 4.3.1 would apply to most of the conventional excavations expected for the proposed development. However, in areas adjacent to existing structures and 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 cohesionless 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/m^3 may be assumed;

h = depth of point of interest in m;

q = equivalent value of any surcharge on the ground surface in kPa.

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

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 them to the engineer for review and comment.

4.3.3 Construction Dewatering

Based on the results of the field investigation, groundwater infiltration should be anticipated within conventional depths for buildings and service trench excavations. It is expected that the groundwater infiltration can likely be accommodated using conventional sump pumping-dewatering techniques at this Site. Where groundwater infiltration persists, more extensive dewatering measures may be required. Based on the grain size analysis conducted on the clayey silt till at BH2/MW the permeability of the soils is approximately in the order of 10^{-5} to 10^{-6} cm/sec. EXP would be pleased to provide additional comments and recommendations for dewatering these soils, when additional design information is available. It is also recommended that contractors bidding on the work conduct further investigation including test pits to further determine groundwater conditions and how it will affect their work.

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

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

4.4 Building Foundations

Foundations for the proposed buildings can be set on the natural, competent soils at the depths shown below.

Table 5 – Suitable Founding Levels for a Bearing Resistance of 145 kPa

Borehole No.	Surface Elevation, m	Depth to Competent Soils, m	Elevation at Competent Soils, m
1	261.3	2.1	259.2
2	259.6	0.9	258.7
3	259.0	0.9	258.1
4	256.3	0.9	255.4
5	257.5	0.9	256.6
6	260.4	0.9	259.5

Notes: Actual founding levels may be affected by existing structures and services including abandoned ones.

The following allowable bearing pressures (net stress increase) can be used on the natural, undisturbed soils at the depths and locations noted in the table above:

Bearing Resistance at Serviceability Limit States (SLS) 145 kPa (3,000 psf)

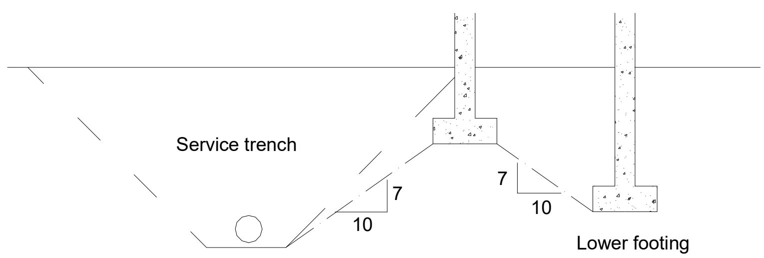
Factored Bearing Resistance at Ultimate Limit States (ULS) 190 kPa (4,000 psf)

Field tiles were encountered at Boreholes BH4/MW and BH6. Founding depths can be influenced by the presence of field tiles and other services. Localized improvement in these areas will be required to ensure competency of founding subgrade soils.

Groundwater levels were recorded between elevations 255.8 to 260.4 m. Founding levels should be at 0.5 m above the observed groundwater table. If basements occur at 0.5 m above the groundwater levels (255.8 m to 260.4 m), or lower, a waterproofing system is recommended. Further comments can be provided once founding elevations are known.

If the grades are to be raised or restored due to unsuitable soils, engineered fill can be used over the competent subgrade, as discussed in Section 4.2. An available SLS bearing capacity for the engineered fill is 145 kPa (3,000 psf). The geometric requirements for the fill placement are shown on **Drawing 2**. For strip footings placed on engineered fill, it is recommended that they are widened to 500 mm (20 inches), and contain nominal reinforcing steel. Verification of the soil conditions and the extent of reinforcement are best determined by the geotechnical engineer at the time of excavation.

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 excavations, etc. to ensure that undermining is not a problem.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

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.

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.

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 (i.e., where more specific information becomes available with respect to conditions between test locations when foundation construction is underway). The interpretation between the boreholes and the recommendations of this report must therefore be checked through field inspections provided by EXP to validate the information for use during the construction stage.

4.5 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 thoroughly proof-rolled. Any soft zones detected during the proof-rolling should be dug out and replaced with clean, compactable material, placed in accordance with the requirements outlined in Section 4.2.

As noted earlier, groundwater levels were observed between elevations 255.8 to 260.4 m. It is preferred to keep basement elevations 0.5 m above the groundwater table. Waterproofing will be required for basements that are near or below the groundwater table levels. See **Drawing 3.**

A minimum 200 mm (8 inch) thick compacted layer of 19 mm (¾ inch) clear crushed stone should be placed between the prepared subgrade and the floor slab to serve as a moisture barrier.

All basement walls should be damp-proofed and must be designed to resist a horizontal earth pressure 'P' at any depth 'h' below the surface as given by the following expression: $P = K(\gamma h + q)$

where, P = lateral earth pressure in kPa acting at depth h;

γ = natural unit weight, a value of 20.4 kN/m^3 may be assumed;

h = depth of point of interest in m;

q = equivalent value of any surcharge on the ground surface in kPa.

K = earth pressure coefficient, assumed to be 0.4

Installation of perimeter drains is required for basements at the Site. The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall. Suggestions for permanent perimeter drainage are given on **Drawing 3**.

4.6 Pipe Bedding and Trench Backfill

The subgrade soils beneath the water pipe and sewers which will service the site are generally expected to consist of clayey silt till. No bearing problems are anticipated for flexible or rigid pipes founded on the natural deposits or compacted on-site soils.

Consideration should be given to placing the bedding in accordance with the specifications outlined in OPSS. The bedding course may be thickened if portions of the subgrade become wet during excavation. The bedding aggregate should be placed around the pipe to at least 300 mm (12 inch) above the pipe. The bedding aggregate should 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.

Clear stone or crushed stone bedding may be used in the service trenches as bedding below the spring line of the pipe if necessary to assist groundwater control and provide stabilization to the excavation base in wet silty soils. Geotextile should be wrapped around the stone bedding to minimize migration of fines. The potential locations for use of stone bedding should be identified during construction and are expected to vary across the site due to seasonal conditions and variations in the groundwater table.

Requirements for backfill in service trenches, etc. should also have regard for OPSS requirements. A program of *in situ* density testing should be set up to ensure that satisfactory levels of compaction are achieved. Trench backfill requirements are provided in **Drawings 4** and **5** and a suggested testing schedule is attached in **Appendix D**.

Based on the results of this investigation, much of the excavated natural soils may be used for construction backfill, provided that reasonable care is exercised in handling as discussed in Section 4.2. In this regard, the material should be within 3 percent of the optimum moisture as determined in the Standard Proctor density test. Stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet, adverse weather. As well, materials should be stockpiled according to composition, i.e., sand soils should not be mixed with clayey silt soils.

Soils excavated from below the stabilized groundwater table may be too wet for re-use as backfill unless adequate time is allowed for drying, or if the material is blended with approved dry fill; otherwise, it may be stockpiled on the Site for re-use as landscape fill. The use of any imported material is subject to review and approval by the contract administrator and geotechnical consultant.

Disposal of excavated materials off site should conform to current MOECC guidelines.

4.7 Infiltration Gallery Considerations

It is understood that an assessment for the potential use of infiltration galleries for stormwater management purposes at this site be addressed as part of the Low Impact Development (LID) considerations. The clayey soil permeability from this investigation was determined to be in the range of 10^{-5} cm/sec to 10^{-6} cm/sec. At a clayey site, the opportunity is limited to enhance the LID designs.

The hydraulic conductivity of this soil type is not conducive for vertical exfiltration purposes. Based on the findings during this geotechnical investigation, a conventional infiltration trench is not a practical option for storm water management. The soils encountered in this investigation are consistent with soils in the area. The permeability of these soils based on our analysis is in the order of 10^{-5} cm/sec to 10^{-6} cm/sec which represents an infiltration rate of approximately 5 to 12 mm / hour (OMMAH, 1997). For design purposes, an infiltration rate should be limited to 8 mm / hour.

It is understood that MOECC requires infiltration galleries to be 1 m above the seasonal high ground water elevation. Based on groundwater levels near elevations 255.8 to 260.4 m, a 1 m separation distance may not be possible.

Therefore, infiltration galleries are not the most feasible solution for this site from the hydraulic conductivity of the soil and the high groundwater elevation point of view. Some LID's such as thickening of the topsoil will be possible.

4.8 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. Excluding the topsoil, the subsoils across the site will generally consist of clayey silt till. It is anticipated that the proposed structures will be founded on these 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 were advanced at this Site to a maximum depth of 9.6 m bgs. Therefore, the Site Classification recommendation would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the area.

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

4.9 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 zones 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.

Based on past experience with this site and provided the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated specified street classifications, a typical design life of 20 years, and the anticipated subgrade soil conditions.

Table 6 – Recommended Pavement Structure

Pavement Layer	Compaction Requirements	Local Roads	Seventh Avenue Restoration
Asphaltic Concrete	97% Marshall Density	40 mm HL-3 50 mm HL-8	50 mm HL-3 60 mm HL-8
Granular 'A' (Base) Granular 'B' (Subbase)	100% SPMDD* 100% SPMDD	150 mm 450 mm	150 mm 450 mm
Notes: 1) *SPMDD denotes Standard Proctor Maximum Dry 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 natural 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.

Due to the shallow ground water table and sensitive clay at this site, it is recommended to have dedicated construction roads away from the proposed road locations. In this way, heavy construction vehicle usage will not produce pumping of the subgrade in the future road allowance. The subgrade preparation and granular sub-base requirements should be reviewed by the geotechnical engineer from this office at the time of construction.

Depending on the staging of the development, and possible areas of concentrated construction access routes, additional granular thicknesses may also be considered. If only a portion of the pavement will be in place during construction, the granular subbase may have to be thickened, and/or the subgrade improved with a geotextile separator or geogrid stabilizing layer. This is best determined in the field during the site servicing stage of construction, prior to road construction.

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

The asphaltic concrete paving materials should conform to the requirements of OPSS 1150. The asphalt should be placed in accordance with OPSS 310 and compacted to at least 97 percent of the Marshall mix design bulk density.

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 catchbasins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. In low areas (at catchbasin locations), subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening, as shown on **Drawing 6**. This is particularly important in heavier traffic areas at the site entrances. The locations and extent of subdrainage required within the paved areas should be reviewed by this office in conjunction with the proposed grading.

Where the roadway from the development intersects the existing roads, the subgrade beneath the new pavement should be tapered to match the existing road subgrade to minimize differential frost heaving for the pavement structure.

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

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

4.10 Curbs and Sidewalks

The concrete for the curbs and gutters should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353 and OPSS 1350 Standards.

During cold weather, the freshly placed concrete should be covered with insulating blankets to protect against freezing.

The subgrade for the sidewalks should consist of undisturbed natural soil or well-compacted fill. A minimum 100 mm thick layer of compacted (minimum 98 percent SPMDD) Granular 'A' should be placed below the sidewalk slabs.

4.11 Inspection and Testing Recommendations

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

- Subgrade examination prior to placement of engineered fill;
- 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;

- Inspection and Materials testing during the road construction, including subgrade examination of the road 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), *in situ* density testing, and concrete sampling and testing for curbs;

- Inspection and Materials testing for base and surface asphalt, including laboratory testing on asphalt sampling to confirm conformance to project specifications and standards;

- Footing Base Examinations for residential footings set on engineered fill or natural soils to confirm their suitability to support the design bearing pressures; and

- Visual examination of concrete reinforcing steel placement in footings set on engineered fill.

5. Hydrogeological Comments

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

The MOECC Well Records for this area are summarized in Section 3.4. Groundwater information provided by MOECC Records indicate that groundwater in the area is generally sourced from deep overburden aquifers (72.5 m depth).

It is noted that the site is located just outside of Belmont and municipal water will be supplied to the development.

Shallow groundwater flow across the site is typically affected by the soil permeability, topography and drainage. Intermediate and deep aquifers are significantly less affected by surface conditions.

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

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

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

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

6. Stormwater Management Design Considerations

It is understood that a Storm Water Management (SWM) Facility is proposed for the south end of the site, for stormwater retention/storage and siltation control. While no design parameters were presented at the time of writing this report, EXP has the following general comments related to SWM Facility construction.

The construction and design of the facility should conform to the current Municipality and OPS Standards.

Based on the soils encountered on site, the predominant subgrade soils for the pond are expected to comprise of clayey silt till.

The following preliminary geotechnical comments are provided for design and construction of the pond and associated structures. During the design stage of the SWM facility, EXP should be afforded the opportunity to review the design to ensure that suitable geotechnical recommendations have been provided and properly interpreted.

6.1 Site Preparation

The surficial topsoil and vegetation should be removed from the pond area. The topsoil varied in thickness across the site, and may be stockpiled on site for possible reuse as landscaping fill.

The disposal of any excavated materials and debris off site should conform to current MOECC Guidelines.

The major soils likely to be generated from the cut areas at the south end of the SWM Pond are clayey silt till. Based on the results of *in situ* moisture contents and a grain size analysis on the clayey silt till, the majority of the on-site excavated clayey silt till can be re-used for the construction of the pond sidewalls, or in areas requiring fill to meet design grades.

6.2 Pond Construction

Following the cutting operations, the exposed subgrade should be scarified and thoroughly proof-rolled and examined by a geotechnical engineer. The glacial till contacted in the test holes is considered suitable for stormwater retention, and the construction of a liner in this material is not required. In the event localized sandy pockets are encountered during the pond construction, this material should be over excavated and replaced with approved, less permeable material.

If larger areas of silty sand or sandy silt soils are contacted, then consideration should be given to install a natural liner in these areas. The excavated clayey silt till is generally considered suitable for use as liner material, where required, and should be placed in a minimum 150 mm thick lifts and compacted with a sheepsfoot roller to minimum 97 percent standard Proctor maximum dry density. A compacted design thickness of 600 mm is recommended. The natural moisture content of the material should be three percent wet of the optimum moisture content as determined by Standard Proctor density test.

It is recommended that the pond slopes be constructed with an inclination flatter than about 2.5 horizontal to 1.0 vertical. Ideally, slopes no steeper than 3.0 horizontal to 1.0 vertical are encouraged, in order to lessen the potential for erosion. The current design is consistent with this recommendation. If the soil is subject to erosion or inundation from water then the slopes should be lined with concrete or rip-rap.

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

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

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

6.3 Inlet/Outlet Structures

In the vicinity of the proposed inlet/outlet structures, culverts and/or pipes should be carefully backfilled with excavated glacial till soils. The backfill should be in intimate contact with complete circumference of the pipe. In places where proper compaction may be difficult to achieve, lean concrete backfill should be used.

The support for outlet structure must be derived from the natural soils or engineered fill. An allowable bearing pressure of 145 kPa (3000 psf) is available in these soils. Any headwalls should be backfilled using free-draining granular material and may be designed using an active earth pressure coefficient of 0.4 and a unit weight of 21 kN/m^3. Any footings must be protected with a minimum 1.2 m of earth cover or equivalent insulation to provide protection against potential frost damage.

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

7. General Comments

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

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

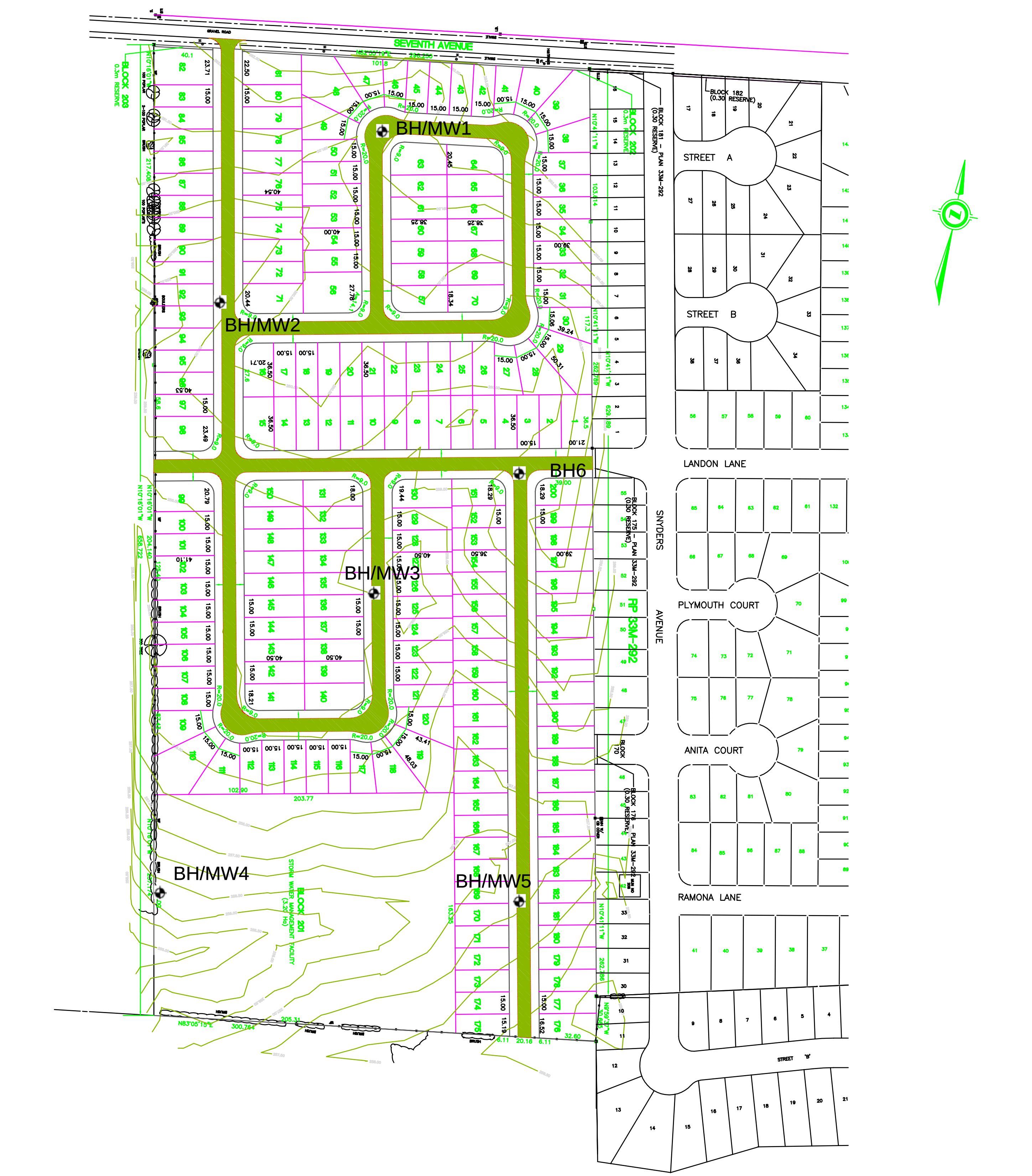
The comments given in this report are intended only for the guidance of design engineers. The number of test holes required to determine the localized underground conditions between test holes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than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

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

This report was prepared for the exclusive use of **Craigholme Estates Ltd.** and may not be reproduced in whole or in part, without the prior written consent of EXP, or used or relied upon in whole or in part by other parties for any purposes whatsoever. Any use which a third party makes of this report, or any part thereof, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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

Drawings



-NOTES-

1. The boundaries and soil types have been established only at test hole locations. Between test holes they are assumed and may be subject to considerable error.
2. Soil samples will be retained in storage for 3 months and then destroyed unless client advises that an extended time period is required.
3. Topsoil quantities should not be established based on the information provided at borehole locations.
4. The site plan has been reproduced from Google Maps and should be read in conjunction with EXP Geotechnical Report LON00016106-GE .

Geotechnical Investigation

Craigholme Estates, Phase 6
Belmont, Ontario

-LEGEND-

BH1 Approximate Borehole Location

CLIENT Craigholme Estates Ltd.

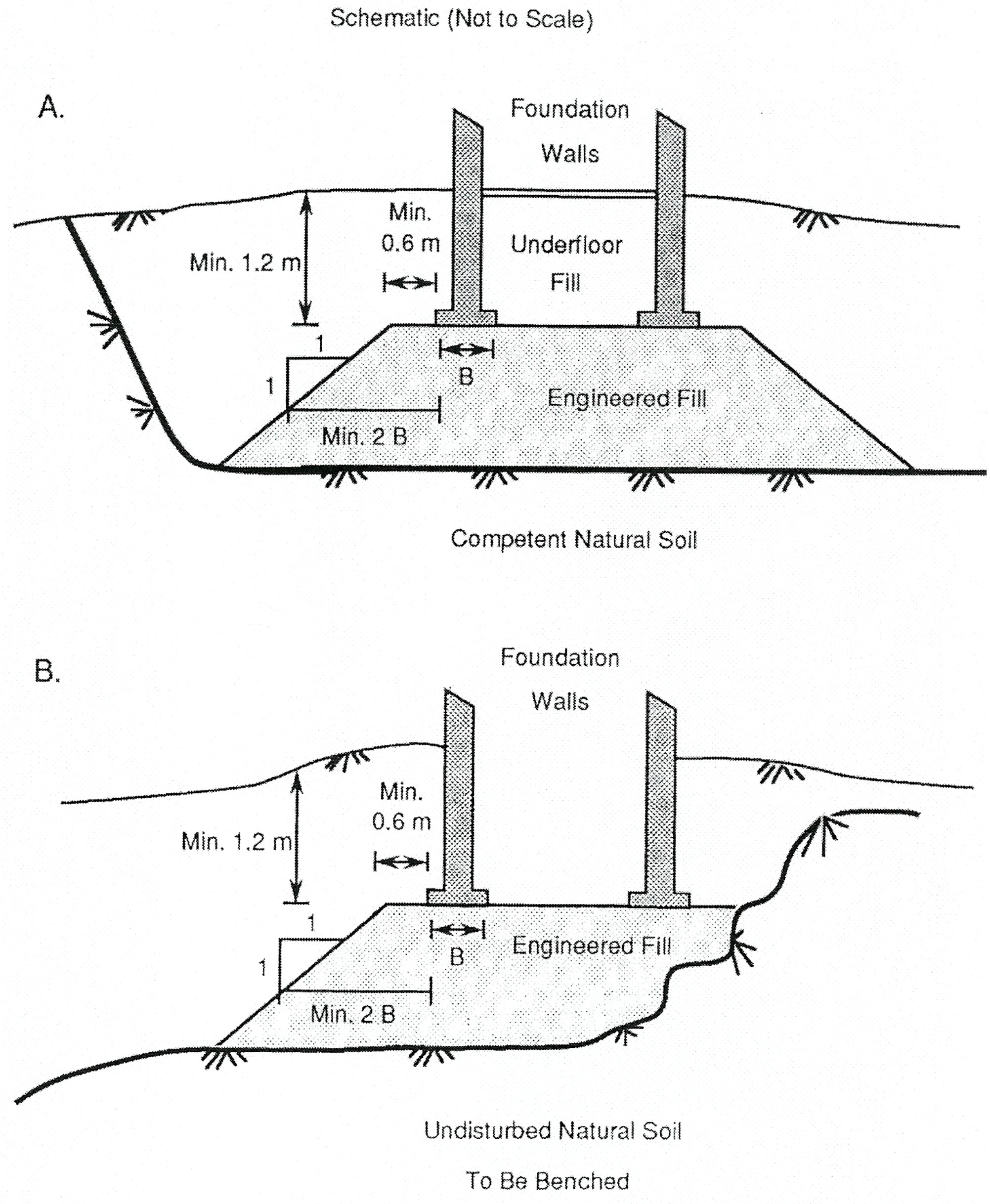
TITLE Borehole Location Plan

DRAWN BY:	REVIEWED BY:	SCALE
V.C.	E.B.	NTS

exp.
EXP Services Inc.
15701 Robin's Hill Road
London, ON, N5V 0A5

DATE	PROJECT NO.	DWG.
April 2018	LON-00016106-GE	1

DRAWING NO. 2 - GEOMETRIC REQUIREMENTS FOR FOUNDATIONS ON ENGINEERED FILL

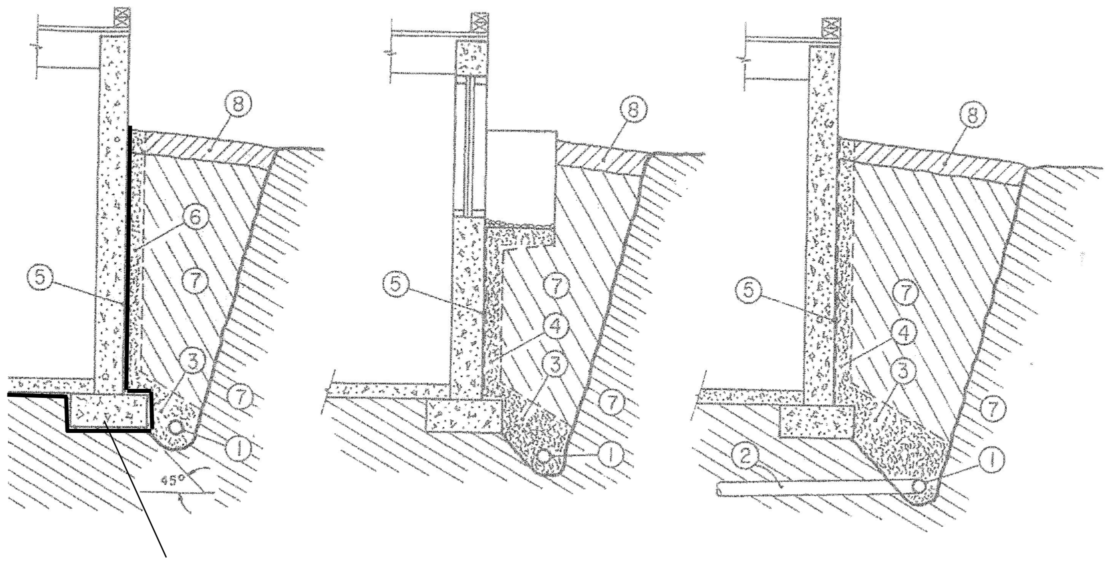


SECTION VIEW
Section A – Typical Section of Slab-on-Grade Building
Section B – Typical Section of Building with Basement
Refer to Detailed Notes on following page.

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 native 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. The imported fill should be reviewed to satisfy MOECC Requirements.
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.
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 (SLS) of 145 kPa (3,000 psf) may be used for foundations set on engineered fill, 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-00016106-GE).
11. Footing Base inspections are required to verify the suitability of the subgrade soils, at the time of construction. *In situ* density tests may also be required at the footing base level to confirm material density.

DRAWING 3 – STORM AND BACKFILL AND BASEMENT DRAINAGE DRAWING, (NOT TO SCALE)



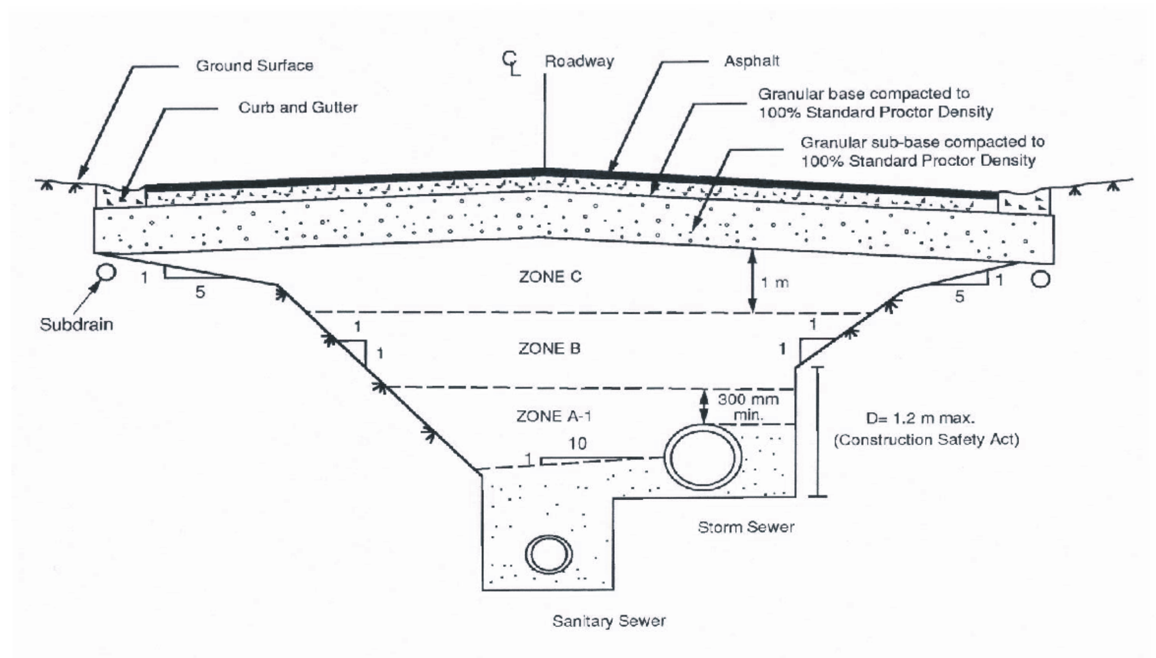
(9) ***Geomembrane Product***

NOTES:

1. Perforated or slotted pipe placed about 300 mm below the upper level of the basement floor slab;
2. Unperforated drain pipe connected to a positive sump; connect to appropriate trap and backwater valve before connecting to sewer, where applicable. Provide appropriate access to the trap for inspection and cleaning;
3. Filter material that is compatible with the grain size characteristics of the foundation and backfill soils, as well as the perforations of the pipe;
4. Filter material continuously or intermittently placed next to the foundation wall to intercept water from window wells and low areas near building (see also 6);
5. Damp-proofing on wall;
6. Use of sheet drain or synthetic filter blanket next to foundation wall as per OBC to replace the soil filter described in 4;
7. Foundation and backfill soils, which may contain fine grain and erodible materials;
8. “Topping-off” material, graded away from the building to redirect surface water from the foundations. Low permeability soils, or concrete or asphalt, are preferred.
9. Provide continuous geomembrane product beneath slab and footings where prevailing groundwater levels may be higher than basement floor slab elevation; extend geomembrane up to existing grade along exterior face of foundation wall.

Reference: Canadian Geotechnical Society 2006, *Canadian Foundation Engineering Manual, 4th Edition, (Figure 12.1, p184).*

DRAWING 4 – TYPICAL BACKFILL DETAIL STORM AND SANITARY SEWER (COMMON TRENCH)



SECTION VIEW

NOTES:

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

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

ZONE B: To be compacted to at least 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 5 – TRENCH BACKFILL REQUIREMENTS

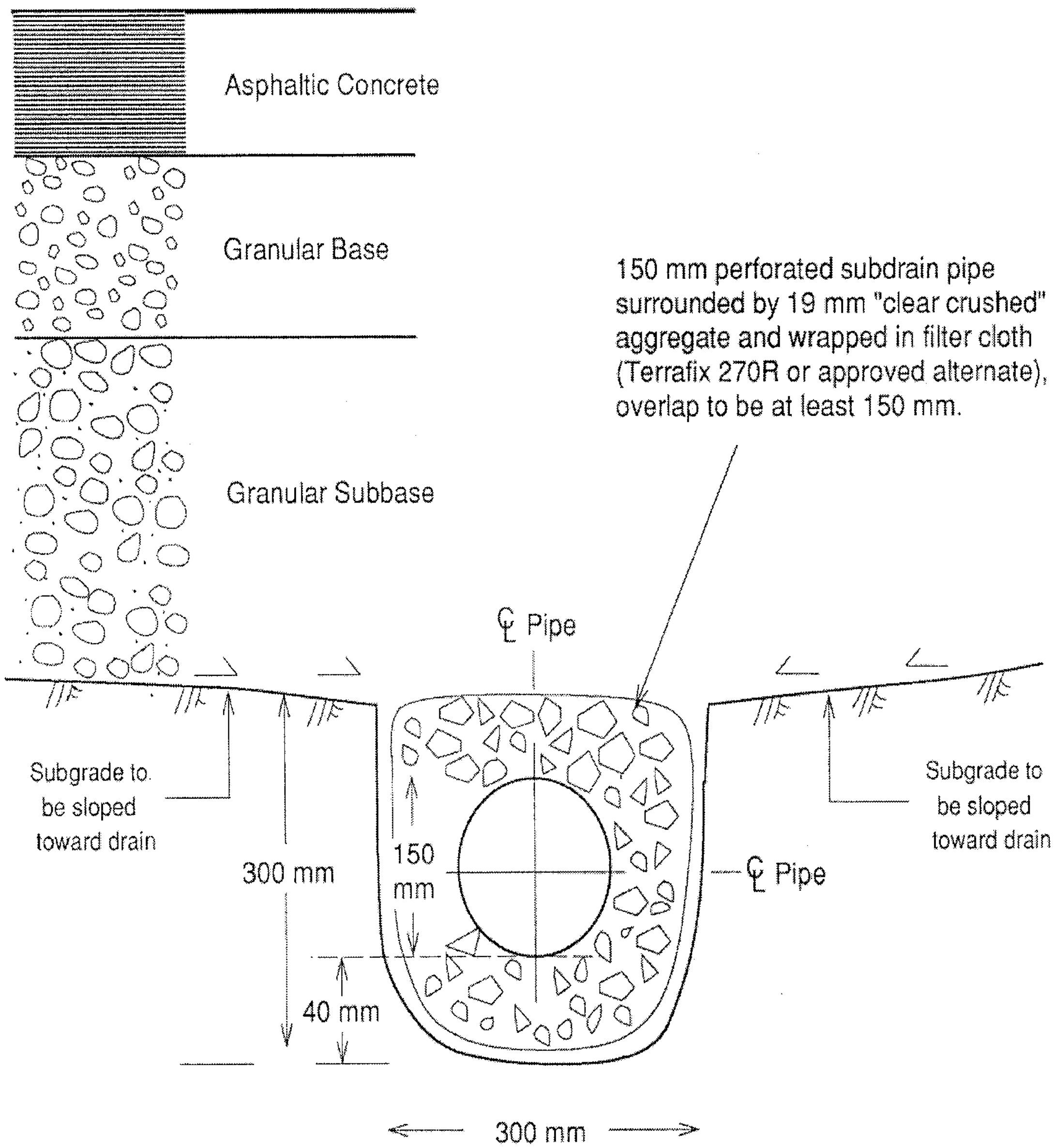
Requirements for backfill in service trenches, etc. should conform to current OPS requirements. A summary of the general recommendations for trench backfill is presented on Drawing 4.

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

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

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

DRAWING 6 – PAVEMENT SUBDRAIN DETAIL



NOTES:

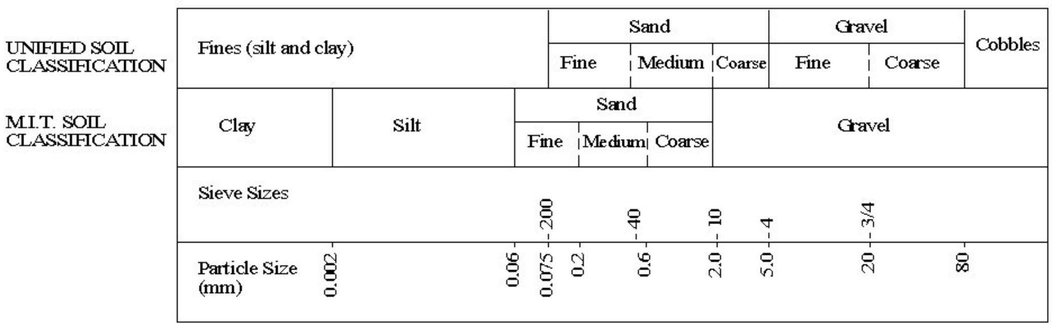
1. All dimensions in millimetres.
2. All subdrains to be set on at least 1% grade draining to a positive outlet.
3. Subgrade soil conditions should be verified onsite, during subgrade preparation works, following site servicing installations.

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.



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

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

BOREHOLE LOG

BH1/MW
Sheet 1 of 1

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**
PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**
LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 29, 2018** Water Level **Apr 24/18**

Depth (m bgs)	Elevation (~m)	Strata Description	Sample Type	Sample Number	Recovery (mm)	N Value (blows)	Moisture Content (%)
0	261.3						
	261.0	**TOPSOIL** - 280 mm					
		CLAYEY SILT TILL - brown/grey, weathered in upper levels, trace sand, trace gravel, firm to stiff, moist					
1			AS	S1			25
			SS	S2	400	7	26
2		- becoming grey, very stiff to hard near 2.1 m bgs					
			AS	S3			18
3			SS	S4	450	30	12
4			AS	S5			13
			SS	S6	400	29	14
5							
6			SS	S7	450	33	13
7							
8		- occasional wet sand seams encountered near 7.9 m bgs	AS	S8			21
	252.7	**SANDY SILT** - grey, some clayey layering, dilatant, compact, wet					
9			SS	S9	350	22	19
	251.7	**End of Borehole at 9.6 m bgs.**					
10							

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 Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) Water Level Readings:
April 2, 2018 - 1.4 m bgs, Elevation 259.9 m
April 18, 2018 - 1.1 m bgs, Elevation 260.2 m
April 24, 2018 - 0.9 m bgs, Elevation 260.4 m
May 11, 2018 - 1.0 m bgs, Elevation 260.3 m

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

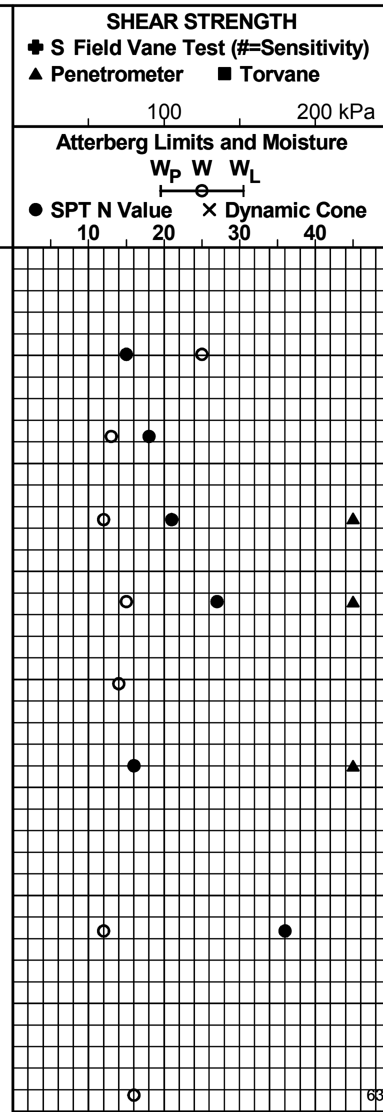
WATER LEVELS
Apparent Measured Artesian (see Notes)

BOREHOLE LOG

BH2/MW

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**
PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**
LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 29, 2018** Water Level **Apr 24/18**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	259.6						
	259.4	**TOPSOIL** - 230 mm					
		CLAYEY SILT TILL - brown/grey, weathered in upper levels, trace sand, trace gravel, very stiff to hard, moist					
1			SS	S1	300	15	25
			SS	S2	300	18	13
2			SS	S3	400	21	12
3		- becoming grey near 2.9 m bgs	SS	S4	400	27	15
4			AS	S5			14
5			SS	S6	450	16	16
6			SS	S7		36	12
8	251.5	- occasional wet sand seams encountered near 7.9 m bgs	SS	S8		63	16
		End of Borehole at 8.1 m bgs.					



NOTES

1) Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) Water Level Readings:
   April 2, 2018 - 2.8 m bgs, Elevation 256.8 m
   April 18, 2018 - 0.6 m bgs, Elevation 259.0 m
   April 24, 2018 - 0.7 m bgs, Elevation 258.9 m
   May 11, 2018 - 0.7 m bgs, Elevation 258.9 m

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)

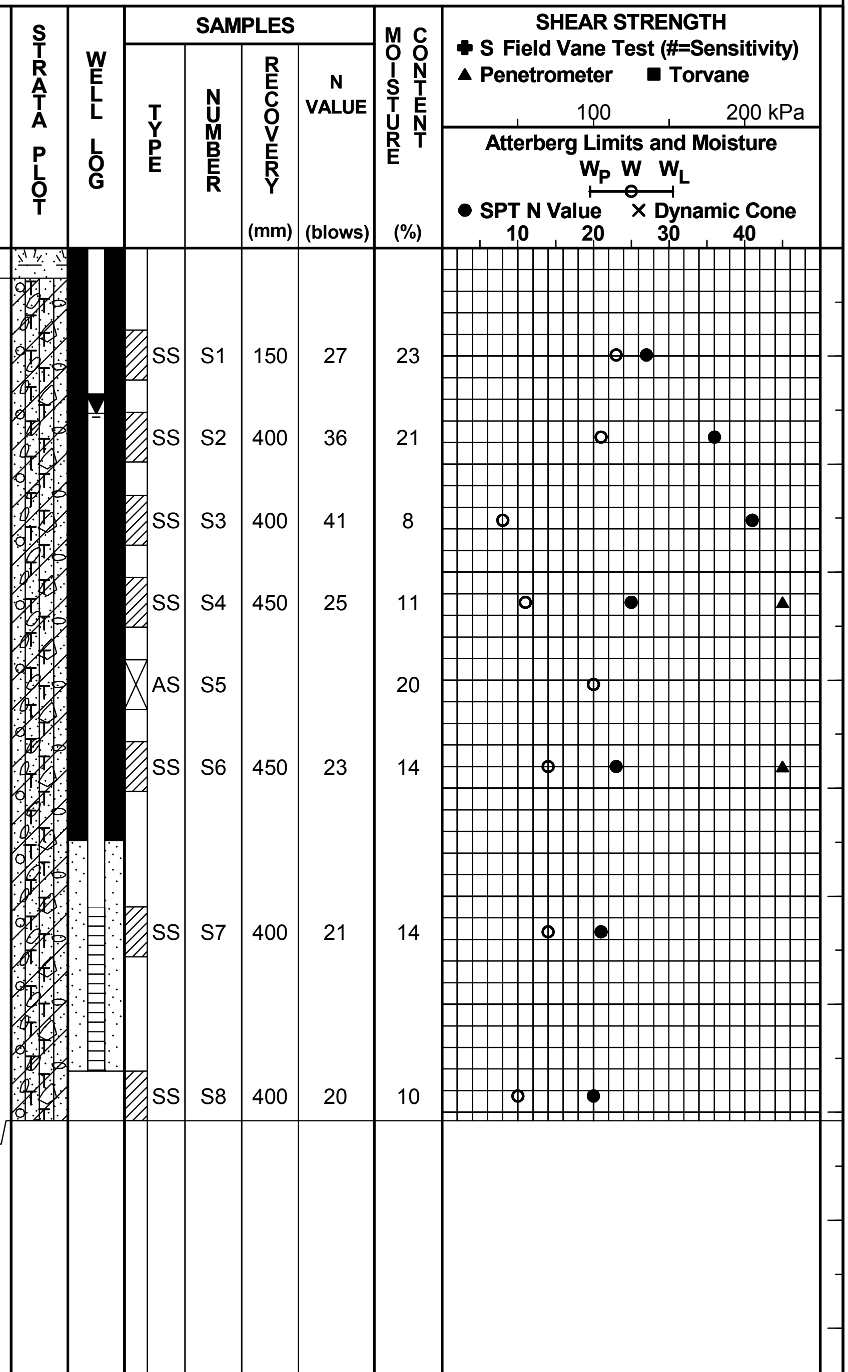
BOREHOLE LOG

BH3/MW
Sheet 1 of 1

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**
PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**
LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 28, 2018** Water Level **Apr 24/18**

Depth (m bgs)	Elevation (~m)	Strata Description
0	259.0	
	258.7	**TOPSOIL** - 280 mm
		CLAYEY SILT TILL - brown/grey, weathered in upper levels, trace sand, trace gravel, very stiff to hard, moist
~1.8		- wet sand seam encountered near 1.8 m bgs
~2.1		- becoming grey near 2.1 m bgs
~6.3		- occasional wet sand seams encountered near 6.3 m bgs
~7.9		- occasional wet sand seams encountered near 7.9 m bgs
8.1	250.9	**End of Borehole at 8.1 m bgs.**

Type	Number	Recovery (mm)	N Value (blows)	Moisture Content (%)
SS	S1	150	27	23
SS	S2	400	36	21
SS	S3	400	41	8
SS	S4	450	25	11
AS	S5			20
SS	S6	450	23	14
SS	S7	400	21	14
SS	S8	400	20	10



NOTES

1) Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) Water Level Readings:
April 2, 2018 - 1.7 m bgs, Elevation 257.3 m
April 18, 2018 - 1.6 m bgs, Elevation 257.4 m
April 24, 2018 - 1.5 m bgs, Elevation 257.5 m
May 11, 2018 - 0.6 m bgs, Elevation 258.4 m

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

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**
PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**
LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 28, 2018** Water Level **Apr 24/18**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	256.3						
	256.0	**TOPSOIL** - 280 mm					
		CLAYEY SILT TILL - brown/grey, weathered in upper levels, trace sand, trace gravel, stiff to very stiff, moist					
1			SS	S1	450	12	14
2			SS	S2	400	20	13
		- becoming grey near 2.1 m bgs	SS	S3	400	24	13
3			SS	S4	450	23	14
4			AS	S5			16
5			SS	S6		26	14
6			SS	S7	450	23	16
8			SS	S8	400	20	20
9	246.7		SS	S9		18	20
10		**End of Borehole at 9.6 m bgs.**					

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

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)

NOTES

1) Borehole Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) Water Level Readings:
April 2, 2018 - 1.5 m bgs, Elevation 254.8 m
April 18, 2018 - 0.4 m bgs, Elevation 255.9 m
April 24, 2018 - 0.5 m bgs, Elevation 255.8 m
May 11, 2018 - 0.6 m bgs, Elevation 255.7 m

BOREHOLE LOG

BH5/MW

Sheet 1 of 1

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**

PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**

LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 29, 2018** Water Level **Apr 24/18**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	257.5						
	257.3	**TOPSOIL** - 250 mm					
		CLAYEY SILT TILL - brown/grey, weathered in upper levels, trace sand, trace gravel, stiff to very stiff, moist					
1			SS	S1	400	11	18
			SS	S2	450	23	13
2			SS	S3	300	20	13
3		- becoming grey near 2.9 m bgs	SS	S4	450	22	13
4			AS	S5			13
5			SS	S6		22	19
		- becoming hard near 5.6 m bgs					
6		- possible cobble encountered near 6.1 m bgs	AS	S7	0	50*	14
	251.0	**End of Borehole at 6.6 m bgs due to auger refusal.**					
7							
8							
9							
10							

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 Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) bgs denotes below ground surface.
3) No significant methane gas concentration was detected upon completion of drilling.
4) * denotes 50 blows per 75 mm split spoon sampler penetration.
5) Water Level Readings:
   April 2, 2018 - 1.5 m bgs, Elevation 256.0 m
   April 18, 2018 - 1.3 m bgs, Elevation 256.2 m
   April 24, 2018 - 1.3 m bgs, Elevation 256.2 m
   May 11, 2018 - 1.4 m bgs, Elevation 256.1 m

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)

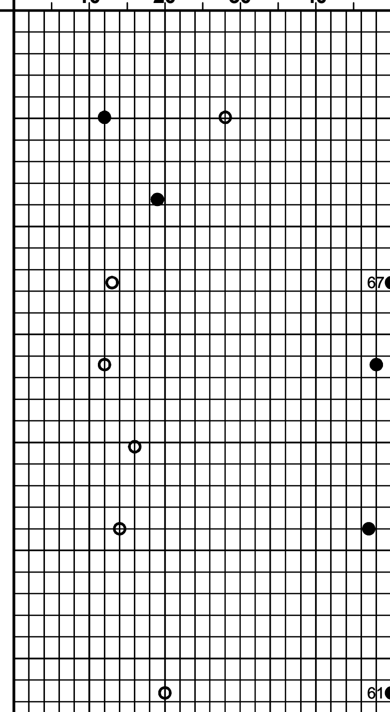
BOREHOLE LOG

BH6
Sheet 1 of 1

CLIENT **Craigholme Estates Ltd.** PROJECT NO. **LON-00016106-GE**
PROJECT **Craigholme Subdivision, Phase 6** DATUM **Geodetic**
LOCATION **Seventh Avenue, Belmont, Ontario** DATES: Boring **March 29, 2018** Water Level **Apr 24/18**

DEPTH (m bgs)	ELEVATION (~m)	STRATA DESCRIPTION	WELL LOG	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N VALUE (blows)	MOISTURE CONTENT (%)
0	260.4	**TOPSOIL** - 300 mm						
	260.1	**CLAYEY SILT TILL** - brown/grey, weathered in upper levels, trace sand, trace gravel, stiff to hard, moist						
1				SS	S1	400	12	28
		- wet field tile encountered near 1.2 m bgs						
			▽	SS	S2	450	19	19
2		- becoming grey near 2.1 m bgs						
				SS	S3	400	67	13
3				SS	S4	450	48	12
4				AS	S5			16
				SS	S6	400	47	14
5								
6				SS	S7		61	20
	253.9	**End of Borehole at 6.6 m bgs.**						

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 Log interpretation requires assistance by EXP before use by others. Borehole Log must be read in conjunction with EXP Report LON00016106GE.
2) Borehole open to 4.3 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.

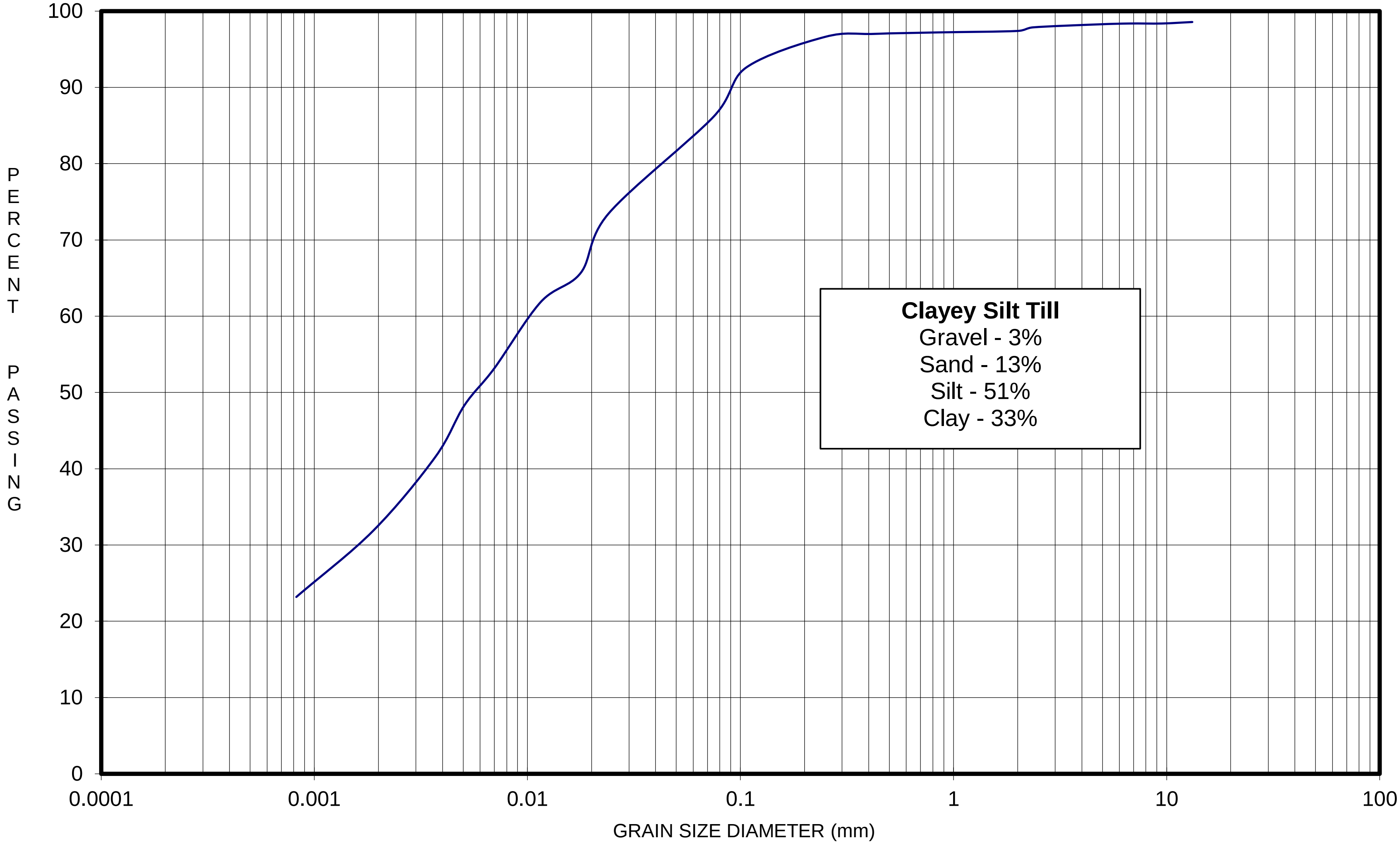
SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS
G Specific Gravity
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)

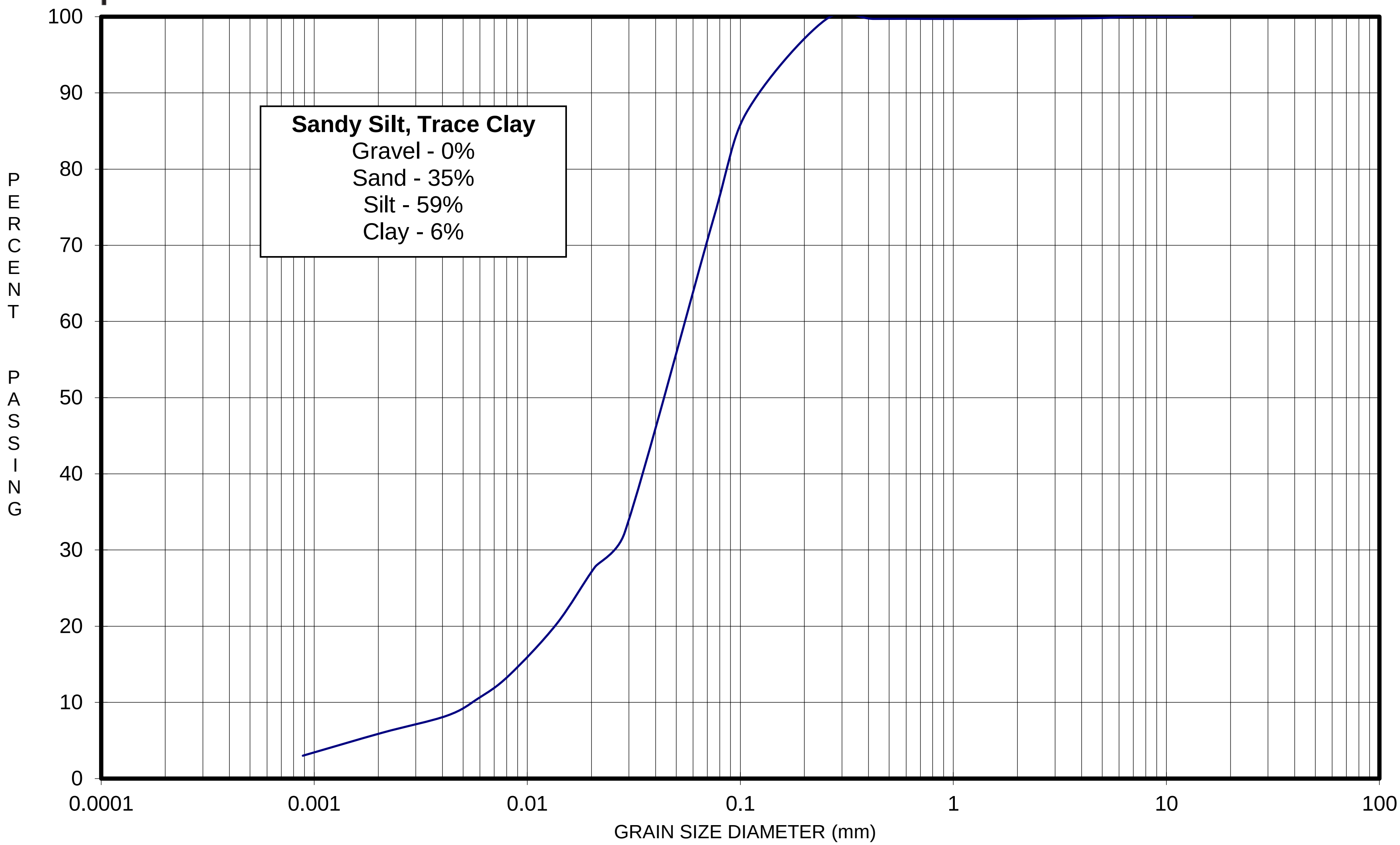
Appendix B – Grain Size Analyses

MECHANICAL GRAIN SIZE ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
	SILT			SAND			GRAVEL			
MODIFIED M.I.T. CLASSIFICATION	**Sample Description:** **Clayey Silt Till (BH2/MW S8, 7.6 - 8.1 m bgs)**					**Project: LON00016106GE** **Craigholme Subdivision Phase 6**			**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: Sandy Silt (BH1/MW S9, 9.1 - 9.6 m bgs)**					**Project: LON00016106GE Craigholme Subdivision Phase 6**			**Figure 2**

Appendix C – MOECC Well Records

Ministry of the Environment
Ontario

The Ontario Water Resources Act
WATER WELL RECORD

1. PRINT ONLY IN SPACES PROVIDED
2. CHECK ☒ CORRECT BOX WHERE APPLICABLE

11 | 4111588 | MUNICIP 41013 | CON. CON 07

COUNTY OR DISTRICT	TOWNSHIP, BOROUGH, CITY, TOWN, VILLAGE	CON., BLOCK, TRACT, SURVEY, ETC.	LOT
Middlesex	Westminster	7	2

[redacted] RR#1 Glanworth

DATE COMPLETED: DAY 20 MO 12 YR 88

[redacted] 47983 | RC | ELEVATION 886 | RC | BASIN CODE

LOG OF OVERBURDEN AND BEDROCK MATERIALS (SEE INSTRUCTIONS)

GENERAL COLOUR	MOST COMMON MATERIAL	OTHER MATERIALS	GENERAL DESCRIPTION	DEPTH - FEET FROM	DEPTH - FEET TO
Black	Top Soil		Porous	0	1
Brown	Gravel	Clay	Mixed	1	5
"	Clay	Gravel	"	5	10
Grey	Clay	"	"	10	32
"	Dry Sand		Packed	32	35
"	Clay	Gravel - Sand	Mixed	35	40
"	"	Stones	Dense	40	135
"	Fine Sand	Clay	Loose	135	143
"	Clay	Stones	Dense	143	180
"	Fine Sand — Clay		Layered - Mixed	180	234
"	Clay	Gravel - Sand	Mixed	234	235
"	Gravel	Boulders	Porous	235	238

31
32

41 WATER RECORD

WATER FOUND AT - FEET	KIND OF WATER
238	1 ☐ FRESH 2 ☐ SALTY 3 ☐ SULPHUR 4 ☐ MINERALS 6 ☐ GAS — 0.5 ppm
	1 ☐ FRESH 2 ☐ SALTY 3 ☐ SULPHUR 4 ☐ MINERALS 6 ☐ GAS
	1 ☐ FRESH 2 ☐ SALTY 3 ☐ SULPHUR 4 ☐ MINERALS 6 ☐ GAS
	1 ☐ FRESH 2 ☐ SALTY 3 ☐ SULPHUR 4 ☐ MINERALS 6 ☐ GAS
	1 ☐ FRESH 2 ☐ SALTY 3 ☐ SULPHUR 4 ☐ MINERALS 6 ☐ GAS

51 CASING & OPEN HOLE RECORD

INSIDE DIAM INCHES	MATERIAL	WALL THICKNESS INCHES	DEPTH - FEET FROM	DEPTH - FEET TO
6	1 ☒ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE 5 ☐ PLASTIC	.188	0	238
	1 ☐ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE 5 ☐ PLASTIC			
	1 ☐ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE 5 ☐ PLASTIC			

SCREEN

SIZE(S) OF OPENING (SLOT NO.)	DIAMETER INCHES	LENGTH FEET

MATERIAL AND TYPE | DEPTH TO TOP OF SCREEN FEET

61 PLUGGING & SEALING RECORD

DEPTH SET AT - FEET FROM	TO	MATERIAL AND TYPE (CEMENT GROUT, LEAD PACKER, ETC.)

71 PUMPING TEST

PUMPING TEST METHOD	1 ☒ PUMP 2 ☐ BAILER
PUMPING RATE	12 GPM
DURATION OF PUMPING	48+ HOURS MINS
STATIC LEVEL	33 FEET
WATER LEVEL END OF PUMPING	37 FEET
WATER LEVELS DURING	1 ☒ PUMPING 2 ☐ RECOVERY
15 MINUTES / 30 MINUTES / 45 MINUTES / 60 MINUTES	FEET / FEET / FEET / FEET
IF FLOWING, GIVE RATE	GPM
PUMP INTAKE SET AT	60 FEET
WATER AT END OF TEST	1 ☒ CLEAR 2 ☐ CLOUDY
RECOMMENDED PUMP TYPE	☐ SHALLOW ☒ DEEP
RECOMMENDED PUMP SETTING	60 FEET
RECOMMENDED PUMPING RATE	12+ GPM

FINAL STATUS OF WELL

1 ☒ WATER SUPPLY
2 ☐ OBSERVATION WELL
3 ☐ TEST HOLE
4 ☐ RECHARGE WELL
5 ☐ ABANDONED, INSUFFICIENT SUPPLY
6 ☐ ABANDONED, POOR QUALITY
7 ☐ UNFINISHED
9 ☐ DEWATERING

WATER USE

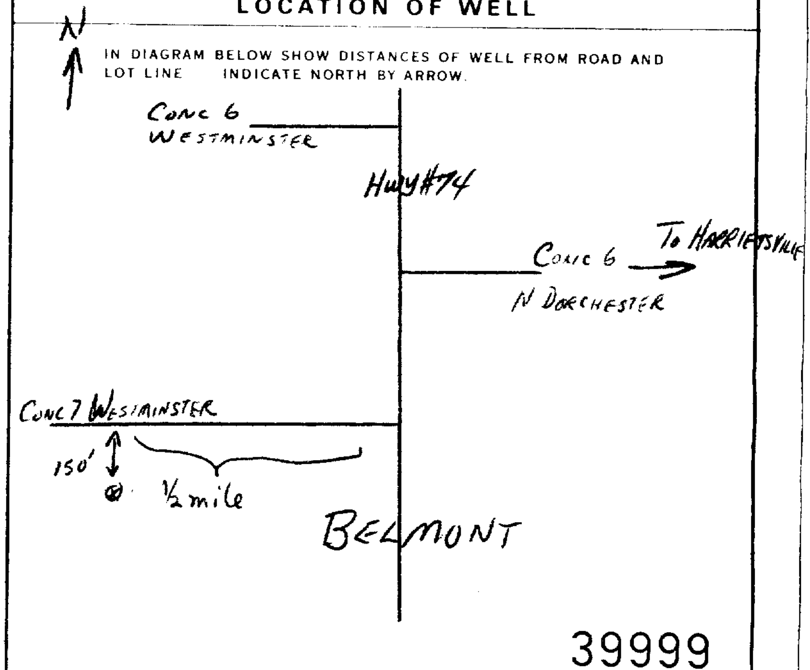
1 ☒ DOMESTIC
2 ☒ STOCK
3 ☐ IRRIGATION
4 ☐ INDUSTRIAL
☐ OTHER Crop Spraying
5 ☐ COMMERCIAL
6 ☐ MUNICIPAL
7 ☐ PUBLIC SUPPLY
8 ☐ COOLING OR AIR CONDITIONING
9 ☐ NOT USED

METHOD OF CONSTRUCTION

1 ☐ CABLE TOOL
2 ☐ ROTARY (CONVENTIONAL)
3 ☐ ROTARY (REVERSE)
4 ☒ ROTARY (AIR)
5 ☐ AIR PERCUSSION
6 ☐ BORING
7 ☐ DIAMOND
8 ☐ JETTING
9 ☐ DRIVING
☐ DIGGING
☐ OTHER

LOCATION OF WELL

IN DIAGRAM BELOW SHOW DISTANCES OF WELL FROM ROAD AND LOT LINE. INDICATE NORTH BY ARROW.



39999

DRILLERS REMARKS

CONTRACTOR

NAME OF WELL CONTRACTOR	T.B. Wilson & Son Ltd
WELL CONTRACTOR'S LICENCE NUMBER	5466
ADDRESS	RR#1 Springfield
NAME OF WELL TECHNICIAN	P. Wilson
WELL TECHNICIAN'S LICENCE NUMBER	T-0362
SIGNATURE OF TECHNICIAN/CONTRACTOR	[signature]
SUBMISSION DATE	DAY ___ MO. ___ YR. ___

OFFICE USE ONLY

DATA SOURCE	CONTRACTOR	DATE RECEIVED
	5466	MAR 29 1989

DATE OF INSPECTION | INSPECTOR
REMARKS

CSS.S8

MINISTRY OF THE ENVIRONMENT COPY

FORM NO. 0506 (11/86) FORM 9

Appendix D – I & T 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 re-compacted 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 borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building 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.

COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to exp by its client ("Client"), communications between exp and the Client, other reports, proposals or documents prepared by exp for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. exp is not responsible for use by any party of portions of the Report.