Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

A REPORT TO
849413 ONTARIO LTD.

HYDROGEOLOGICAL ASSESSMENT PROPOSED RESIDENTIAL DEVELOPMENT

27-31 BLAKE STREET
CITY OF BARRIE

REFERENCE NO. 1809-W012

JULY 2021

(REVISION OF REPORT DATED NOVEMBER 2019)

DISTRIBUTION

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1.0 EXECUTIVE SUMMARY

Soil Engineers Ltd. conducted a hydrogeological assessment for a proposed residential development, located at 27-31 Blake Street, in the City of Barrie. Surrounding land use includes; Blake Street, and residential buildings to the north, east, south, and west of the site. The site is currently occupied by two (2) existing residential buildings which are currently occupied. It is proposed to construct a five (5) storey apartment building with 35 units, having a one (1) level underground parking structure at the site.

The subject site is located within the physiographic region of Southern Ontario known as the Simcoe Lowlands, which is is located on mapped Undifferentiated Till deposits, consisting predominantly of sandy silt to silt matrix, which is high in matrix calcium carbonate content which is considered as having moderate to high clast content.

The subject site is located within the Barrie Creek sub-watershed of the Lake Simcoe Watershed.

A review of the local topography shows that the subject site is relatively flat having a gentle decline in elevation relief towards its south limits.

The study has disclosed that beneath a layer of topsoil, and earth fill, the native soils underlying the subject site consists of silty sand till, sandy silt till, and fine to medium sand, extending to the maximum investigated depth of 6.5 m.

The findings of this current study confirm that the groundwater levels range from El. 228.90 to 232.41 masl (i.e., 0.99 to 2.70 m below ground surface). Review of the average of shallow groundwater level elevations suggests that it flows in a southerly direction, towards Lake Simcoe.

The single well response tests yielded hydraulic conductivity (K estimate) for the silty sand till is 4.0×10^{-7} m/s. The K estimate for the sandy silt till is 4.40×10^{-5} m/s, and the K estimate for the silty clay till, and medium sand, is 9.30×10^{-7} m/s. The above results suggest that the hydraulic conductivity for the groundwater-bearing sub-soils at the depths of the well screens is moderate to high, with corresponding moderate to high anticipated groundwater seepage rates into open excavations, below the water table.

The Hazen Equation calculated results indicates that the K estimate for the silty sand till, having traces of clay and gravel, retrieved from a depth of 4.8 mbgs at BH/MW 1 is 9.0×10^{-8} m/sec, the K estimate for the sandy silt till, having traces of clay, retrieved from a depth of 6.3 mbgs at BH/MW 2 is 4.84 x 10^{-6} m/sec, and the K estimate for the fine to medium sand retrieved from a depth of 4.8 mbgs at BH/MW 3 is 2.5 x 10^{-5} m/sec. The K estimate determined from the Hazen method suggests a low to high hydraulic conductivity (K) for any encountered shallow perched groundwater found beneath the subject site.

The groundwater levels beneath the site are approximately 2.27 m above the proposed basement floor slab elevation for the proposed apartment building and the stormwater storage chamber, and they are 3.46 m above the proposed elevator pit structure.

The groundwater levels at the site range from approximately 0.06 to 1.02 m below, to 0.21 m below the invert levels for the proposed underground services.

The dewatering flow estimates for construction of the proposed apartment building, underground parking structure, including the stormwater storage chamber, suggests that it is about 106,797 L/day; by applying a safety factor of three (3), it could reach a maximum of 320,392 L/day. The dewatering flow estimates for the construction of the proposed elevator pit structure suggests that the rate is about 10,597 L/day; by applying a safety factor of three (3), it could reach a maximum of 31,790 L/day. These anticipated dewatering rates for earthworks excavation are below the PTTW threshold limit of 400,000 L/day but are above 50,000 L/day groundwater taking approval requirement threshold, whereby the approval for the proposed water takings for construction dewatering program to complete the basement structures would be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP.

The dewatering flow estimates for the installation of the underground services suggests that they could range from between 2,753 L/day and 40,911 L/day; by applying a safety factor of three (3), they could reach maximums of between 8,074 L/day and 120, 759 L/day. Construction dewatering rates that are below the 50,000 L/day limit threshold will not require any registration or filing with the MECP; construction dewatering rates that are below the PTTW threshold of 400,000 L/day, but are above the 50,000 L/day groundwater taking approval requirement threshold, will be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP.

The estimated zone of influence for construction dewatering could reach a maximum of 88.8 m away from the conceptual dewatering alignments around the proposed building footprint. There are existing neighbouring residential properties that are within the conceptual zone of influence for construction dewatering; however, no groundwater receptors, such as water wells, bodies of water, watercourses or wetlands are present within the conceptual zone of influence for construction dewatering for the proposed development. The local shallow groundwater flow pattern may be temporarily affected during construction.

It is anticipated to collected any anticipated effluent, within a temporary storage tank for later disposal management off site at an MECP approved receiving facility.

The long-term foundation drainage rates from both an under-slab basement floor drainage network and from a mira wall drainage network for a conventionally shored excavation foundation for the proposed apartment building underground parking structure, is approximately 73,273.51 L/day. By applying a safety factor of three (3), the anticipated drainage flow rates could reach a maximum of 219,820.53 L/day.

The Long-term foundation drainage rates from both an under-slab floor drainage network and from a mira drainage network for a conventionally shored excavation foundation, and for the proposed elevator pit structure for the apartment building is approximately 7,956.35 L/day. By applying a safety factor of three (3), the foundation drainage flow rates could reach a maximum of 23,869.05 L/day.

It is our understanding that the proposed underground foundation structure will be built completely waterproof, to cut off any groundwater seepage to the excavation and completed underground structure, with no connection being needed to the City's Sewer System, as no longer foundation drainage is anticipated.

The groundwater levels lie at depths, ranging from between 0.99 to 2.70 m below the existing ground surface. As such passive LID measures such as implementation of bioswales, rain gardens and the thickening topsoil should be considered to divert storm runoff away from the municipal storm sewers, and to recharge groundwater table where possible, to address future stormwater management planning for the proposed development.

2.0 INTRODUCTION

2.1 Project Description

In accordance with authorization, dated August 31, 2018, from 849413 Ontario Ltd., Soil Engineers Ltd. (SEL) conducted a hydrogeological assessment for a proposed residential development site located at 27-31 Blake Street, in the City of Barrie. The location of the subject site is shown on Drawing No. 1.

Surrounding land use includes; Blake Street, and residential buildings to the north, east, south, and west of the site. The site is currently occupied by two (2) existing residential buildings which are currently occupied. It is proposed to construct a five (5) storey condominium apartment building with 35 units, having a one (1) level underground parking structure at the site.

This report summarizes the findings of the field study and the associated groundwater level monitoring and hydraulic testing programs, providing a description and characterization of the hydrogeostratigraphy for the site and local surrounding area. The current study provides preliminary recommendations for any construction dewatering needs, and for any anticipated long-term foundation drainage needs prior to detailed design.

Furthermore, the report provides a recommendation for any need to acquire an Environmental Activity and Sector Registry (EASR) approval, or a Permit-To-Take Water (PTTW) as approvals, to facilitate groundwater taking for any anticipated construction dewatering program, or for any anticipated long-term foundation drainage needs.

2.2 Project Objectives

The major objectives of this Hydrogeological Study Report are as follows:

1. Establish the local hydrogeological setting for the site and surrounding areas;
2. Interpretation of shallow groundwater flow and runoff patterns;
3. Identify zones of higher groundwater yield as potential sources for ongoing shallow groundwater seepage;
4. Characterizing the hydraulic conductivity (K) for the groundwater-bearing sub-soil strata;
5. Prepare an interpreted hydrostratigraphic cross-section across the development footprint and the subject site;

6. Estimate the anticipated dewatering flows that may be required to lower the shallow water table to facilitate earthworks construction, or for any required long-term foundation drainage needs following construction;
7. Evaluate potential impacts to any nearby groundwater receptors within the anticipated zone of influence for construction dewatering; and to develop preliminary estimates for any temporary dewatering flow rates that may be required to facilitate excavation for construction, or for any long-term foundation drainage needs.
8. Comment on the feasibility of the site to accommodate any Low Impact Development (LID) infrastructure to address future stormwater management planning.

2.3 Scope of Work

The scope of work for the Hydrogeological Study is summarized below:

1. Borehole drilling and installation of three (3) monitoring wells within the site's development footprint;
2. Monitoring well development and groundwater level measurements at the three (3) installed monitoring wells;
3. Performance of Single Well Response Tests (SWRTs) at the installed monitoring wells to estimate the hydraulic conductivity (K) for the groundwater-bearing subsoils at the depths of the well screens;
4. Describing the geological and hydrogeological setting for the subject site and the surrounding local area; and,
5. Estimating the hydraulic conductivity (K) for the groundwater bearing subsoil strata, based on the SWRT results and from the soil grain size analyses.
6. Review of the findings of the concurrent geotechnical study; review of available engineering development plans and profiles for the proposed residential development; assessing preliminary dewatering needs, and estimation of any anticipated dewatering flows to lower the groundwater levels for construction, or for any anticipated long-term foundation drainage.

3.0 METHODOLOGY

3.1 Borehole Advancement and Monitoring Well Installation

Borehole drilling and monitoring well construction were conducted on October 2, 2018. The program comprised the drilling of three (3) boreholes (BH) and the installation of three (3) monitoring wells, one in each of three (3) boreholes advanced beneath the site. The locations of the boreholes/monitoring wells are shown on Drawing No. 2.

The borehole drilling and monitoring well construction were completed by a licensed water well contractor, DBW Drilling Ltd., under the full-time supervision of a geotechnical technician from SEL, who also logged the soil sub-strata encountered during borehole advancement, and collected representative soil samples for textural classification. The boreholes were drilled using continuous flight power augers. Detailed descriptions of the encountered subsurface soil and groundwater conditions are presented on the borehole and monitoring well logs, on the enclosed Figures 1 to 3, inclusive.

The monitoring wells were constructed using 50-mm diameter PVC riser pipes and screens, which were and installed in each of the boreholes in accordance with Ontario Regulation (O. Reg.) 903. All of the monitoring wells were provided with monument-type surface protective steel casings at the ground surface. The details of the monitoring well construction are provided on the enclosed Borehole Logs (Figures 1 to 3, inclusive).

The UTM coordinates and ground surface elevations at the borehole/monitoring well locations, together with the monitoring well construction details, are provided on Table 3-1.

Table 3-1 - Monitoring Well Installation Details

Well ID	Installation Date	UTM Coordinates		Ground El. (masl)	Monitoring Well Depth (mbgs)	Screen Interval (mbgs)	Casing Dia. (mm)
		East (m)	North (m)				
BH/MW 1	October 2, 2018	664613.81	4870424.05	233.4	6.1	3.1 – 6.1	50
BH/MW 2	October 2, 2018	664588.50	4870494.67	232.4	6.1	3.1 – 6.1	50
BH/MW 3	October 2, 2018	664700.71	4870450.79	231.6	6.1	3.1 – 6.1	50

Notes:

- mbgs -- metres below ground surface - masl -- metres above sea level

3.2 Groundwater Monitoring

The groundwater levels in the monitoring wells were measured, manually on October 16, 24 and 31, 2018, and again on April 23, and May 21, 2019 to record the spring high groundwater table beneath the site.

3.3 Mapping of Ontario Water Well Records

SEL received the Ministry of the Environment, Conservation and Parks (MECP) Water Well Records (WWRs) for registered well records located on the subject site and within 500 m of the site boundaries (study area). The records indicate that five (5) registered wells are located within a 500 m study area relative to the subject site boundaries. The WWR well locations are shown on Drawing No. 3, and a summary of the WWRs that were reviewed for this study are listed in Appendix 'A'.

3.4 Monitoring Well Development and Single Well Response Tests

All of the BH/MWs underwent development in preparation for the single well response testing (SWRT) to estimate the hydraulic conductivity (K) for subsoil strata at the depths of the well screens. Monitoring well development involved the purging and removal of several casing volumes of groundwater from each monitoring well to remove remnants of clay, silt, sand, and other debris introduced into the monitoring wells during construction, and to induce the flow of formation groundwater through the well screens, thereby improving the transmissivity of the subsoil strata at the well screen depths.

The K estimates derived from the SWRT's provide an indication of the yield capacity for the shallow groundwater-bearing subsoil strata at the well screen depths, and can be used to estimate the flow of groundwater through the water-bearing sub-soil.

The SWRT involves the placement of a slug of known volume into the monitoring well, below the water table, to displace the groundwater level upward. The rate at which the groundwater level recovers to static conditions (falling head) is tracked using a data logger/pressure transducer, and/or manually using a water level tape. The rate at which the groundwater table recovers to static conditions is used to estimate the K value for the groundwater-bearing sub-soil strata formation at the well screen depth.

All of the BH/MWs underwent SWRT's on October 24, 2018. The detailed test results are provided in Appendix 'B', with a summary of the findings provided in Table 6-2.

3.5 **Estimating Hydraulic Conductivity using the Hazen Equation Method**

The Hazen equation method was also used to estimate the hydraulic conductivity (K) for saturated subsoils at or below the anticipated groundwater level depths, beneath the subject site. The method provides alternative K estimates which are derived from the soil grain size diameter, whereby 10% by weight of the soil particles are finer and 90% are coarser (Freeze and Cherry, 1979). The results of the Hazen based estimates are discussed in Section 6.6.

4.0 REGIONAL AND LOCAL SETTING

4.1 Regional Geology

The subject sites lie within the Physiographic Region of Southern Ontario known as the Simcoe Lowlands, which covers an area of approximately 2,850 square kilometers. It lies at elevations ranging from between 177.0 masl and 259.0 masl. The area was flooded by the former glacial Lake Algonquin and is bordered by shore cliffs, beaches, and bouldery terraces. As such, the area is floored by lacustrine deposits sand, silt, and clay (Chapman and Putnam, 1984). The lowlands fall into two major subdivisions; the Nottawasaga Basin draining into Nottawasaga Bay (Georgian Bay) and Lake Simcoe Basin, which drains into Lake Simcoe (Chapman and Putnam, 1984). The site is mapped as being on sand plains deposits.

Review of the surface geological map of Ontario shows that the subject site is located on Undifferentiated Till deposits, consisting predominantly of sandy silt to silt matrix, which is high in matrix calcium carbonate content which is considered as having moderate to high clast content. Drawing No. 4, as reproduced from Ontario Geological Survey mapping, illustrates the quaternary surface soil geology for the subject site and surrounding areas.

The bedrock is comprised mainly of Middle Ordovician aged shale, limestone, dolostone, siltstone, and sandstone of the Ottawa and the Simcoe Group (Ontario Ministry of Northern Department and Mines, 1991). The approximate elevation for the top of the bedrock beneath the site is at about 122 m masl (Bedrock Topography of Barrie Area, 1974). At this elevation, the approximate depth of overburden soil to bedrock is about 109.6 to 113 m beneath the subject site.

4.2 Physical Topography

A review of the local topography shows that the subject site is relatively flat, exhibiting a gentle decline in elevation relief towards its south limits. Runoff from the site is expected to drain in a southerly direction towards Lake Simcoe. Based on the topographic map for the area, and from review of the ground surface elevations at the borehole and monitoring well locations, the elevation relief across the subject site is about 3.4 m. Drawing No. 5 shows the mapped topographical contours for the site and surrounding area.

4.3 Watershed Setting

The subject site is located within the Barrie Creek Sub-watersheds of the Lake Simcoe Watershed. The Lake Simcoe Watershed comprises a total land and water surface area of 3,324 km^2, of which the lake occupies about 20 percent, or 722 km^2. The land portion of the watershed is approximately 2,600 km^2 which is drained by 35 tributary creeks and rivers, with five major tributaries accounting for more than 60 percent of the total drainage area. The Lake Simcoe Watershed has been divided into 18 sub-watersheds, or hydrological units (excluding Lake Simcoe Islands) (LSRCA).

The Barrie Creeks sub-watershed occupies an area of approximately 37.53 km^2. The watershed area in recent times has been impacted by a rapid growth in development, especially within the City of Barrie, and has the lowest percentage of natural vegetative cover in the Lake Simcoe Watershed.

Drawing No. 6 shows the location of the subject site within the Barrie Creek Sub-watershed.

4.4 Local Surface Water and Natural Features

Lake Simcoe is located approximately 120 m south of the site, and a tributary of Lake Simcoe is located approximately 1,125 m west of the site, where it flows in a north to south direction, before emptying into the Lake. Scattered wooded areas are located about 75 m north of the site.

The locations of the site and the noted natural features are shown on Drawing No. 7.

5.0 SOIL LITHOLOGY

The study has disclosed that beneath a layer of topsoil, and earth fill, the native soils underlying the subject site consists of silty sand till, sandy silt till, and fine to medium sand, extending to the maximum investigated depth of 6.5 m. A Key Plan, and the interpreted geological cross-sections along the delineated northwest-southeast, and east-west transects are presented on Drawing Nos. 8-1 and 8-2.

5.1 **Topsoil** (All BH/MWs)

Topsoil, approximately 20 cm thick, was observed beneath the ground surface at at all of the BH/MWs locations.

5.2 **Earth Fill** (BH/MWs 2 and 3)

Earth fill, approximately 0.6 m thick, was observed beneath the topsoil horizon at the BH/MWs 2 and 3, locations. The fill is brown in colour and consists of silty clay and silty sand, having traces of gravel and root inclusions.

5.3 **Silty Sand Till** (All BH/MWs)

Silty sand till was encountered at depths, ranging between 0.8 mbgs and 2.3 mbgs, at all the BH/MWs locations. It is brown in colour at the BH/MW 3, location and is grey at BH/MWs 1 and 2 locations. It is compact to very dense in consistency, having traces of clay and gravel. The thickness of the unit ranges from 2.2 to 2.3 m at BH/MWs 2 and 3, respectively, where it extends from a depth of 1.5 m to the maximum investigated depth of 6.5 m at the BH/MW 1, location. The moisture content for the silty clay unit ranges from 7% to 17%, indicating damp to moist conditions.

The estimated permeability for the silty sand till unit encountered at the BH/MW 1 location, at a depth of 4.8 mbgs is about 10^{-7} m/sec. Grain size analysis was performed on one (1) sample, and the gradation is plotted on Figure 4.

5.4 **Sandy Silt Till** (BH/MW 2)

Sandy silt till was encountered at BH/MW 2 at a depth of 3.0 mbgs, at the BH/MW 2, location. It is grey in colour and is very dense in consistency, having traces of clay and gravel. This unit was encountered at a depth of 3.0 m, where it extends to the maximum investigation depth of 6.5 m. The moisture content for the sandy silt till unit ranges from 7% to 18%, indicating damp to moist conditions.

The estimated permeability for the sandy silt unit encountered at the BH/MW 2 location, at a depth of 6.3 mbgs is about 10^{-5} m/sec. Grain size analysis was performed on one (1) sample, and the gradation is plotted on Figure 5.

5.5 **Sand** (BH/MW 3)

Sand was encountered at the BH/MW 3 location. An upper unit consisting of fine sand was encountered at a depth of 0.2 m beneath the topsoil unit. It is brown in colour, is loose to compact in consistency, and is approximately 2.21 m thick. The moisture content for the sand unit ranges from 10% to 15%, indicating damp conditions.

Another sand unit consisting of fine to medium sand, was encountered at a depth of 4.6 m. It is grey in colour, and very dense in consistency, having some silt, traces of clay, and coarse sand and gravel. This sand unit extends to the maximum investigated depth of 6.5 m. The moisture content for this sand unit ranges from 17% to 19%, indicating moist conditions.

The estimated permeability for the fine to medium sand unit, at a depth of 4.8 mbgs is about 10^{-5} m/sec. Grain size analysis was performed on one (1) sample, and the gradation is plotted on Figure 6.

6.0 GROUNDWATER STUDY

6.1 Review of Ontario Water Well Records

The Ministry of the Environment, Conservation and Parks (MECP) water well records for the subject site and for the properties within a 500 m radius of the boundaries of the subject site (study area) were reviewed.

The records indicate that five (5) well records are located within the study area relative to the boundaries of the subject site. The locations of these well records, based on the UTM coordinates provided by the records, are shown on Drawing No. 3. Details of the MECP water well records that were reviewed are provided in Appendix 'A'.

A review of the final status and of the well records within the study area reveals that two (2) are registered as test hole wells, one (1) is registered as an observation well, and two (2) wells are registered as having unknown statuses.

A review of the first use of the well records reveals that two (2) are registered as monitoring wells, and three (3) wells are registered as having unknown statuses.

6.2 Groundwater Monitoring

The groundwater levels in the monitoring wells were measured on three (3) occasions over the study period, on the following dates; October 16, 24 and 31, 2018, and again on April 23, and May 21, 2019, to record the fluctuation of the groundwater table beneath the site. The groundwater levels and their corresponding elevations are given in Table 6-1.

Table 6-1 - Groundwater Level Measurements

Well ID		October 16, 2018	October 24, 2018	October 31, 2018	April 23, 2019	May 21, 2019	Average	Fluctuation (m)
BH/MW 1	mbgs	1.93	2.00	1.94	0.99	1.38	1.65	1.01
	masl	231.47	231.40	231.46	232.41	232.02	231.75	
BH/MW 2	mbgs	2.10	2.17	1.72	1.30	1.85	1.83	0.87
	masl	230.30	230.23	230.68	231.10	230.55	230.57	
BH/MW 3	mbgs	2.70	2.39	2.28	1.62	1.88	2.17	1.08
	masl	228.90	229.21	229.32	229.98	229.72	229.43	

Notes: mbgs -- metres below ground surface masl -- metres above sea level

As shown above, all of the groundwater levels at BH/MWs 1 and 2, fluctuated, where they decreased between October 16 and 24, 2018, and increased again, between October 24, and April 23, 2019, and, decreased again between April 23, and May 21, 2019. The groundwater levels at BH/MW 3, showed an increasing trend between October 16, 2018, and April 23, 2019, and decreased, between April 23, and May 21, 2019.

The greatest fluctuation was observed at BH/MW 3, where the groundwater level increased by 1.08 m during the monitoring period.

6.3 Shallow Groundwater Flow Pattern

The shallow groundwater flow pattern was interpreted from the average of groundwater level measurements recorded in the BH/MWs. The measured levels indicate that shallow groundwater flows in a south-easterly directions from the north western portion of the site, towards Lake Simcoe. The interpreted shallow groundwater flow pattern for the site is illustrated on Drawing No. 9.

6.4 Single Well Response Test Analysis

All of the BH/MWs underwent single well response tests (SWRTs) to assess the hydraulic conductivity (K) for saturated shallow aquifer sub-soils at the depths of the well screens.

The results of the SWRTs are presented in Appendix 'B', with a summary of the findings shown in Table 6-2.

Table 6-2 - Summary of SWRT Results

Well ID	Ground El. (masl)	Monitoring Well Depth (mbgs)	Borehole Depth (mbgs)	Screen Interval (mbgs)	Screened Sub-Soil Strata	Hydraulic Conductivity (K) (m/sec)
BH/MW 1	233.40	6.10	6.50	3.1-6.1	Silty Sand Till	4.0×10^{-7}
BH/MW 2	232.40	6.10	6.50	3.1-6.1	Sandy Silt Till	4.4×10^{-5}
BH/MW 3	231.60	6.10	6.50	3.1-6.1	Silty Sand Till/ Medium Sand	9.3×10^{-7}

Notes: mbgs -- metres below ground surface masl -- metres above sea level

As shown above, the K estimate for the silty sand till is 4.0×10^{-7} m/s. The K estimate for the sandy silt till is $4.40 \text{ x } 10^{-5}$ m/s, and the K estimate for the silty clay till, and medium sand, is 9.30×10^{-7} m/s. The above results suggest that the hydraulic conductivity for the groundwater-bearing sub-soils at the depths of the well screens is moderate to high, with corresponding moderate to high anticipated groundwater seepage rates into open excavations, below the water table.

6.5 Assessment of Hydraulic Conductivity Based on the Hazen Equation

The Hazen Equation method was also adopted to estimate the hydraulic conductivity (K) for different subsoil layers which may contain high groundwater levels during the seasonal (spring) period, or if encountered within the deeper excavations. These subsoil layers are primarily above the well screen depths.

The Hazen Equation method relies on the interrelationship between hydraulic conductivity and effective soil particle grain size, d_{10}, (mm) for the sub-soil media as determined from soil grain size analysis. This empirical relation predicts a power-law relation with K, as follow:

$$K = Ad_{10}^2$$

where;

d_{10}: Value of the soil grain size gradation curve (mm) as determined by sieve analysis, whereby 10% by weight of the soil particles are finer and 90% by weight of the soil particles are coarser.

A: Coefficient; it is equal to 1 when K is in m/sec and d_{10} is in mm

The Hazen Equation estimation provides an indication of the yield capacity for groundwater-bearing sub-soil strata at the depths where the soil samples that underwent grain size analyses were collected from. The calculated results indicate that the K estimate for the silty sand till, having traces of clay and gravel, retrieved from a depth of 4.8 mbgs at BH/MW 1 is 9.0 x 10^{-8} m/sec, the K estimate for the sandy silt till, having traces of clay, retrieved from a depth of 6.3 mbgs at BH/MW 2 is 4.84 x 10^{-6} m/sec, and the K estimate for the fine to medium sand retrieved from a depth of 4.8 mbgs at BH/MW 3 is 2.5 x 10^{-5} m/sec.

The results of the Hazen method determined K estimates are provided in Table 6-3 below. The K estimate determined from the Hazen method suggests a low to high hydraulic conductivity (K) for the groundwater bearing sub-soil layers beneath the subject site.

Table 6-3 - Summary of Hazen Equation Estimated K Results

Well ID	Soil Sample Depth (mbgs)	Sample El. (masl)	Description of Soil Strata	D_{10} (mm)	Hydraulic Conductivity (K) (m/sec)
BH/MW 1	4.8	228.6	Silty Sand Till, traces of Clay and Gravel	0.003	9.0×10^{-8}
BH/MW 2	6.3	226.1	Sandy Silt Till, traces of Clay and Gravel	0.022	4.84×10^{-6}
BH/MW 3	4.8	226.8	Fine to Medium Sand, some Silt, traces of Clay, Coarse Sand and Gravel	0.05	2.50×10^{-5}

Notes: mbgs -- metres below ground surface masl -- metres above sea level
D10 – diameter (mm) of soil grain size at 10% fine, 90% coarse

7.0 GROUNDWATER CONTROL

The hydraulic conductivity (K) estimates for the silty sand till, sandy silt till, silty sand, and the fine to medium sand units, suggest that groundwater seepage rates into open excavations below the groundwater table will range from low to high. To provide safe, dry and stable conditions for earthworks excavations for construction of the proposed 1-level underground parking and basement structures, the groundwater table should be lowered in advance of, or, during construction. The preliminary estimates for construction dewatering flows required to locally lower the water table, based on the SWRT, K test estimates, are discussed in the following sections.

7.1 Groundwater Construction Dewatering Rates

The proposed development plans, provided by Studio K Architects, Drawing No. A302, dated January 8, 2019, indicate that it is planned to construct a five (5) storey, residential building having a 1-level underground parking structure. The proposed development footprint encompasses an area of approximately 2,422 square meters.

Five (5) Storey Apartment/Condominium Building Construction – 1-Level Underground Parking Structure (56.39 m x 43.0 m) with a Finished Floor Elevation of 231.1 masl:

For the proposed five (5) storey apartment condominium block building, the site grade elevation is approximately 235.0 masl. For the preliminary dewatering calculations, the estimated area of excavation for the proposed 1-level underground parking structure is approximately 2,422 square meters which is approximately 56.39 m long by 43.0 m wide, having a perimeter of approximately 198.78 m. An elevation of 231.1 masl was provided for the finished floor basement slab elevation. An additional excavation depth of 0.96 m (El. 230.14 masl) was considered to accommodate the proposed underground parking level structure and footings which were considered for dewatering needs assessment. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the shallow groundwater table be lowered to an elevation of 229.14 masl, which is about 1 m below the lowest proposed excavation depth. The subsoil comprises topsoil, earth fill, silty sand till, and fine sand, extending to the maximum proposed depths for excavation. Comparison of the lowest proposed excavation depth with the highest measured shallow groundwater level indicates that the lowest proposed excavation depth is about 2.27 m below the highest measured shallow groundwater level elevation of 232.41 masl, as recorded at the BH/MW 1, location. By having the anticipated groundwater table lowered by one (1) additional meter, it is anticipated that construction dewatering will be required in support of the proposed earthworks for construction of this apartment building and the associated underground parking structure.

Assuming an excavation, being approximately 56.39 m long by 43 m wide for the proposed underground parking structure, having a perimeter of about 198.78 m, and using the estimated hydraulic conductivity of 4.4 x 10^{-5} m/s, the anticipated construction dewatering flow rate could reach an estimated daily rate of 110,909 L/day. By applying a safety factor of three (3), it could reach a maximum of 332, 726 L/day. The estimated zone of influence could extend to a maximum of 65.1 m away from the conceptual dewatering array around the excavation footprint for the underground parking structure.

This dewatering flow rate for excavation is below the PTTW threshold limit requirement of 400,000 L/day, but is above 50,000 L/day limit for requiring an approval, whereby the approval for the proposed groundwater takings would be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP. This higher dewatering flow estimate may only occur at the beginning of the dewatering process, and includes any rapid removal of collected runoff within the excavation after a high intensity storm event. It is anticipated that, following the lowering of the localized water table, groundwater seepage removed via dewatering from the open excavation will be a fraction of the above estimate, since much of the groundwater within the proposed construction servicing trenches areas will have been removed from local storage. Furthermore, upon excavation for, and if encountered, any perched groundwater within the shallow fill horizons is expected to dissipate relatively quickly following commencement of earthworks and excavations. If construction is completed during the dry season (Summer), there may be only minimal or negligible construction dewatering required as the shallow perched groundwater conditions may not be present during the dry season, typically expected between mid-July through mid-October.

Installation of Elevator Pit Beneath the Apartment/Condominium Building at an elevation of 228.95 masl:

The estimated finished floor elevation for the proposed underground parking structure is at 231.10 masl. An excavation depth of approximately 2.15 m (El. 228.95 masl) below the proposed elevation for the underground parking structure was considered for the proposed elevator pit construction. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 227.95 masl, which is about 1 m below the lowest proposed excavation depth. The subsoil at this depth is comprised of earth fill, silty sand till, sandy silt till, and sand extending to the lowest proposed excavation depth. Comparison of the lowest proposed excavation depth with the highest groundwater level of 232.41 masl, as measured at the BH/MW 1, location, indicates that the proposed elevation for the elevator pit footing is about 3.46 m below the highest shallow groundwater level. By having the anticipated groundwater table lowered by one (1) additional meter, it is anticipated that construction dewatering will be required for the proposed earthworks for construction of this portion of the apartment/condominium building.

Assuming an excavation, being approximately 4 m long by 4 m wide for the proposed elevator pit structure, having a perimeter of about 16 m, and using the estimated hydraulic conductivity of 4.4×10^{-5} m/s, the anticipated dewatering flow rate could reach an estimated daily rate of 10,597 L/day. By applying a safety factor of three (3), the dewatering flow rate could reach a maximum of 31,790 L/day. The estimated zone of influence could extend to a maximum of 88.8 m away from the conceptual dewatering array being considered for construction of the proposed elevator pit structure.

This dewatering flow rate for excavation is below the PTTW threshold limit requirement of 400,000 L/day, but is above 50,000 L/day limit threshold for requiring an approval, whereby the approval for the proposed water takings would be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP. This higher dewatering flow estimate may only occur at the beginning of the dewatering process, and includes any rapid removal of collected runoff within the excavation after a high intensity storm event. It is anticipated that, following the lowering of the localized water table, groundwater seepage removed via dewatering from the open excavation will be a fraction of the above estimate, since much of the groundwater within the proposed construction servicing trenches areas will have been removed from local storage. Furthermore, upon excavation for, and if encountered, any perched groundwater within the shallow fill horizons is expected to dissipate relatively quickly following commencement of earthworks and excavations. If construction is completed during the dry season (Summer), there may be only minimal or negligible construction dewatering required as the shallow perched groundwater conditions may not be present during the dry season, typically expected between mid-July through mid-October.

Installation of underground services:

The site servicing plans provided by RV Santos and Associates Limited; Servicing Plan, Drawing No. G-2, dated May 2019, indicate that the proposed underground services will be installed to depth elevations, ranging from 232.88 masl beneath the northern portion of the site, to 230.81 masl beneath the southern portion of the site.

Installation of underground services within the Northern Portion of site at an elevation of 232.20 masl:

The dewatering needs assessment was based on the lowest proposed servicing depth elevations, being approximately at 232.20 masl. The highest shallow groundwater elevation is at 232.41 masl, as measured at the BH/MW 1, location. Based on the current assessment, the subsoil underlying the subject site consists of earth fill, extending to the proposed depth for underground services installations. Comparison of the lowest proposed excavation depth with the highest measured shallow groundwater level indicates that the lowest proposed excavation elevation is about 0.21 m above the shallow groundwater level. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the water table be lowered to an elevation of 231.20 masl, which is about 1.0 m below the lowest proposed servicing invert excavation depth. A maximum anticipated groundwater level drawdown of 1.21 m will be needed to facilitate service trench excavation in this area.

Based on a 20 m length service trench excavation being open at any time, and using the estimated hydraulic conductivity of 4.40×10^{-5} m/s, the estimated dewatering flow rate is anticipated to reach a daily rate of 40,253 L/day; by applying a safety factor or three (3), it could reach an approximate daily maximum of 120,759 L/day. The estimated zone of influence could extend to a maximum of 24.1 m away from the conceptual dewatering alignment for underground services installation in this area.

This dewatering flow rate for excavation, is below the PTTW threshold of 400,000 L/day but is above 50,000 L/day threshold limit for requiring an approval, with the approval for proposed water takings being required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP. This higher dewatering flow estimates may only occur at the beginning of the dewatering process, which includes any rapid removal of collected runoff after a high intensity storm. It is anticipated that, following lowering of the localized water table, groundwater seepage removed via dewatering from the open trench excavation will be a fraction of the above estimate, since much of the groundwater in the proposed construction alignment areas will have been removed from local storage. Furthermore, upon excavation for, any encountered perched groundwater within the shallow fill horizons is expected to dissipate relatively quickly following commencement of earthworks. If construction is completed during the dry season (Summer), there may be only minimal or negligible construction dewatering required as shallow perched groundwater conditions may not be present during the dry season, typically expected between mid-July through mid-October.

Installation of underground services within the southern portion of site at an elevation of 230.81 masl:

The dewatering needs assessment was based on the lowest proposed servicing depth elevation, being approximately at 230.81 masl. The highest shallow groundwater elevation is at 229.98 masl, as measured at the BH/MW 3 location. Based on the current assessment, the subsoil underlying the subject site consists of topsoil, and fine sand, extending to the proposed depth for underground services installations. Comparison of the lowest proposed excavation depth with the highest measured shallow groundwater level indicates that the lowest proposed excavation elevation is about 0.83 m above the measured shallow groundwater level. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the water table be lowered to an elevation of 229.81 masl, which is about 1.0 m below the lowest proposed servicing invert excavation depth. A maximum anticipated groundwater level drawdown of 0.17 m will be needed to facilitate service trench excavation in this area.

Based on a 20 m length service trench excavation being open at any time, and using the estimated hydraulic conductivity of 4.40×10^{-5} m/s, the estimated dewatering flow rate is anticipated to reach a daily rate of 27,651 L/day; by applying a safety factor or three (3), it could reach an approximate daily maximum of 82,952 L/day. The estimated zone of influence could extend to a maximum of 3.4 m away from the conceptual dewatering alignment for underground services installation in this area.

This dewatering flow rate for excavation, is below the PTTW threshold of 400,000 L/day but is above 50,000 L/day threshold limit for requiring an approval, with the approval for proposed water takings being required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP. This higher dewatering flow estimates may only occur at the beginning of the dewatering process, which includes any rapid removal of collected runoff after a high intensity storm. It is anticipated that, following lowering of the localized water table, groundwater seepage removed via dewatering from the open trench excavation will be a fraction of the above estimate, since much of the groundwater in the proposed construction alignment areas will have been removed from local storage. Furthermore, upon excavation for, any encountered perched groundwater within the shallow fill horizons is expected to dissipate relatively quickly following commencement of earthworks. If construction is completed during the dry season (Summer), there may be only minimal or negligible construction dewatering required as shallow perched groundwater conditions may not be present during the dry season, typically expected between mid-July through mid-October.

Installation of Sanitary Manholes – Northwestern Portion of Site to a Depth Elevation of 232.88 masl:

It is proposed to install a sanitary manhole- MH1A, at the northwestern portion of the site. Excavation required for the construction of MH1A is approximately 1.5 m in diameter, and the manhole is proposed to be installed to an elevation of 232.88 masl (an approximate depth of 2.07 m).

Comparison of the proposed sanitary manhole with the measured groundwater levels indicates that the high groundwater elevation of 232.41 masl as measured at BH/MW 1 is about 0.47 m above the base elevation for the manhole. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to elevation of 231.88 masl for excavation in the vicinity of the sanitary manhole, which is about 1 m below the lowest proposed excavation depth. Based on the current assessment, the subsoil underlying the subject site consists of topsoil, and fine sand, extending to the proposed depth for underground services installations. By having the anticipated groundwater table lowered by one (1) additional meter, it is anticipated that construction dewatering will be required for the proposed earthworks for construction of the installation of the proposed manhole.

Assuming an excavation, being approximately 2 m long by 2 m wide for the construction of the proposed sanitary manhole structure, having a perimeter of about 8 m, and using the estimated hydraulic conductivity of 4.4×10^{-5} m/s, the anticipated dewatering flow rate could reach an estimated daily rate of 4,091 L/day. By applying a safety factor of three (3), the dewatering flow rate could reach a maximum of 12,273 L/day. The estimated zone of influence could extend to a maximum of 10.5 m away from the conceptual dewatering array being considered for installation of the proposed sanitary manhole structure.

This construction estimation dewatering rate for excavation is below the 50,000 L/day limit threshold for requiring an approval for any proposed construction related groundwater takings, which will not require any registration or filing with the MECP.

It is anticipated that, following the localized lowering of the groundwater table, the groundwater seepage removal via dewatering from the open excavation will be a fraction of the above estimate, since much of the shallow groundwater in the proposed development footprint area will have been removed from local storage. If construction is completed during the dry season (late Summer and early Fall), this might minimize the construction dewatering requirements as the groundwater levels are anticipated to be significantly lower during the dry season, typically expected between mid-July through mid-October.

Installation of Sanitary Manholes to Depth Elevations ranging between 230.67 masl and 230.04 masl at the Southern Portion of the Site

It is proposed to install several sanitary manholes along the southern limits of the site. The depth elevations for the manholes range from 230.67 masl at the south western limits at MH 4, to 230.04 masl at MH5 at its southeastern limits. The manholes are each anticipated to be approximately 1.5 m in diameter.

Comparison of the proposed depth elevations for the sanitary manholes with the measured groundwater levels indicates that the high groundwater level elevation could range from between 229.98 masl, as measured at BH/MW 3 which is about 0.06 to 0.69 m below the proposed base elevations for the manholes. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to depths ranging between 229.04 masl and 229.67 masl for excavations in the vicinity of the sanitary manholes, which are about 1 m below the lowest proposed excavation depths. Based on the current assessment, the subsoil underlying the subject site consists of topsoil, and fine sand, extending to the proposed depths for underground services installations. By having the anticipated groundwater table lowered by one (1) additional meter, it is anticipated that construction dewatering will be required for the proposed earthworks for installation of the proposed manholes.

Assuming excavations, being approximately 2 m long by 2 m wide for the construction of the proposed sanitary manhole structures, having a perimeter of about 8 m, and using the estimated hydraulic conductivity of 4.4×10^{-5} m/s, the anticipated dewatering flow rate could reach estimated daily rates of between 2,691 L/day and 2,753 L/day. By applying a safety factor of three (3), the dewatering flow rate could reach maximums of between 8,074 L/day and 8,260 L/day. The estimated zones of influence could extend to about 6.2 to 18.7 m away from the conceptual dewatering array being considered for installation of the proposed sanitary manhole structure.

These estimated construction dewatering rates for excavation are below the 50,000 L/day limit threshold for requiring an approval for any proposed construction related groundwater takings, which will not require any registration or filing with the MECP.

It is anticipated that, following the localized lowering of the groundwater table, the groundwater seepage removal via dewatering from the open excavation will be a fraction of the above estimate, since much of the shallow groundwater in the proposed development footprint area will have been removed from local storage. If construction is completed during the dry season (late Summer and early Fall), this might minimize the construction dewatering requirements as the groundwater levels are anticipated to be significantly lower during the dry season, typically expected between mid-July through mid-October.

Installation of the Stormwater Storage Chamber within the Northeastern Portion of the Underground Parking Structure at an Elevation of 231.10 masl:

The estimated construction footprint for the proposed stormwater chamber within the northeastern portion of the underground parking structure is approximately 60.0 square meters, with the bottom of the chamber being proposed at an elevation of 231.10 masl. The stormwater storage chamber is anticipated to be a pre-cast concrete structure, which is enclosed at the bottom, with its base anticipated to be at the finished floor elevation of the underground parking structure. Given that the stormwater storage chamber is at the same elevation of the underground parking structure, and is within the footprint for the underground parking structure, no additional dewatering anticipated for its installation, as this dewatering estimate has already been accounted for in the dewatering calculations for the underground parking structure.

It is anticipated to collected any short-term construction dewatering effluent, within a temporary storage tank, for later disposal management off site at an MECP approved receiving facility during construction.

7.2 <u>Groundwater Control Methodology</u>

Given that moderate to high groundwater seepage rates are being anticipated into open excavations below the water table. Short term construction dewatering for excavation of small servicing trenches, can likely be controlled by occasional pumping from sumps when and where required during construction. Well points can be considered to lower the water table if wet sand or unstable soils are encountered and seepage cannot be controlled via sump pumping. The final design for the dewatering system will be the responsibility of the construction contractors.

Tables 7-1 and 7-2 which follows, summarizes the dewatering flow rate estimates for the proposed residential structures, the underground services and stormwater storage chamber, respectively.

Table 7-1 - Summary of Dewatering Flow Estimates-Apartment Building

Residential Block Development/Structure	Anticipated Unit Type	Finished Floor Elevation (masl)	Area (square meters)	Depth Elevation for Underground Parking/Elevator Pit Structures (masl)	Highest Interpreted Water Level Elevation (masl)	Groundwater Elevation from Nearest BH/MW	Anticipated Groundwater Level Drawdown for Construction Dewatering (m)	Estimated Zone of Influence (m)	Dewatering Flow Estimates (L/day)	Flow Estimates with x 3 Safety Factor (L/day)
Underground Parking for Condominium/Apartment Building (Including Stormwater Storage Chamber)	5- Storey Building, with 1-Level Underground Parking	231.10	2,422	230.14	232.41	BH/MW 1	3.27	65.1	106,797	320,392
Elevator Pit Structure		-	8	228.95	232.41	BH/MW 1	4.46	88.8	10,597	31,790

Notes:
masl -- metres above sea level

Table 7-2 - Summary of Dewatering Flow Estimates-Proposed Underground Services Construction

Site Area	Lowest Invert Elevation (masl)	Highest Interpreted Water Level Elevation (masl)	Estimated Zone of Influence (m)	Anticipated Groundwater Level Drawdown for Construction Dewatering (m)	Dewatering Flow Estimates (L/day)	Flow Estimates with x 3 Safety Factor (L/day)
Underground Services – North portion of Site	232.20	232.41 (BH/MW 1)	24.1	1.21	40,253	120,759
Underground Services – South portion of Site	230.81	229.98 (BH/MW 3)	3.4	0.17	27,651	82,952
Sanitary Manhole Northwest portion of site- MH 1A	232.88	232.41 (BH/MW 1)	10.5	0.53	4,091	12,273
Sanitary Manhole Southwest to Southeast portion of site- MHs 4-5	230.04-230.67	229.98 (BH/MW3)	6.2- 18.7	0.31-0.94	2,691-2,753	8,074-8,260
Stormwater Storage Chamber within the Northeast portion of the Underground Parking Garage	231.10	No Additional Dewatering Anticipated				

Notes: masl -- metres above sea level

7.3 <u>Mitigation of Potential Impacts Associated with Dewatering</u>

The zone of influence for any dewatering well or dewatering array used during construction, could range between 3.4 m and 88.8 m away from the conceptual dewatering array wells or sump pits alignment around the excavation footprint for the proposed development portions of the site. No private supply wells, bodies of water, watercourses, wetlands or any natural features are present within the conceptual zone of influence for temporary construction dewatering.

The subject site is located within an existing developed existing residential area, which is surrounded by adjacent buildings which could potentially be affected by ground settlement associated with the zone of influence for any construction dewatering. A geotechnical engineer should be consulted to review potential ground settlement concerns prior to construction.

7.4 Permanent Drainage for Underground Structures

The development plans indicate that it is proposed to construct a five (5) storey, residential building having a 1-level underground parking structure, at the site.

The anticipated finished floor elevation for the proposed 1-level underground parking structure is at 230.14 masl. As such, the highest shallow groundwater elevation is about 2.27 m above the base of the proposed underground parking structure, and it is about 3.46 m above the proposed elevator pit structure. Based on this, it is anticipated that some long-term foundation drainage will be required for the proposed underground parking and elevator pit structures.

Permanent Drainage for the Proposed 1- Level Underground Parking and Elevator Pit Structures

Five (5) Storey Apartment/Condominium Building Construction – 1-Level Underground Parking Structure (56.39 m x 43.0 m) with a Finished Floor Elevation Footing of 231.1 masl:

For the proposed five (5) storey apartment/condominium building, for the preliminary foundation drainage calculations, the estimated excavation for the 1-level underground parking structure occupies an area of approximately 2,422 square meters. It is approximately 56.39 m long by 43 m wide, with a perimeter of approximately 198.78 m, and having a finished floor elevation at 231.1 masl. The proposed elevation for the underground parking structure footings for the apartment building is at approximately 230.14 masl. A comparison of the proposed lowest excavation depth with the highest measured groundwater level, indicates that the highest shallow groundwater level elevation of 232.41 masl, as measured at BH/MW 1, location is about 2.27 m above the lowest proposed underground parking. As such, it is anticipated that that some permanent long-term foundation drainage will be required for the proposed basement structures. Darcy's Expression below, was used to assess the long-term foundation seepage flow estimates to the underground structure:

$$Q = KiA$$

Where:

Q = Estimated groundwater seepage drainage rate (m^3/day)

K = 4.40×10^{-5} m/sec (highest hydraulic conductivity (K) assessed for the sandy silt till encountered at the proposed underground structure depths during the study)

A = 451.23 m^2 for the surface area for weeper tiles around the perimeter of foundation footings and 152.28 m^2 for the total under-slab floor drainage network which are the approximate total surface areas for weeper tiles used to estimate groundwater seepage to the under-slab drainage network, below the water table (cross-sectional area of flow) (m)

iv = 0.015849 [unitless], Vertical Hydraulic Gradient for groundwater considered for the under-slab basement drainage network

ih = 0.0374 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter, shore wall, mira drainage network

Based on the proposed underground basement structure, the long-term seepage drainage flow rate to a Mira perimeter drainage network for a conventionally shored excavation is 64,098.71 L/day. The long-term, average drainage rate for an under-slab basement floor drainage network is 9,174.80 L/day. The combined, long-term seepage rate from both the perimeter foundation and the under-slab basement floor drainage networks is estimated at 73,273.51 L/day. By applying a safety factor of three (3), the combined drainage flow rate is estimated at 219,820.53 L/day.

The pumping facility and sump systems should be designed for the maximum expected drainage flow rates. The drainage piping should be properly constructed using weeper tiles surrounded by filter cloth, in turn surrounded by bedding stone or concrete sand to minimize potential losses of fines and to prevent silt from clogging of weeper tiles. Over time, the foundation drainage flows to the underground structures may diminish to a lower, or possibly negligible rate, but more likely to a lower, steady-state rate that will remain relatively constant over time. During the expected dry season, minimal or negligible long-term foundation drainage flows may be experienced.

Permanent Drainage for Elevator Pit Beneath the Proposed Apartment Building at an elevation of 228.95 masl:

An excavation depth elevation of 231.1 masl was indicated for the proposed basement finish floor elevation for the proposed underground parking structure. An additional excavation depth of 2.153 m (El. 228.95 masl) was considered for the base of the proposed elevator pit/shaft structure. Based on this depth, the shallow groundwater level elevation is about 3.46 m above the base for the proposed elevator pit structures.

Given the low anticipated groundwater seepage rate estimates for any long-term foundation drainage, a standard drainage network can be included with the design for a conventionally shored excavation, along with a simple basement under-slab drainage network to address any long-term foundation seepage to the excavation and the completed underground elevator pit structure. These systems can be drained to sump pits. The drainage network should be designed by a qualified mechanical engineer, having experience with the designs for under-slab and footing drainage networks.

It is our understanding, that a sump pit is required within an elevator pit to satisfy building code requirements for fire retardant sprinklers to meet fire protection codes. The sump pit to meet fire protection sprinkler effluent can be drained to the sanitary sewer.

In order to estimate the long-term foundation drainage needs associated with a perimeter foundation drainage network and the under-slab elevator pit structure drainage system, Darcy's Equation was used, as described below:

$$Q = KiA$$

Where:

- Q = Estimated seepage drainage rate (m^3/day)
- K = 4.4×10^{-5} m/sec (highest hydraulic conductivity (K) assessed for the sandy silt till encountered at the depth for the elevator pit during the study)
- A = 55.36 m^2 for the surface area for the mira drain shored wall perimeter around the elevator pit and 1.0 m^2 for the total under-slab floor drainage network beneath the elevator pit, which are the approximate total surface areas for shore walls and weeper tiles respectively that were used to estimate groundwater seepage to under slab drainage network, below the water table (cross-sectional area of flow) (m)
- iv = 0.02416 [unitless], Vertical Hydraulic Gradient for groundwater considered for the under-slab of elevator pit drainage system
- ih = 0.0374 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter, shore wall, mira drainage system.

Based on the proposed elevator pit structure, the long-term seepage drainage rate to the Mira perimeter drainage network for a conventionally shored excavation is 7,864.06 L/day. The long-term, average drainage seepage rate for an under-slab basement floor drainage network is 92.29 L/day. The combined, long-term seepage rate from both the perimeter foundation and the under-slab basement floor drainage networks are estimated at 7,956.35 L/day. By applying a safety factor of three (3), the combined drainage flow rate is estimated at 23,869.05 L/day.

The pumping facility and sump systems should be designed for the maximum expected drainage flow rate. The systems should be designed by a qualified mechanical engineer with experience in design for foundation drainage systems. The drainage piping should be properly constructed using weeper tiles surrounded by filter cloth, in turn surrounded by bedding stone or concrete sand to minimize potential losses of fines and to prevent silt from clogging of weeper tiles. Over time, the foundation drainage flow for the underground structures may diminish to a lower, or possibly negligible rate, but more likely to a lower, steady-state rate that will remain relatively constant over time. During the expected dry season, minimal or negligible long-term foundation flows may be experienced. The drainage networks should have separate connections to the proposed sump pits, with one pit connected to the shored wall/mira drainage network and a second pit connected to the basement underslab drainage network.

It is our understanding that the proposed underground foundation structure will be built, completely waterproof, to cut off any groundwater seepage to the excavation and underground structure, with no connection being needed to the City's Sewer System, as no long-term foundation drainage is anticipated.

7.5 **Groundwater Function of the Subject Site**

The subject site is located within an existing developed residential neighbourhood. Lake Simcoe is located approximately 120 m south of the site, and a tributary of Lake Simcoe is located approximately 1,125 m west of the site, where it flows in a north to south direction, before emptying into the Lake. Scattered wooded areas are located about 75 m north of the site.

Due to the distances of these features from the site, and given the relatively small area of the site, minimal groundwater contribution to these features is anticipated from the subject site. As such negligible impacts to these features are anticipated.

Any construction dewatering will be temporary and the long-term foundation drainage rates are anticipated to be low.

7.6 **Low Impact Development**

The surficial shallow subsoil beneath the site consists, predominantly of silty sand till, sandy silt till, and sand deposits. Opportunities may exist to infiltrate collected runoff to the subsurface at the developed site, using appropriate Low Impact Development Infrastructure, such as infiltration galleries or underground storage/exfiltration tanks. The groundwater lies at depths, ranging between 0.99 m and 2.70 m below the ground surface. LID infrastructure can be implemented in areas where the shallow groundwater is deeper than 1.0 m below the ground surface and where it is possible to maintain a minimum of 1.0 m separation between the bases of any proposed LID stormwater management infiltration infrastructure and the high groundwater table. Any proposed LID infrastructure should be designed by the stormwater engineer for the project. Based on the preliminary plans, the proposed LID infrastructure will comprise a bioretention swale, storage chamber, and a soak away pit, at southern limits of the site.

8.0 CONCLUSIONS

1. The subject site is located within the physiographic region of Southern Ontario known as the Simcoe Lowlands, which is located on native soils comprised of Undifferentiated Till deposits, consisting predominantly of sandy silt to silt matrix, which is high in matrix calcium carbonate content, considered as having moderate to high clast content.
2. The subject site is located within the Barrie Creek sub-watershed, of the Lake Simcoe Watershed.
3. A review of the topography shows that the subject site is relatively flat, having a gentle decline in elevation relief towards its south limits.
4. The study has disclosed that beneath a layer of topsoil, and earth fill, the native soils underlying the subject site consists of silty sand till, sandy silt till, and fine to medium sand, extending to the maximum investigated depth of 6.5 m.
5. The findings of this current study confirm that the groundwater levels range from El. 228.90 to 232.41 masl (i.e., 0.99 to 2.70 m below ground surface). Review of the average of shallow groundwater level elevations suggests that it flows in a southerly direction, towards Lake Simcoe.
6. The single well response tests yielded hydraulic conductivity (K) estimates for the silty sand till is 4.0×10^{-7} m/s. The K estimate for the sandy silt till is 4.40×10^{-5} m/s, and the K estimate for the silty clay till, and medium sand, is 9.30×10^{-7} m/s. The above results suggest that the hydraulic conductivity for the groundwater-bearing sub-soils at the depths of the well screens is moderate to high, with corresponding moderate to high anticipated groundwater seepage rates into open excavations, below the water table.
7. The Hazen Equation calculated results indicates that the K estimate for the silty sand till, having traces of clay and gravel, retrieved from a depth of 4.8 mbgs at BH/MW 1 is 9.0×10^{-8} m/sec, the K estimate for the sandy silt till, having traces of clay, retrieved from a depth of 6.3 mbgs at BH/MW 2 is 4.84×10^{-6} m/sec, and the K estimate for the fine to medium sand, retrieved from a depth of 4.8 mbgs at BH/MW 3 is 2.5×10^{-5} m/sec. The K estimate determined from the Hazen method suggests low to high hydraulic conductivity (K) estimates for any encountered shallow perched groundwater found beneath the subject site.
8. The groundwater levels beneath the site are approximately 2.27 m above the proposed basement floor level for the apartment/condominium building and the stormwater storage chamber, and it is 3.46 m above the proposed elevator pit structure.
9. The groundwater levels at the site are approximately 0.06 to 1.02 m below to 0.21 m below the invert levels for the proposed underground services.

10. The dewatering flow estimates for the installation of the underground services suggests that they could range from between 2,753 L/day and 40,911 L/day; by applying a safety factor of three (3), they could reach maximums of between 8,074 and 120,759 L/day. Construction dewatering rates that are below the 50,000 L/day limit threshold will not require any registration or filing with the MECP. Construction dewatering rates below the PTTW threshold of 400,000 L/day, but which are above the 50,000 L/day groundwater taking approval requirement threshold, will be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP.
11. The dewatering flow estimates for construction of the proposed apartment buildings underground parking structure including the stormwater storage chamber suggests that the dewatering rate is about 106,797 L/day; by applying a safety factor of three (3), it could reach a maximum of 320,392 L/day. The dewatering flow estimates for the construction of the proposed elevator pit structure suggests that the rate is about 10,597 L/day; by applying a safety factor of three (3), it could reach a maximum of 31,790 L/day. These anticipated dewatering rates for earthworks excavation are below the PTTW threshold limit of 400,000 L/day but are above 50,000 L/day water taking approval requirement threshold, whereby the approval for the proposed groundwater takings for construction dewatering program to complete the basement and elevator pit structures would be required to be registered through an Environmental Activity and Sector Registry (EASR) with the EASR filing through the MECP.
12. It is anticipated to collected any short-term construction dewatering effluent, within a temporary storage tank, for later disposal management off site at an MECP approved receiving facility during construction.
13. The estimated zone of influence for construction dewatering could reach a maximum of 88.8 m away from the conceptual dewatering alignments for the construction areas. There are neighbouring residential properties that are within the conceptual zone of influence for construction dewatering; however, no groundwater receptors, such as water wells, bodies of water, watercourses or wetlands are present within the conceptual zone of influence for dewatering for the proposed development. The local shallow groundwater flow pattern for the site may be temporarily affected during construction.
14. Long-term foundation drainage rates from a mira drainage network for a conventionally shored excavation foundation for the proposed/condominium apartment building underground parking structure, is approximately 73,273.51 L/day. By applying a safety factor of three (3), the anticipated drainage flow rates could reach a maximum of 219,820.53 L/day.

15. The Long-term foundation drainage rates from both an under-slab floor drainage network and from a mira drainage network for a conventionally shored excavation foundation and for the proposed elevator pit structure for the apartment building is approximately 7,956.35 L/day. By applying a safety factor of three (3), the foundation drainage flow rates could reach a maximum of 23,869.05 L/day.
16. It is our understanding that the proposed underground foundation structures will be built, completely waterproof to cut off any groundwater seepage to the excavation and underground structure, with no connection being needed to the City's Sewer System as no long-term foundation drainage is anticipated.
17. The groundwater levels lie at depths, ranging from between 0.99 to 2.70 m below the existing ground surface. As such passive LID measures such as bioswales, rain gardens and the thickening topsoil should be considered to divert storm runoff away from the municipal storm sewers and to recharge groundwater table where possible, to address future stormwater management planning for the proposed development.

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Angella Graham, M.Sc.

Gavin O'Brien, M.Sc., P.Geo.
AG/GO

PROFESSIONAL GEOSCIENTIST
GR OB
GAVIN R. O'BRIEN
PRACTISING MEMBER
1267
July 23, 2021
ONTARIO

9.0 REFERENCES

1. The Physiography of Southern Ontario (Third Edition), L. J. Chapman and D. F. Putnam, 1984.
2. Bedrock Geology of Ontario, 1974, Preliminary Map P. 979, Ontario Division of Mines.
3. D.P. Rogers, R.C. Ostry and P.F. Karrow, 1961, Metropolitan Toronto Bedrock Contours, Ontario Department of Mines, Preliminary Map 102.
4. Barrie Creeks, Lovers Creek, and Hewitt's Creek Subwatershed Plan, Lake Simcoe Conservation Authority 2012.

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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

FIGURES 1 to 3

BOREHOLE AND MONITORING WELL LOGS

REFERENCE NO. 1809-W012

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

□ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

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JOB NO.: 1809-W012

LOG OF BOREHOLE NO.: BH/MW 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

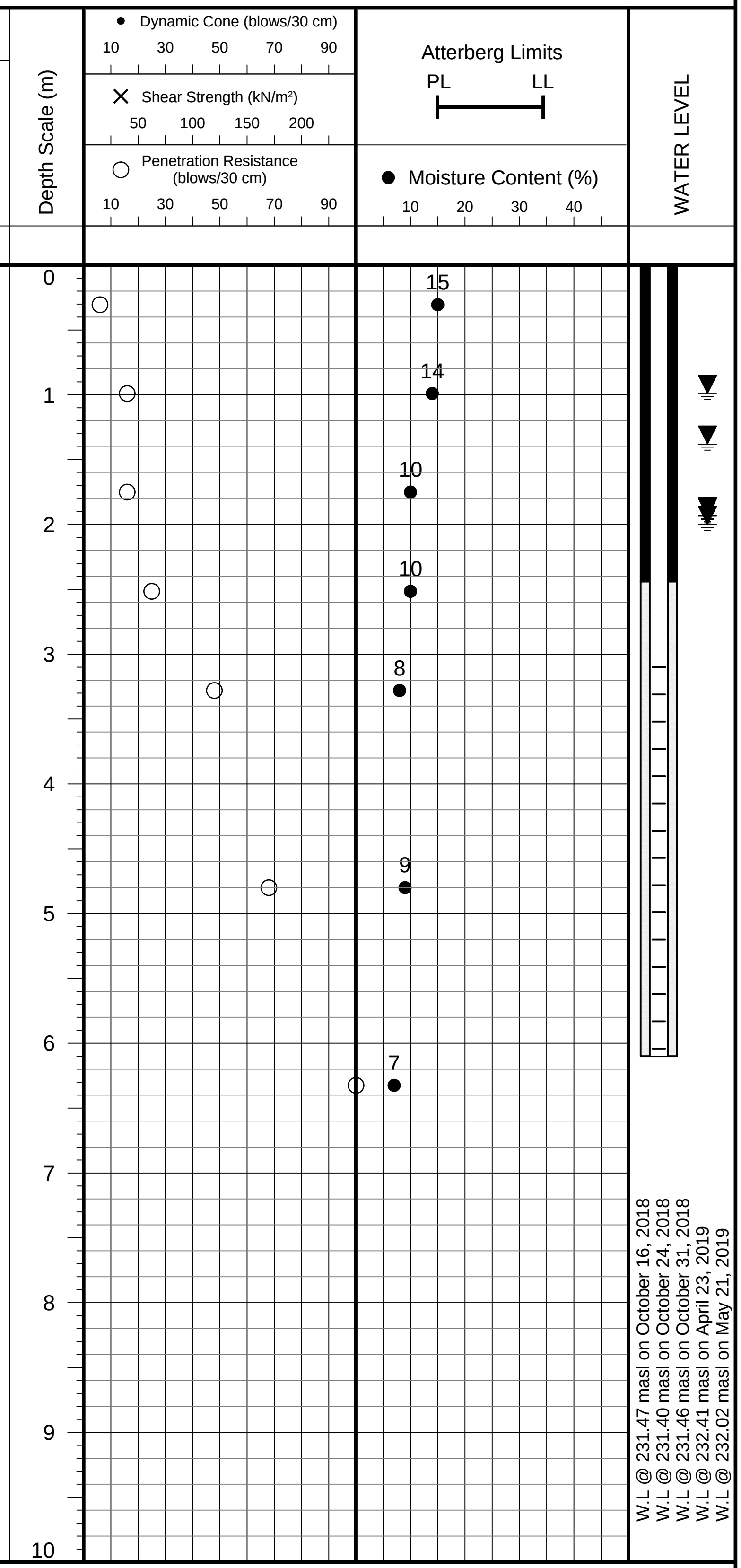
METHOD OF BORING: Solid Stem Flight-Auger

PROJECT LOCATION: 27-31 Blake Street, City of Barrie

DRILLING DATE: October 2, 2018

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Moisture Content (%)
233.4	Ground Surface				
0.0 – 0.2	**20 cm TOPSOIL**				
0.2 – 232.6 / 0.8	Brown **EARTH FILL** silty clay, tace of gravel, and root inclusions	1	DO		15
232.6 / 0.8 – 231.9 / 1.5	Brown **EARTH FILL** silty sand, a trace of clay, and root inclusions	2	DO		14
231.9 / 1.5 – 226.8 / 6.5	Grey, compact to very dense **SILTY SAND TILL** traces of clay and gravel	3	DO		10
		4	DO		10
		5	DO		8
		6	DO		9
		7	DO		7
226.8 / 6.5	**END OF BOREHOLE**				

Installed 50 mm dia. monitoring well to 6.1 m
Slotted screen from 3.1 to 6.1 m
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0 to 2.4 m
Provided with a flushmount casing



W.L @ 231.47 masl on October 16, 2018
W.L @ 231.40 masl on October 24, 2018
W.L @ 231.46 masl on October 31, 2018
W.L @ 232.41 masl on April 23, 2019
W.L @ 232.02 masl on May 21, 2019

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JOB NO.: 1809-W012

LOG OF BOREHOLE NO.: BH/MW 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Flight-Auger

PROJECT LOCATION: 27-31 Blake Street, City of Barrie

DRILLING DATE: October 2, 2018

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Moisture Content (%)
232.4	Ground Surface					
0.0	**20 cm TOPSOIL**				0	
0.2	Black **EARTH FILL** silty clay with root inclusions	1	DO			20
231.6 / 0.8	Brown, dense to very dense **SILTY SAND TILL** a trace of clay and gravel	2	DO		1	9
		3	DO		2	8
		4	DO			7
229.3 / 3.0	Grey, very dense **SANDY SILT TILL** traces of clay and gravel	5	DO		3	7
		6	DO		4–5	16
		7	DO		6	18
225.8 / 6.5	**END OF BOREHOLE** Installed 50 mm dia. monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a flushmount casing				7–10	

Dynamic Cone (blows/30 cm): 10 30 50 70 90

Shear Strength (kN/m²): 50 100 150 200

Penetration Resistance (blows/30 cm): 10 30 50 70 90

Atterberg Limits: PL LL

Moisture Content (%): 10 20 30 40

WATER LEVEL

W.L @ 230.30 masl on October 16, 2018
W.L @ 230.23 masl on October 24, 2018
W.L @ 230.68 masl on October 31, 2018
W.L @ 231.10 masl on April 23, 2019
W.L @ 230.55 masl on May 21, 2019

Soil Engineers Ltd.

JOB NO.: 1809-W012

LOG OF BOREHOLE NO.: BH/MW 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Flight-Auger

PROJECT LOCATION: 27-31 Blake Street, City of Barrie

DRILLING DATE: October 2, 2018

El. (m) / Depth (m)	SOIL DESCRIPTION
231.6	Ground Surface
0.0	**20 cm TOPSOIL**
0.2	Brown, loose to compact **FINE SAND**
229.3 / 2.3	Brown, compact to dense **SILTY SAND TILL**
227.0 / 4.6	Grey, very dense **FINE TO MEDIUM SAND** some silt traces of clay coarse sand and gravel
225.0 / 6.5	**END OF BOREHOLE** Installed 50 mm dia. monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a flushmount casing

SAMPLES

Number	Type	N-Value	Moisture Content (%)
1	DO		10
2	DO		10
3	DO		15
4	DO		17
5	DO		16
6	DO		19
7	DO		17

Depth Scale (m): 0 – 10

● Dynamic Cone (blows/30 cm): 10 30 50 70 90

× Shear Strength (kN/m²): 50 100 150 200

○ Penetration Resistance (blows/30 cm): 10 30 50 70 90

Atterberg Limits: PL — LL

● Moisture Content (%): 10 20 30 40

WATER LEVEL

W.L @ 228.90 masl on October 16, 2018
W.L @ 229.21 masl on October 24, 2018
W.L @ 229.32 masl on October 31, 2018
W.L @ 229.98 masl on April 23, 2019
W.L @ 229.72 masl on May 21, 2019

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FIGURES 4 to 6

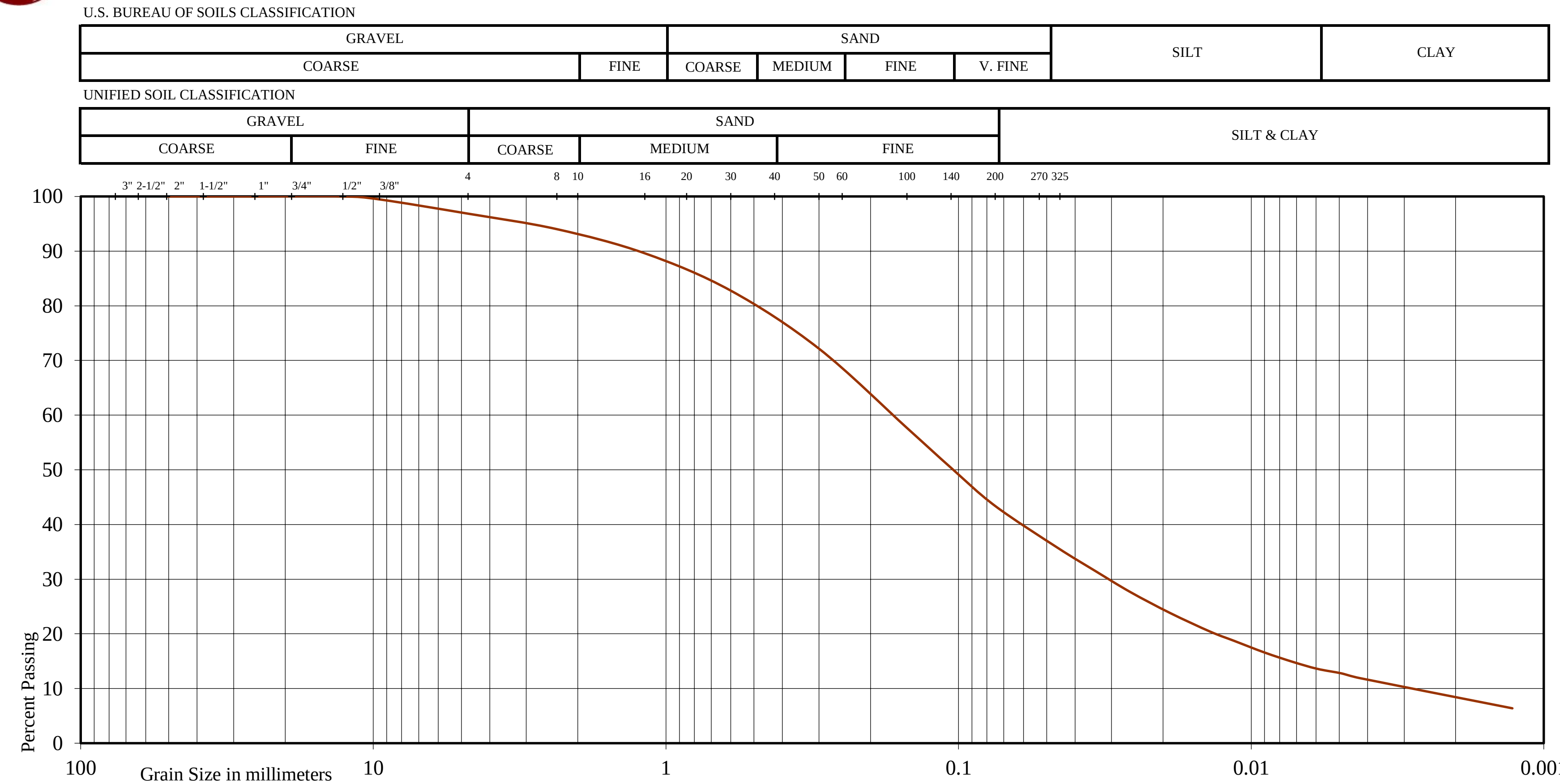
GRAIN SIZE DISTRIBUTION GRAPHS

REFERENCE NO. 1809-W012

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1809-W012



Project: Proposed Residential Development

Location: 27-31 Blake Street

Borehole No: 1

Sample No: 6

Depth (m): 4.8

Elevation (m): 228.6

Estimated Permeability (m./sec.) = 10^{-7}

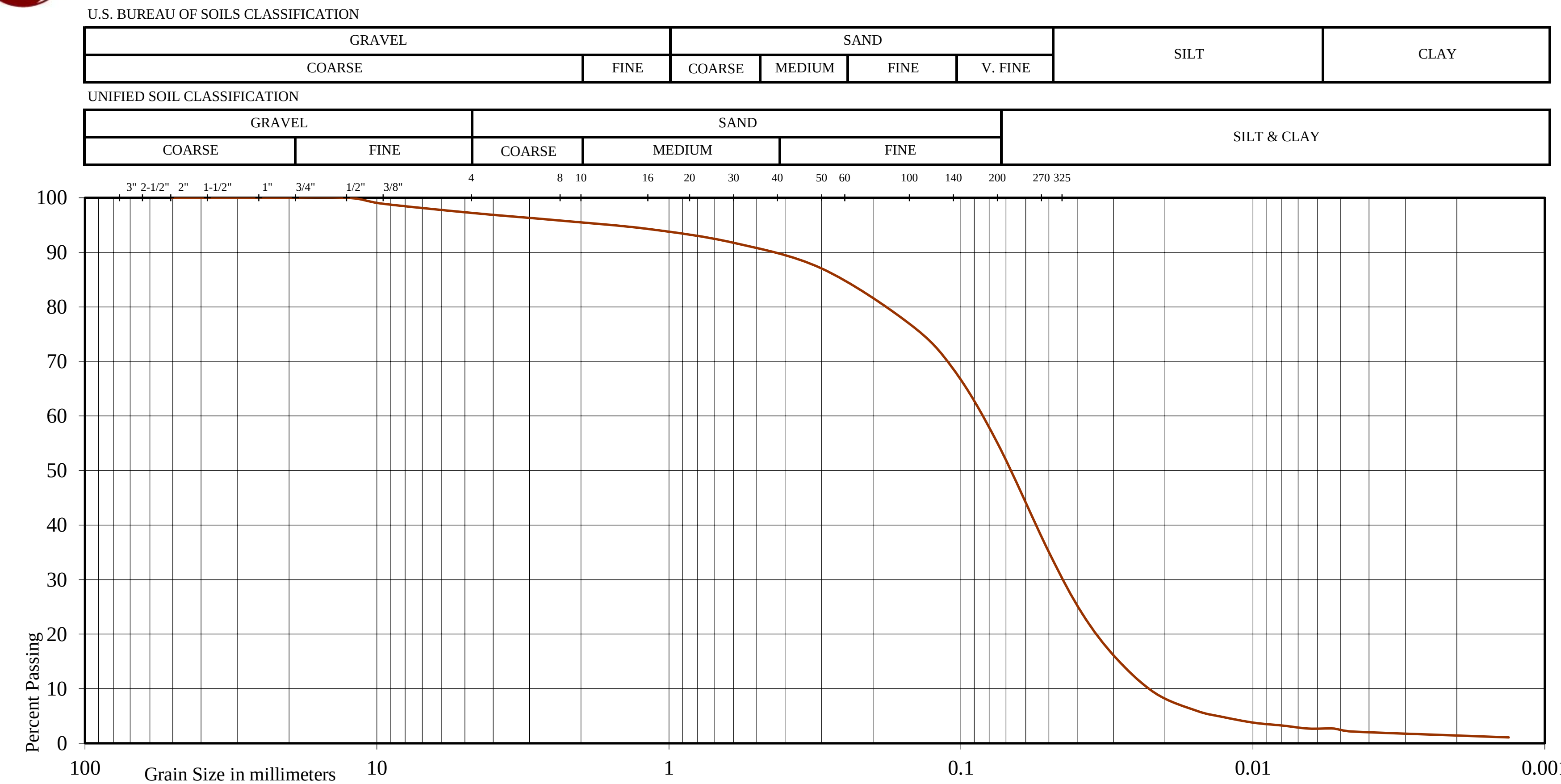
Classification of Sample [& Group Symbol]: SILTY SAND TILL, traces of clay and gravel

Figure: 4

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1809-W012



Project: Proposed Residential Development

Location: 27-31 Blake Street

Borehole No: 2

Sample No: 7

Depth (m): 6.3

Elevation (m): 226.1

Estimated Permeability (m./sec.) = 10^{-5}

Classification of Sample [& Group Symbol]: SANDY SILT TILL, traces of clay and gravel

Figure: 5

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

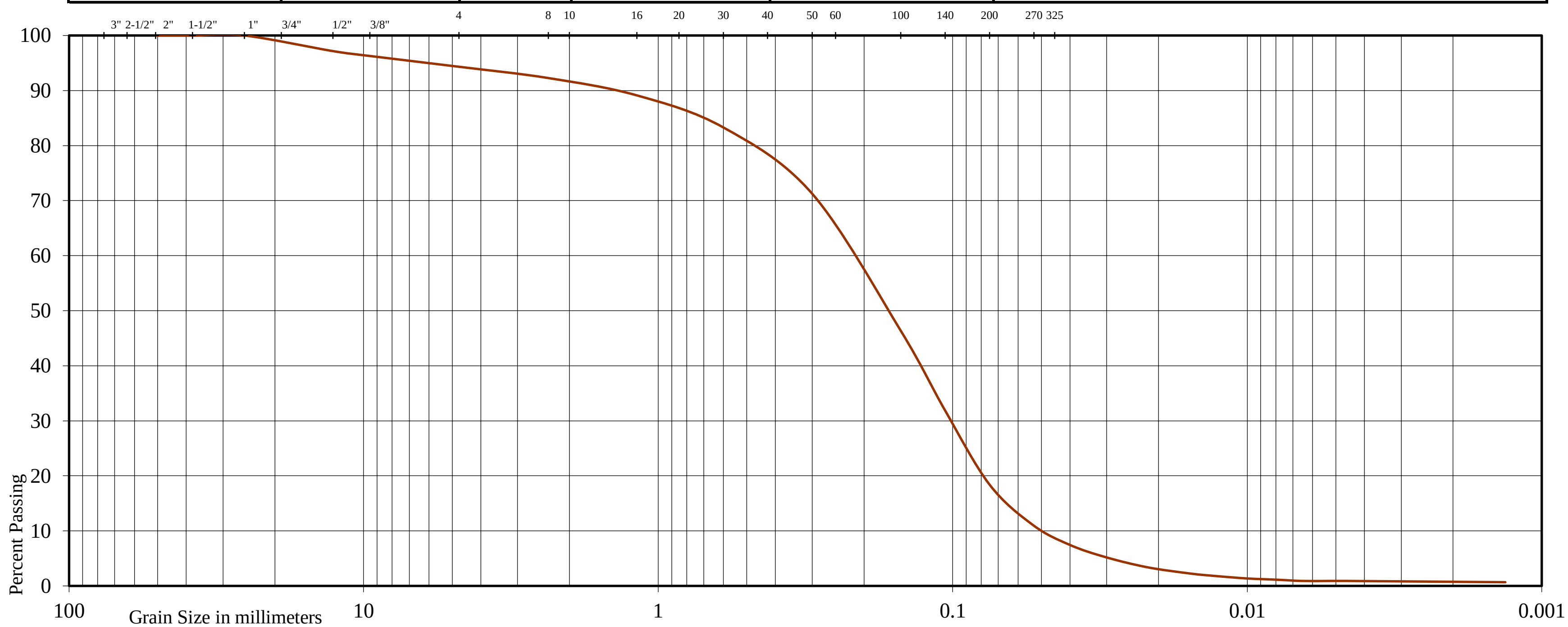
Reference No: 1809-W012

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Residential Development

Location: 27-31 Blake Street

Borehole No: 3

Sample No: 6

Depth (m): 4.8

Elevation (m): 226.8

Estimated Permeability (m./sec.) = 10^{-5}

Classification of Sample [& Group Symbol]: FINE TO MEDIUM SAND, some silt, traces of clay, coarse sand and gravel

Figure: 6

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DRAWINGS 1 to 9

REFERENCE NO. 1809-W012

Source: Water Course, Ontario Ministry of Natural Resources and Forestry
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Source: Water Body, Ontario Ministry of Natural Resources and Forestry
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N

Approximate Boundary of Subject Site

Waterbody

Major Road

Local Road

Soil Engineers Ltd.

Title: Site Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:
0 30 60 120 180 240 300
Metres

Drawing No. 1

N

Approximate Boundary of Subject Site

Borehole with Monitoring Well

Major Road

Soil Engineers Ltd.

Title: Borehole and Monitoring Well Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:
0 3 6 12 18 24 30
Metres

Drawing No. 2

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Source: Water Course, Ontario Ministry of Natural Resources and Forestry
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Source: Water Body, Ontario Ministry of Natural Resources and Forestry
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N

Approximate Boundary of Subject Site

500 metres from Subject Site Boundary

1 Well Location from MECP Well Records (see Appendix 'A')

Waterbody

Major Road

Local Road

Soil Engineers Ltd.

Title: MECP Well Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

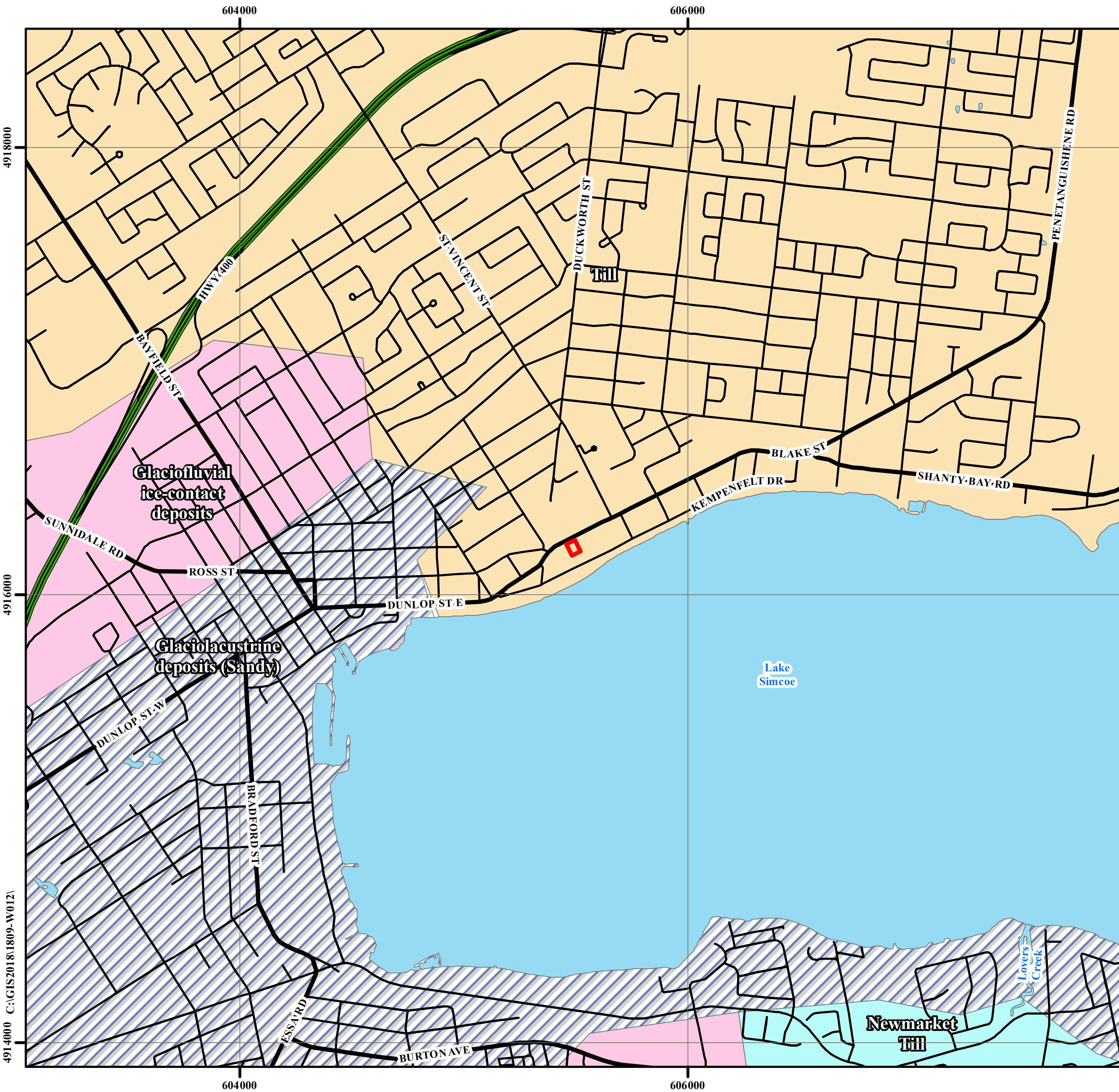
Reference No. 1809-W012

Date: November 27, 2018

Scale:

0 30 60 120 180 240 300
Metres

Drawing No. 3



N

Approximate Boundary of Subject Site

Glaciofluvial ice-contact deposits
Material: gravel and sand, minor till, includes esker, kame, end moraine, ice-marginal delta and subaqueous fan deposits

Glaciolacustrine deposits (Sandy)
Material: sand, gravelly sand and gravel, nearshore and beach deposits

Newmarket Till
Material: sandy silt to silt matrix, moderate to high matrix carbonate content, clast content moderate to high

Till
Material: undifferentiated, predominantly sandy silt to silt matrix, commonly rich in clasts, often high in total matrix carbonate content

Waterbody

Expressway/Freeway

Major Road

Local Road

Soil Engineers Ltd.

Title: Quarternary and Surface Geology Map

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

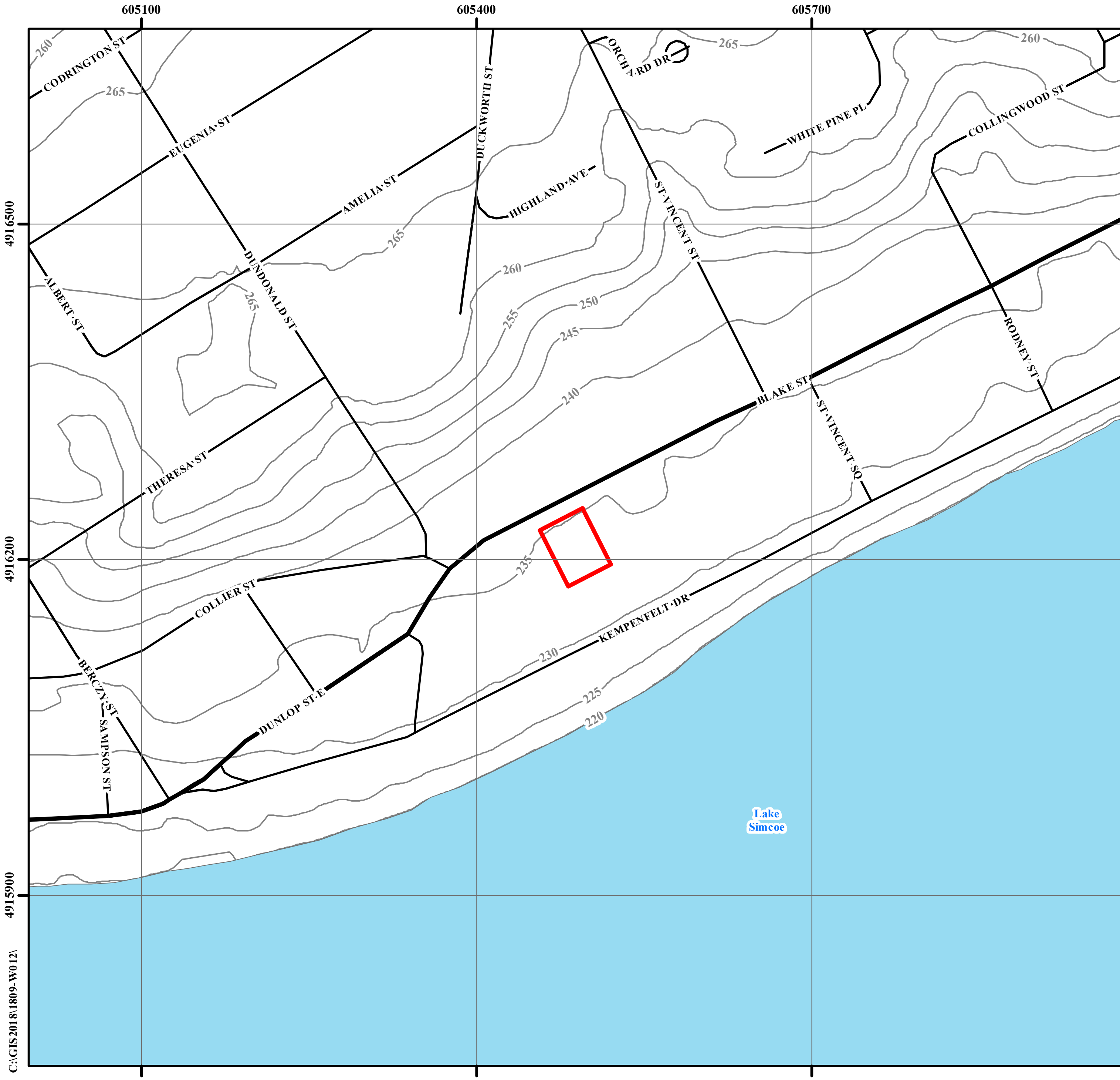
Reference No. 1809-W012

Date: November 27, 2018

Scale:
0 125 250 500 750 1,000 1,250
Metres

Drawing No. 4

Source: Ontario Geological Survey, 1997, Surface Geology of Ontario; Ontario Geological Survey, Miscellaneous Released-Data 0014



N

Approximate Boundary of Subject Site

Waterbody

Major Road

Local Road

235 Topographic Contour (masl)

Soil Engineers Ltd.

Title: Topographic Map

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:

0 25 50 100 150 200 250

Metres

Drawing No. 5

Source: Water Body, Ontario Ministry of Natural Resources and Forestry, 2015
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Source: Contour, Ontario Ministry of Natural Resources and Forestry, 2015
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Source: Water Course, Ontario Ministry of Natural Resources and Forestry, 2015
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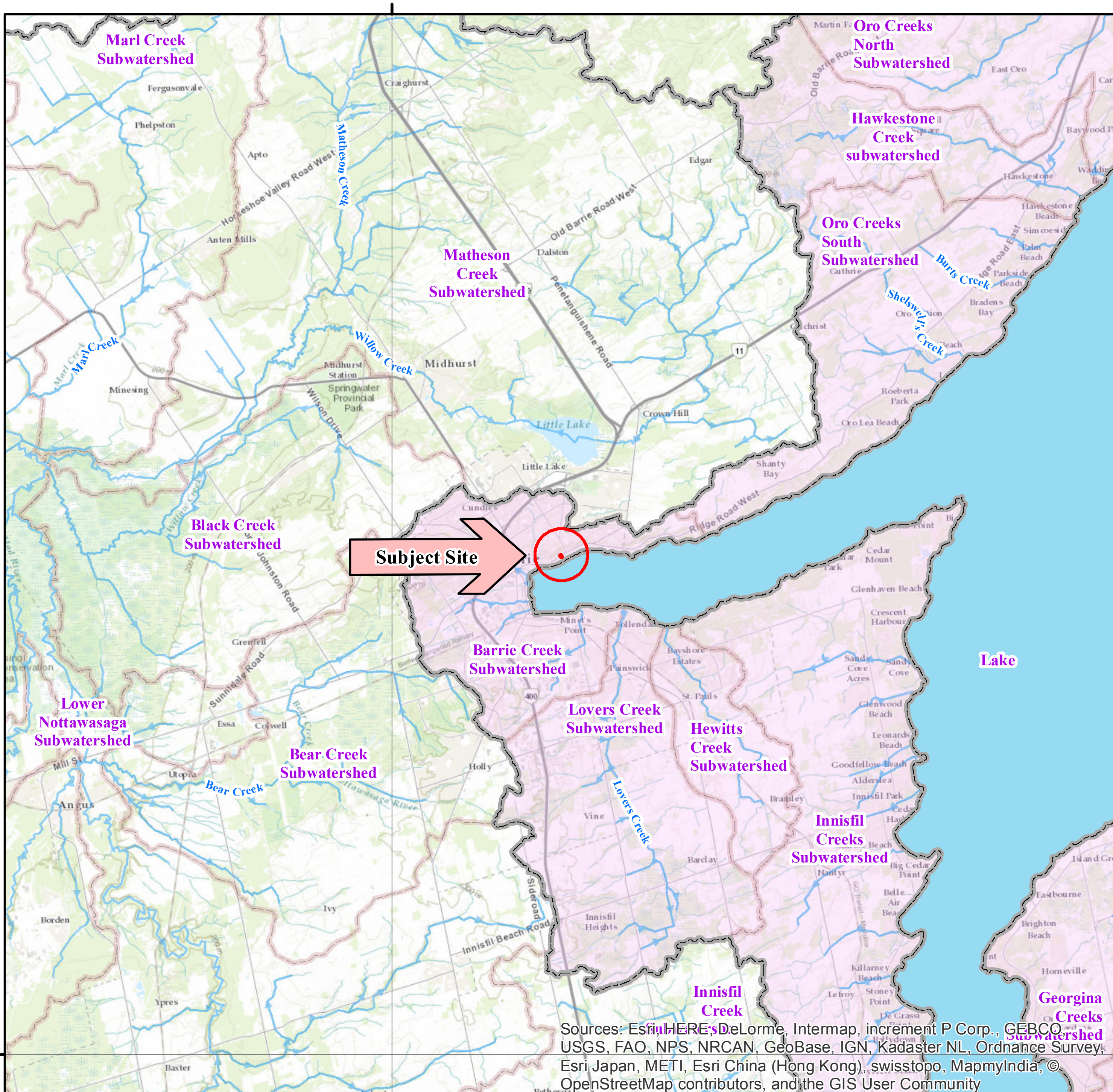
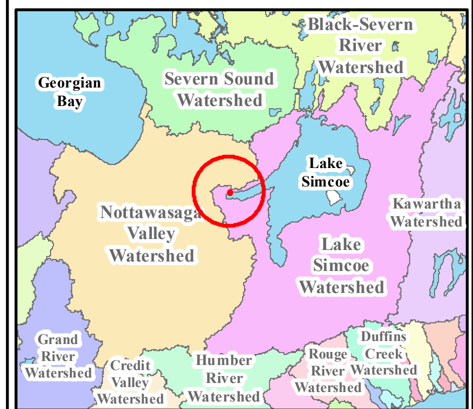


C:\GIS2018\1809-W012\

this mapping was produced by SEL and should be used for information purposes only.
Data sources used in its production are of varying quality and accuracy and all boundaries should be considered approximate.

N

Watershed:



Approximate Boundary of the Subject Site

Watershed Boundaries

Waterbody

Watercourse

Expressway/Major Road

Soil Engineers Ltd.

Title: Watershed and Subwatershed Map

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:
0 11,250 22,500 45,000 67,500 90,000 112,500
Metres

Drawing No. 6

N

Approximate Boundary of Subject Site

Wooded Area

Water Body

Watercourse

Expressway/Freeway

Major Road

Local Road

Soil Engineers Ltd.

Title: Natural Features and Protection Area Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:

0 75 150 300 450 600 750

Metres

Drawing No. 7

C:\GIS2018\1809-W012\

N

Approximate Boundary of Subject Site

Borehole with Monitoring Well

Major Road

A A' Cross-Section Direction

235 Topographic Contour (masl)

Soil Engineers Ltd.

Title: Cross-Section Key Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:

0 3 6 12 18 24 30
Metres

Drawing No. 8-1

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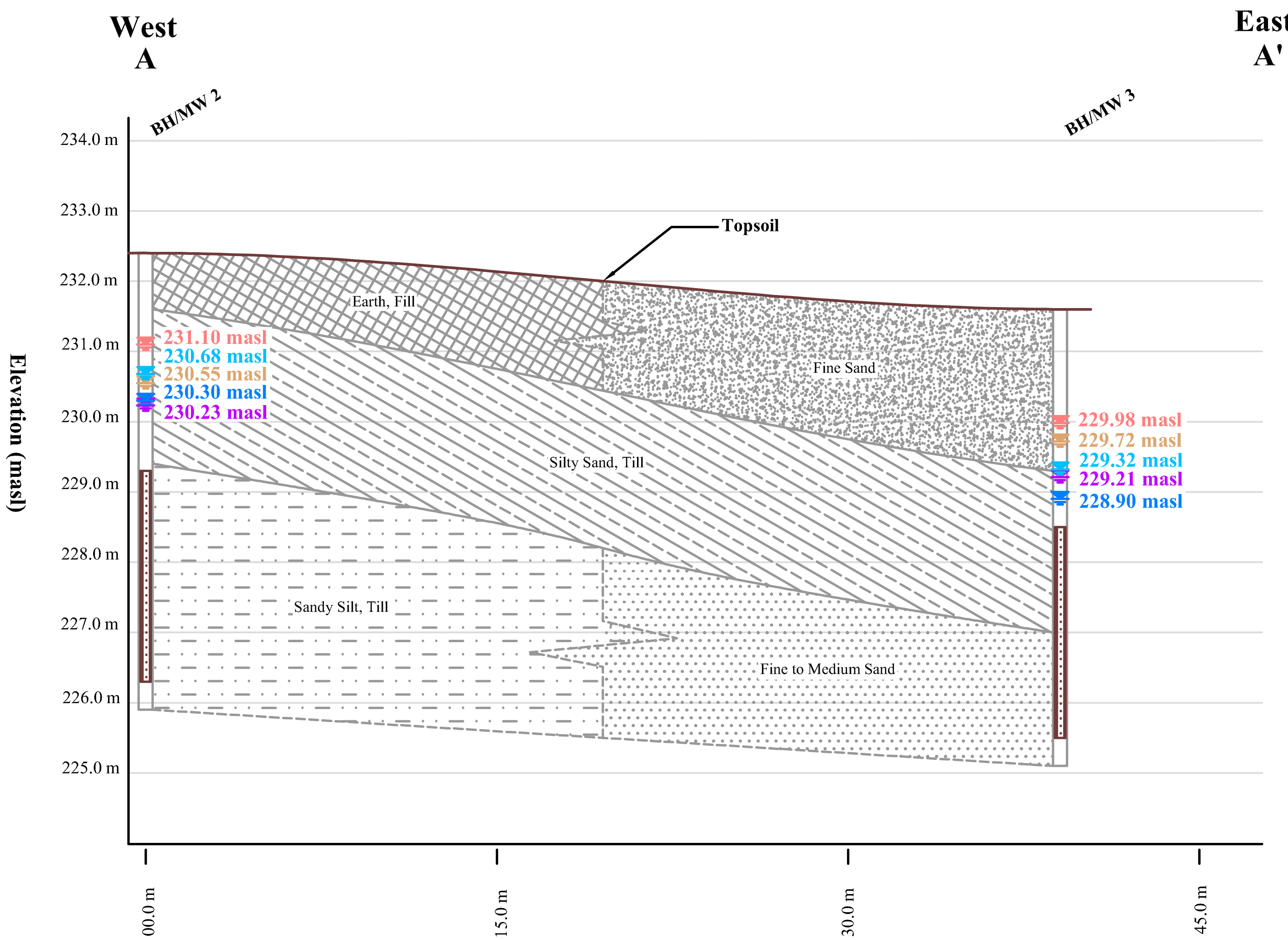
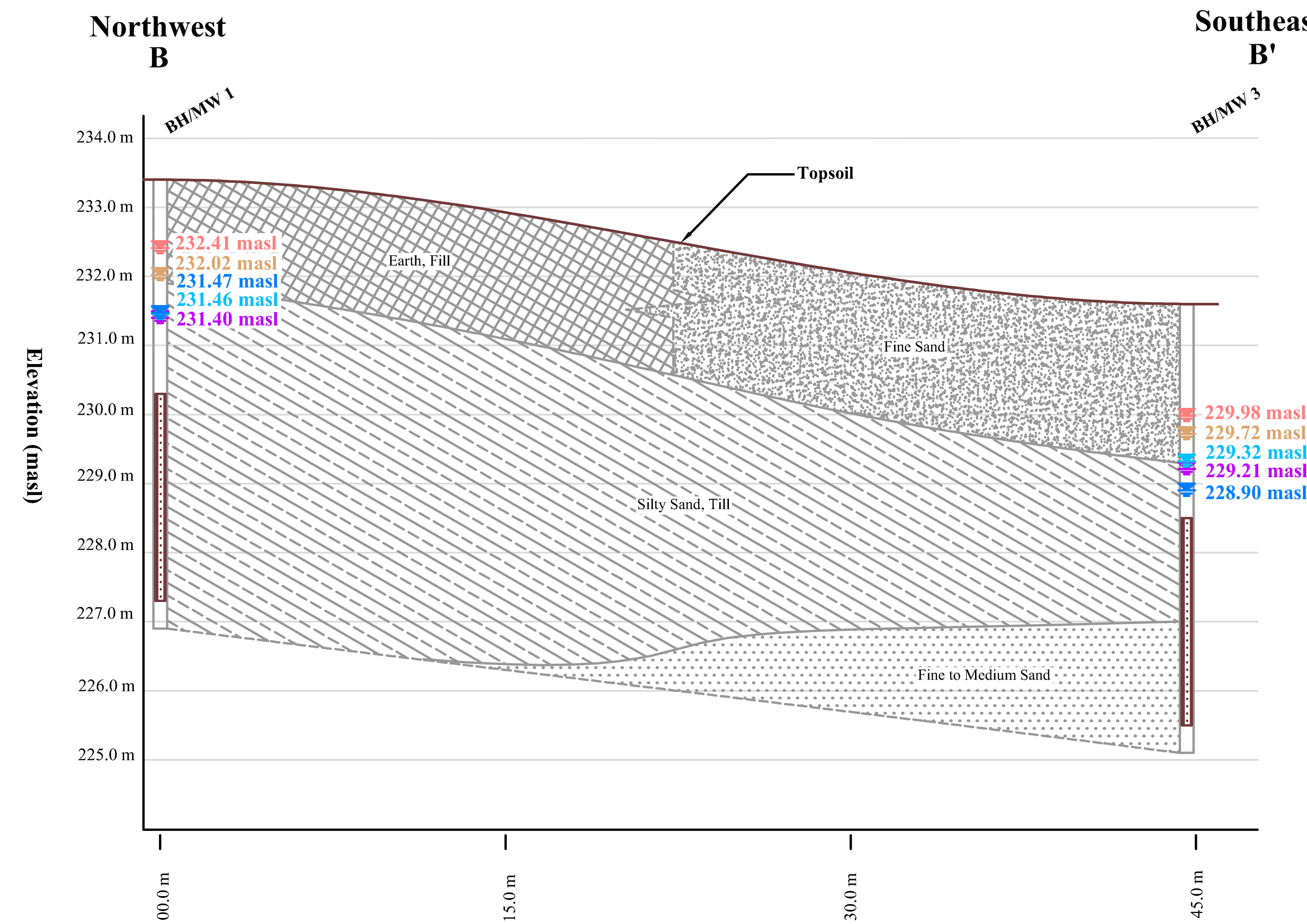


Section A-A'



Section B-B'

C:Projects/2018 Jobs/1809-W012

Earth, Fill

Silty Sand, Till

Sandy Silt, Till

Fine Sand

Fine to Medium Sand

3.0 m Screen

Water Table on October 16, 2018

Water Table on October 24, 2018

Water Table on October 31, 2018

Water Table on April 23, 2019

Water Table on May 21, 2019

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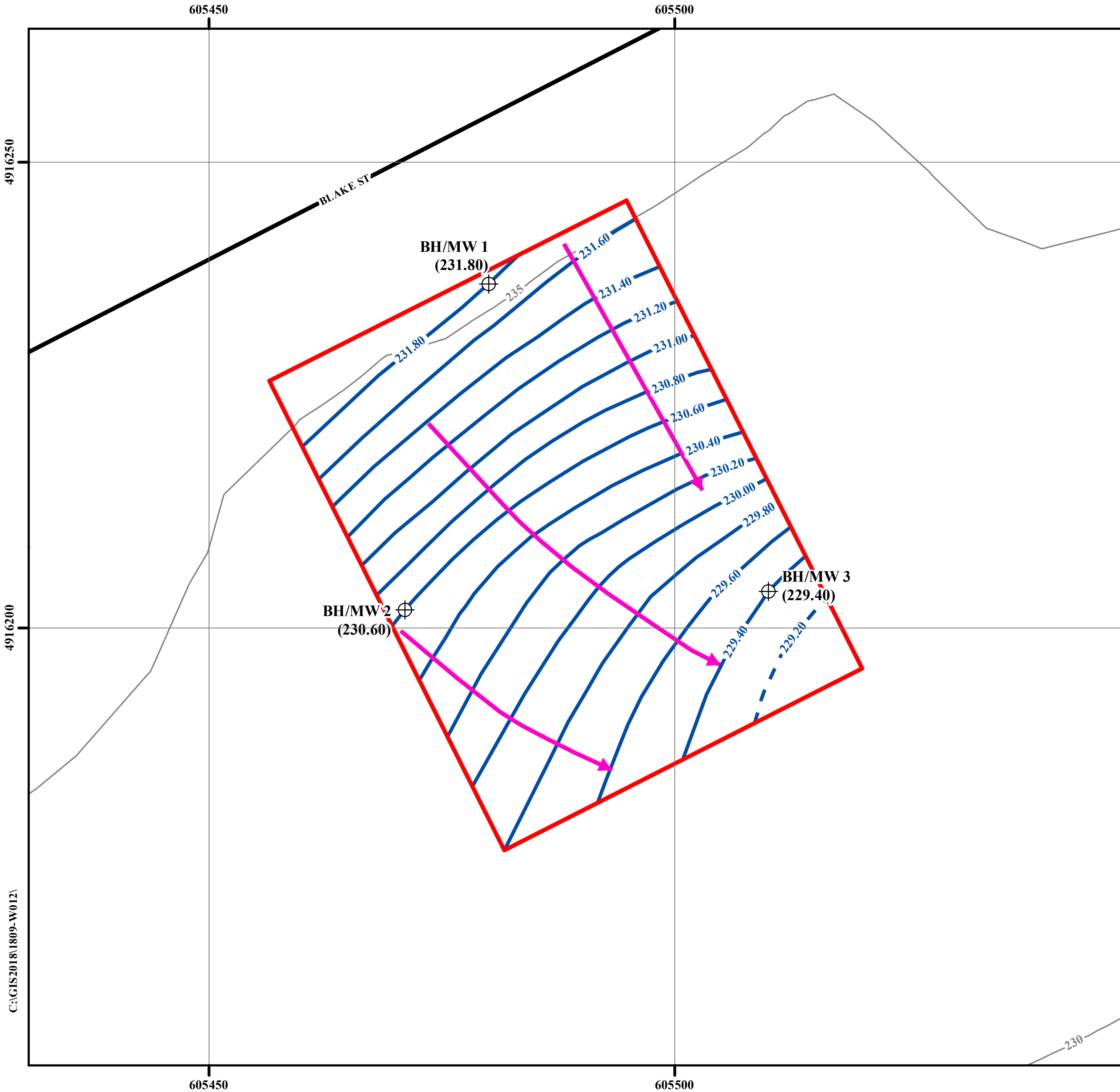
CONSULTING SOIL, FOUNDATION & ENVIRONMENTAL ENGINEERS

Title: Geological Cross-Sections (A-A' and B-B')

Project: Hydrogeological Assessment
Proposed Residential Development
27-31 Blake Street, City of Barrie

Reference No:	Date:	Scale: V	Scale: H	Drawing No.
1809-W012	November, 2018	1:100	1:300	8-2



Approximate Boundary of Subject Site

Borehole with Monitoring Well

Interpreted Shallow Groundwater Flow Direction

Interpreted Shallow Groundwater Level Elevation (masl)

Inferred Shallow Groundwater Level Elevation (masl)

Major Road

235 Topographic Contour (masl)

230.60 Average Shallow Groundwater Level Elevation (masl)

Soil Engineers Ltd.

Title: Shallow Groundwater Flow Pattern Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
27- 31 Blake Street
City of Barrie

Reference No. 1809-W012

Date: November 27, 2018

Scale:

0 3 6 12 18 24 30

Metres

Drawing No. 9

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90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL (416) 754-8515 · FAX (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	PETERBOROUGH	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 440-2040	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

APPENDIX 'A'

MECP WATER WELL RECORDS SUMMARY

REFERENCE NO. 1809-W012

Ontario Water Well Records

WELL ID	MOECC WWR ID	Construction Method	Well Depth (m)**	Well Usage		Water Found (m)**	Static Water Level (m)**	Top of Screen Depth (m) **	Bottom of Screen Depth (m) **
				Final Status	First Use				
1	7038503	Direct Push	5.50	-	-	-	-	3.60	5.50
2	7191645	Rotary (Convent.)	18.30	-	Monitoring	4.58	-	16.77	18.30
3	5700233	Rotary (Convent.)	58.50	Test Hole	-	-	-	-	-
4	5714857	Rotary (Convent.)	128.30	Test Hole	-	-	-	-	-
5	7188109	Rotary (Convent.)	6.10	Observation Wells	Monitoring	-	-	3.05	6.10

Notes:

*MECP WWID: Ministry of the Environment, Conservation and Parks Water Well Records Identification

**metres below ground surface

APPENDIX 'B'

RESULT OF SINGLE WELL RESPONSE TESTS

REFERENCE NO. 1809-W012

Falling Head Test (Slug Test)

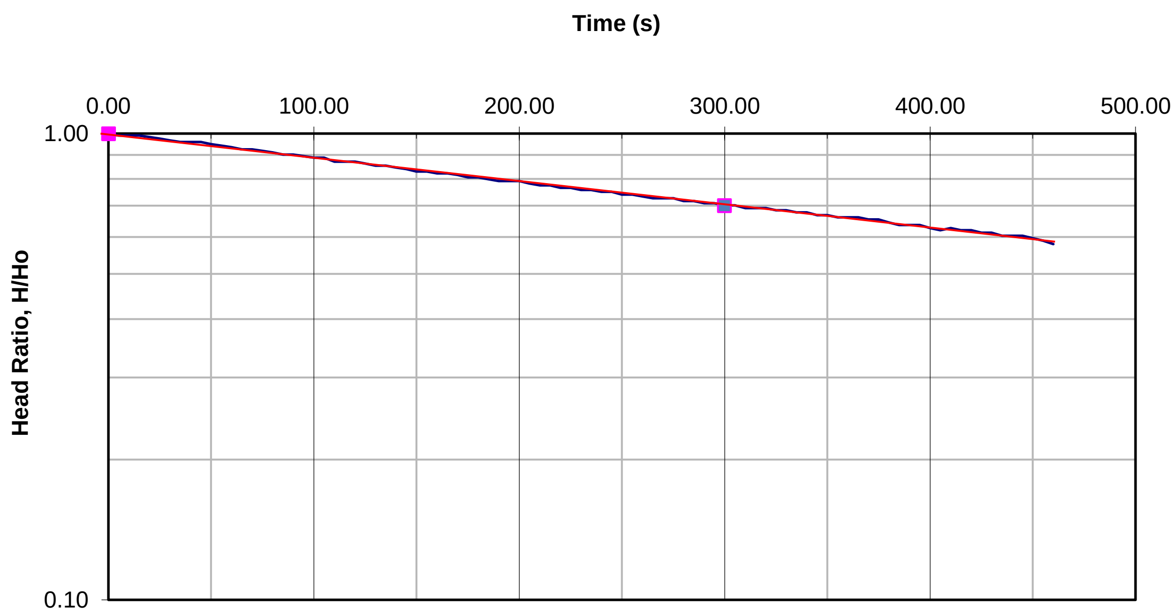
Test Date:		24-Oct-18	
Piezometer/Well No.:		BH/MW 1	
Ground level:		233.40	m
Screen top level:		230.40	m
Screen bottom level:		227.30	m
Test El. (at midpoint of screen):		228.85	m
Test depth (at midpoint of screen):		4.55	m
Screen length	L=	3.1	m
Diameter of undisturbed portion c	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.416	m
Initial water depth		1.45	m
Aquifer material:		**SILTY SAND TILL**	

Shape factor F= $\dfrac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.83401 m

Permeability K= $\dfrac{3.14 \times r2}{F \times (t2 - t1)}$ x ln (H1/H2) (Bouwer and Rice Method)

$\dfrac{\ln(H1/H2)}{(t2 - t1)}$ = 0.00118892

K= **4.0E-05** cm/s
4.0E-07 m/s



Falling Head Test (Slug Test)

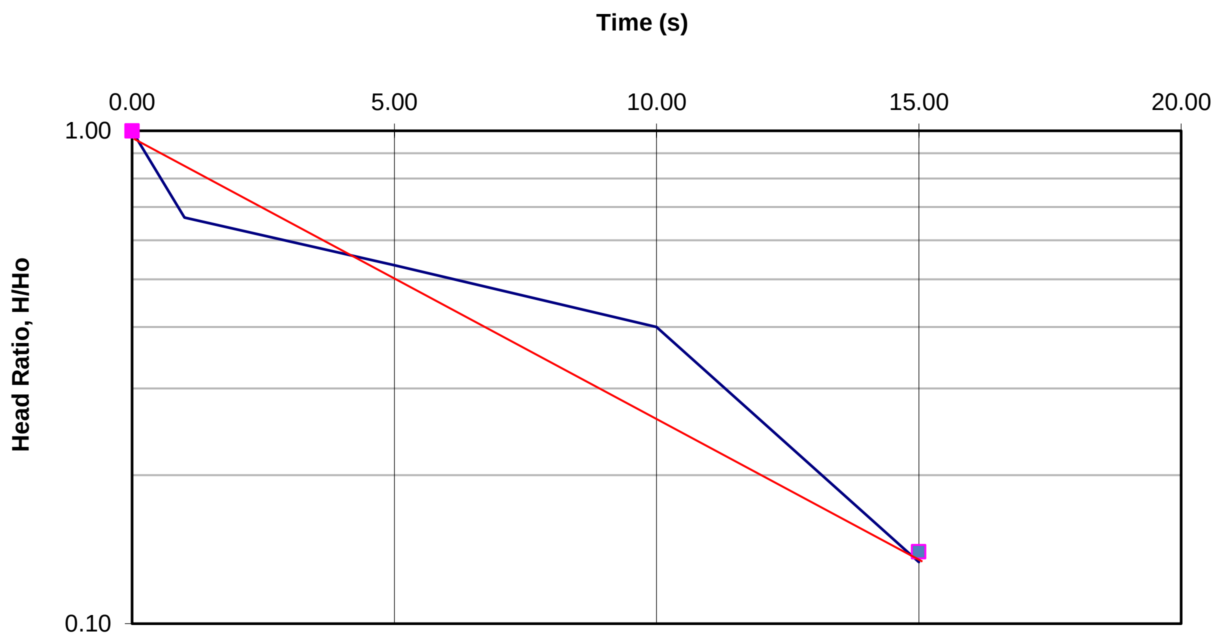
Test Date:		24-Oct-18		
Piezometer/Well No.:		BH/MW 2		
Ground level:		232.40	m	
Screen top level:		229.30	m	
Screen bottom level:		227.80	m	
Test El. (at midpoint of screen):		228.55	m	
Test depth (at midpoint of screen):		3.85	m	
Screen length	L=	3.1	m	
Diameter of undisturbed portion c	2R=	0.22	m	
Standpipe diameter	2r=	0.05	m	
Initial unbalanced head	Ho=	-0.15	m	
Initial water depth		2.76	m	
Aquifer material:		**SANDY SILT TILL**		

Shape factor F= 2 x 3.14 x L / ln(L/R) = 5.83401 m

Permeability K= 3.14 x r2 / F x (t2 - t1) x ln (H1/H2) (Bouwer and Rice Method)

ln (H1/H2) / (t2 - t1) = 0.13107419

K= **4.4E-03** cm/s
4.4E-05 m/s



Falling Head Test (Slug Test)

Test Date:		24-Oct-18	
Piezometer/Well No.:		BH/MW 3	
Ground level:		231.60	m
Screen top level:		228.60	m
Screen bottom level:		225.50	m
Test El. (at midpoint of screen):		227.05	m
Test depth (at midpoint of screen):		4.55	m
Screen length	L=	3.1	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.265	m
Initial water depth		1.77	m
Aquifer material:		**SILTY SAND TILL/FINE TO MEDIUM SAND**	

Shape factor F= $\frac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.83401 m

Permeability K= $\frac{3.14 \times r2}{F \times (t2 - t1)}$ x ln (H1/H2) (Bouwer and Rice Method)

$\frac{\ln(H1/H2)}{(t2 - t1)}$ = 0.00277259

K= **9.3E-05** cm/s
9.3E-07 m/s