APPENDIX

H

HYDROGEOLOGY REPORT

EXQUISITE DEVELOPMENTS

HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

78 - 82 EASTVIEW ROAD, GUELPH ONTARIO

FEBRUARY 19, 2019

HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

78 - 82 EASTVIEW ROAD, GUELPH ONTARIO

EXQUISITE DEVELOPMENTS

PROJECT NO.: 17M-015626-00

DATE: FEBRUARY 19, 2019

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Additional limitations to this report are presented in the final section of the report.

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1 INTRODUCTION

WSP Canada Group Limited (WSP) was retained by Exquisite Developments (2589615 Ontario Inc.) to complete a Scoped Environmental Impact Study (EIS) for a proposed re-development at 78 and 82 Eastview Road, Guelph Ontario. The study area for the EIS will encompass the subject property and natural features within 120 meters of the subject property. This report addresses the geology and hydrogeology components of the EIS.

The subject property encompasses 3.25 hectares and is bounded by Eastview Road to the south, residential properties fronting onto Starwood Drive to the East and is primarily bounded by a large natural area to the west and north. A small residential development is located adjacent to the southwest limit of the property at 66 Eastview Road. The closed Eastview Road Landfill is located approximately 400 meters east of the site on Eastview Drive. The site location is shown on attached **Figure 1**. Current land use at the site includes two residential properties and natural areas. The site is proposed to be developed into a townhouse complex with approximately fifty-seven (57) townhouses, one (1) storm water management pond and natural areas including a wetland and wetland buffer. The complete proposed site plan is provided in **Appendix A**.

The following **Table 1** provides a summary of the approximate existing and proposed pervious and impervious areas of the site based on Google Earth (2018) and the proposed site plan (**Appendix A**).

Table 1: Summary of Site Area

Land	Existing		Proposed	
	Area (square meters (m^2))	Percent (%)	Area (m^2)	Percent (%)
Pervious	31,665	97.5	20,657	63.58
Impervious	825	2.5	11,833	36.42
Total	32,490	100	32,490	100

Based on the proposed plan the pervious land area will decrease by approximately 33.9 % following construction of the proposed development. It is noted that the proposed storm water management pond will occupy approximately 1,430 m^2 of pervious land at the site.

WSP has recently completed work in this area, including a monitoring study at 66 Eastview Road, which is directly adjacent to this site. This work included monitoring the surface and groundwater levels for the wetland on the north side of the property. Premier Environmental Services Inc. completed Phase 1 Environmental Site Assessments at 78 and 82 Eastview Road, including background reviews of available information as well as a visit to each property. Premier Environmental Services Inc. also completed a Groundwater and Soil Sampling and Analysis program at these properties, which included sampling two (2) existing monitoring wells and collecting two (2) shallow soil samples. GeoPro Consulting Limited completed preliminary geotechnical investigations at both properties, which included drilling three (3) to four (4) shallow boreholes at each of the properties. All of these studies will be discussed further in the following relevant sections. The complete geotechnical reports are provided in **Appendix B**.

2 BACKGROUND

2.1 CLIMATE

WSP reviewed climate data from the closest available Government of Canada weather station, the Kitchener-Waterloo weather station (located approximately 16 kilometers southwest of the site), for the last five years (2014 to 2018). The following **Figure 2** depicts the historical temperature and precipitation.

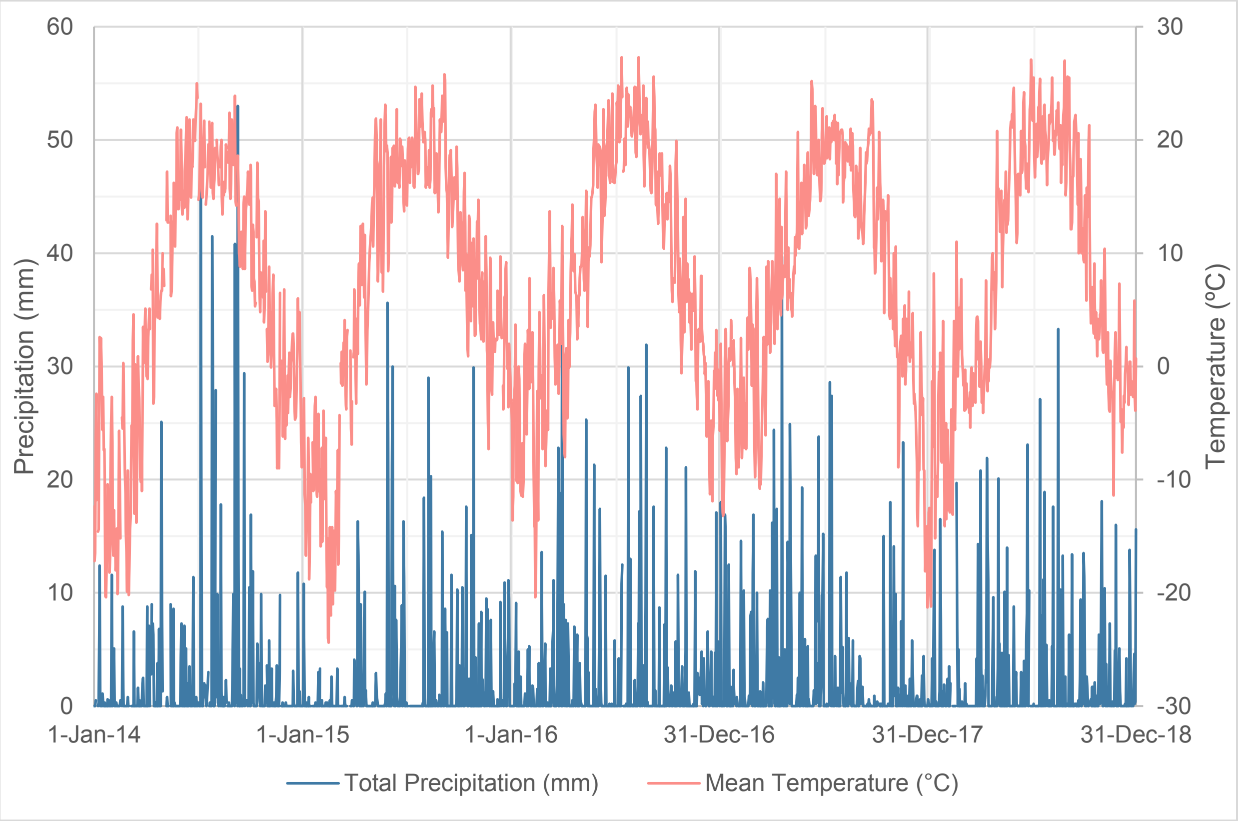

Figure 2: Historical Climate Data (Government of Canada, 2018)

Climate data recorded at the Kitchener-Waterloo weather station follows typical seasonal trends for Southwestern Ontario, with high temperatures in the summer months and low temperatures in the winter months and moderate precipitation year-round. Mean daily temperatures over the last five years have reached a maximum of 27.3 degrees Celsius (ºC) and a minimum of -24.4 º C, with an average of 7.3 ºC. Total daily precipitation over the last five years has ranged from 0 millimeters (mm) to 53 mm, with an average of 2.1 mm. From 2014 to 2018 the annual precipitation ranged from 637 mm/year to 818 mm/year with an average of 740 mm/year.

Parkin’s et al. (1999) approximated the following annual climate values for the Guelph area:

- Precipitation: 862 mm/year;
- Evapotranspiration: 497 mm/year;
- Surplus (365 mm/year):

- Infiltration: 284 mm/year; and
- Runoff: 81 mm /year.

Although this precipitation value is higher then the calculated average of the last five years, it is still within the range of the precipitation of the last five years.

2.2 NATURAL FEATURES

The site is mainly surrounded by recent residential developments and natural areas. The Guelph Northeast Wetland Complex, a provincially significant wetland surrounded by wooded areas, is present on 78 Eastview Road and extends to the north and west of the site. A portion of this wetland on the adjacent 66 Eastview Road property was monitored by WSP since 2014 for pre, during and post construction of the residential development on this property. As per the proposed plan this area will remain natural during and after development. Additionally, the Hadati Creek, a warm watercourse, and Clythe Creek Wetland Complex, a provincially significant wetland, are located to the southeast of the site. Natural features are shown on attached **Figure 3**.

It is interpreted that the Guelph Northeast Wetland Complex outlets to the Speed River Tributary north of the site as shown on **Figure 1**. It is understood that the topography is relatively flat in the area and the quaternary watershed shows the water draining towards the south (Hadati Creek). However, based on an assessment of the topography and existing conditions at the site is interpreted that flow is towards the north. No culverts are present along Eastview Road, with the exception of the Hadati Creek culvert to the east. Culverts are present along Speedvale Avenue East and Wellington Road 24 (as shown on **Figure 1**), further supporting that flow is towards the north.

2.3 PHYSIOGRAPHY AND TOPOGRAPHY

The study area is in the physiographic region of Guelph Drumlin Field as per Chapman and Putnam (1984). The drumlins in the region are aligned in the North-West direction. The drumlins are sparse with extensive low-lying areas that are covered by fluvial materials. The field is characterized by parallel valleys running at near right angles to the trend of the drumlins. The bottoms of these valleys are found to be swampy (Chapman and Putnam,1984). Chapman and Putnam (1984) describe the area as a sloping plain with the topography varying between 1,000 and 1,400 feet above sea level. Topographic contours are shown on attached **Figure 3**. The site is relatively flat, with a slight slope downward slope to the north.

2.4 GEOLOGY

2.4.1 REGIONAL GEOLOGY

The surficial geology of the study area consists of glaciofluvial deposits. These deposits include proglacial and deltaic deposits primarily composed of sand and gravel. Surficial geology is shown on attached **Figure 4**. The study area is also encompassed by Wentworth Till which is characterized by a silt to sandy silt matrix and low to moderate clast content. The bedrock underlying the study area belongs to the Guelph Formation (Silurian age) that consists of buff to cream colored crystalline dolomite.

2.4.2 SITE-SPECIFIC GEOLOGY

Three (3) boreholes were drilled by AECOM as a part of the nearby Closed Eastview Road Landfill monitoring in 2003, 2011 and 2012. Seven (7) boreholes were drilled by GeoPro Consulting Limited in 2016 and 2017 as a part of preliminary geotechnical investigations for 78 and 82 Eastview Road. Boreholes drilled by AECOM were completed into the bedrock at depths around 34 mbgs, and boreholes drilled by GeoPro Consulting Limited were completed in overburden at depth of approximately 4 mbgs.

As expected from the geology of the region the boreholes encounter the dolostone bedrock overlain by sandy silt and/or sandy silt till. As a part of the geotechnical investigation by GeoPro Consulting Limited (2017), the lithology of the study area is characterized by overlying topsoil, fill material, sandy silt to sand and silt and sandy silt till respectively. The bedrock at the site is composed of Dolostone from Lockport Formation and is characterized by fine to medium crystalline, thinly bedded dolostone. The complete geotechnical reports in **Appendix B**.

2.5 HYDROGEOLOGY

2.5.1 REGIONAL HYDROGEOLOGY

Groundwater in the region occurs in both the bedrock and overburden due to their favorable composition. Bedrock aquifers of the Guelph Formation act as the major regional aquifer in the study area with most of the domestic/private wells completed into the bedrock encountered at around 28 meters below ground surface (mbgs). The overburden deposits composed of sand and gravel also represent good aquifers that are often hydraulically connected to the underlying bedrock aquifer. The till at the site acts as aquitards due to the presence of clays and finer sediments.

2.5.2 SITE-SPECIFIC HYDROGEOLOGY

The study area exhibits similar hydrogeological conditions as the regional hydrogeology with the boreholes drilled in the area encountering water in the overburden at depths ranging from 2.1 m BGS to approximately 4.26 m BGS. As expected the aquifers carrying the water are composed of sand and gravel or sand and silt. The groundwater levels indicate a shallow overburden groundwater flow from northeast to southwest of the study area as per GeoPro Consulting Limited (2017). WSP collected groundwater levels at monitoring wells on site on November 23, 2018, as follows:

- MW02-12: 4.26 mbgs; and
- MW05-12: 3.26 mbgs.

2.5.3 MECP DATABASES

There are no Permits to Take-Water (PTTW’s), Environmental Activity and Sector Registry (EASR - water takings), Provincial Water Quality Monitoring Network (PWQMN) stations or Provincial Groundwater

Monitoring Network (PGMN) stations within the vicinity of the site as per the MECP (2018). Three (3) Environmental Compliance Approval (ECA) were found within the study area, as summarized below in **Table 2**.

Table 2: Active ECA's in the Study Area

Approval Number	Business Name	Address	Date	Approval Type	Status
5767-82ZTQH	Jason David Evans	50 Elginfield Dr	2010-03-01	ECA-Waste Management Systems	Approved
1473-8UXRUZ	The Corporation of the City of Guelph		2012-06-20	ECA-Municipal and Private Sewage Works	Approved
1565-5M6RQW	Cox Construction Limited	687 Eramosa Road	2003-05-09	ECA-Waste Management Systems	Approved

As per above these ECA's are for a waste management system and a municipal and private sewage works. The ECA locations are shown on attached **Figure 5**.

2.5.4 MECP WATER WELL RECORDS

As per the MECP (2018) Water Well Records (WWR) there are nine (9) WWRs present within 250 meters of the site, as summarized in the following **Table 3**.

Table 3: MECP Water Well Records

Well ID	Completion Date	Well Use		Well Depth (mbgs)	Static Water Level (mbgs)
6701118	13-Nov-56	Domestic		35.1	10.7
6701121	8-Sep-58	Domestic		39.0	10.7
6701123	26-Mar-65	Domestic		36.9	9.1
6715658	13-Feb-06	Monitoring		9.3	
7156437		Abandoned			
7182711	1-Mar-12	Monitoring		34.0	
7188875	6-Jun-12				
7234987	26-Sep-14	Abandoned			
7240599	12-Mar-15				

Of the nine (9) WWRs, three (3) are domestic water supplies, two (2) are monitoring wells, two (2) are well abandonments and two (2) have unknown use. It is likely that the three (3) domestic wells are no longer in use and may have been abandoned, as the area has been under development. Reported well depths range from 9.3 to 39 mbgs. Reported static water levels range from 9.1 to 10.7 mbgs. The location of these wells is shown in **Figure 5**.

2.5.5 SOURCE WATER PROTECTION

The study area falls under the jurisdiction of Grand River Conservation Authority. It has been assigned the status of a Well Head Protection Area B (WHPA-B) (MECP, 2018). As per the Clean Water Act (CWA) (2006), a wellhead protection area is the area that surrounds the well through which contaminants are reasonably likely to move towards or reach the well. WHPA-B is the area where groundwater is estimated to take up to 2 years to the well from within the aquifer. This area is important to protect from bacteria and viruses from human and animal wastes as well as hazardous chemicals.

The study area is also classified as an Intake Protection Zone-3 (IPZ-3). Under the CWA (2006) water and land surrounding a surface water intake are to be protected in order to protect the surface water intake and are referred to as IPZs. An IPZ-3 identifies areas where certain land based activities could lead to conditions under which a contaminant could exceed a threshold in the raw water. Source Water Protection areas are shown on attached **Figure 6**.

2.6 ERIS REPORTS

Premier Environmental Services (2017) obtained ERIS (Environmental Risk Information Services) reports for 250-meter radius around both properties, no records were found for either property.

2.7 GROUNDWATER AND SOIL QUALITY

Premier Environmental Services Inc. (2017) also collected groundwater and soil samples at the site for laboratory analysis. Premier Environmental Services Inc. (2017) found that the soil meets the 2011 MOE Table 2 SCS for metals and Petroleum Hydrocarbons (PHCs) and the groundwater meets the 2011 MOE Table 2 SCS for metals and Volatile Organic Compounds (VOCs).

3 HYDROGEOLOGICAL ASSESSMENTS

3.1 UNSATURATED HYDRAULIC CONDUCTIVITY

WSP completed infiltration testing at three locations at the site, the specific locations are shown on attached **Figure 5**. Shallow (approximately 20 centimeters (cm) deep) and deep (approximately 60 cm deep) boreholes were hand augured into the silty sand, shallow and deep boreholes were within 50 cm of each other. The tests were completed using the Guelph Permeameter to determine unsaturated hydraulic conductivity. Two tests were completed at each borehole, first with a head of 5 cm and second with a head of 10 cm. The tests were completed with the combined reservoir method and water levels were measured periodically at consistent intervals. Unsaturated hydraulic conductivity calculations were completed based on the field measurements, the calculations were completed for the Soil Moisture method (2012). The complete calculations are provided in **Appendix C**. Calculations were completed using one head and two head methods. The following **Table 4** summarizes the calculated unsaturated hydraulic conductivities. It is noted that for the tests at GP18-2 the two-head test method was not applicable as the rate of the 5-cm head test was greater then the rate of the 10-cm head test.

Table 4: Guelph Permeameter Unsaturated Hydraulic Conductivity Averages

Borehole ID	Unsaturated Hydraulic Conductivity (meters per second (m/s))
GP18-1S	1.30E-05
GP18-1D	3.68E-06
GP18-2S	1.62E-05
GP18-2D	6.14E-06
GP18-3S	1.10E-05
GP18-3D	6.15E-06

Generally, the unsaturated hydraulic conductivities range within one order of magnitude for the three calculations completed for each borehole. Overall the unsaturated hydraulic conductivity ranged from $3.68\text{x}10^{-6}$ m/s to $1.62\text{x}10^{-5}$ m/s with an average of $9.36\text{x}10^{-6}$ m/s.

3.2 WATER BALANCE

An annual water balance was completed for the site based on the MOE method (2003). The approximate pre- and post construction pervious and impervious areas were used along with the precipitation and evapotranspiration values from (Parker et al., 1999). Infiltration was re-calculated for this site based on a local infiltration factor (MOE, 2003). This infiltration factor is based on three conditions of the site, topography, soils and cover. Based on the flat land (average slope <0.6m/km), a medium combination of clay and loam and the uncultivated land of the site, an infiltration factor of 0.55 was calculated for both the pre- and post construction periods. **Appendix D** includes the breakdown for calculating the infiltration factor. Based on this infiltration factor the annual infiltration was calculated as 201 mm/year (surplus value of 365 mm/year multiplied by the 0.55 infiltration factor) and the runoff value was calculated as the reminder of the surplus (164 mm/year).

Based on these climate values and the pre- and post construction areas a water balance was completed as summarized in the following **Table 5**. For the water balance it was assumed that only ten percent of evapotranspiration occurs on impervious areas from wetting and ponding in shallow depressions and that ninety percent of runoff occurs on impervious areas.

3.2.1 POTENTIAL LID BENEFITS

An additional water balance was completed assuming that Low Impact Development (LID) features would be incorporated at the site, particularly an infiltration trench which receives roof runoff. LIDs are storm water management techniques that act to reduce runoff at development sites. Infiltration trenches are geotextile fabric lined trenches filled with clean granular stone (void forming). Infiltration trenches have been shown to reduce runoff by 73 to 95% in Ontario as per CVC and TRCA (2010). For the proposes of this water balance exercise it was assumed the infiltration trench would receive some of the site’s roof runoff, approximately 2,695 m^2 based on the Draft Functional Servicing and Stormwater Management Report (MTE Consultants Inc., 2019). Additionally, 1,720 m^2 of the proposed landscaped area will also be conveyed to the infiltration trench as per the MTE Consultants Inc. (2019).

Table 5: Water Balance Summary

Category	Existing Undeveloped Site	Proposed Developed Site (no LIDs)	Difference between Existing Undeveloped Site and Proposed Developed (no LIDs)	Proposed Developed Site with LIDs	Difference between Existing Undeveloped Site and Proposed Developed Site with LIDs
Areas (m^2)					
Impervious	825	11,833			
Impervious (Roofs going to LID)				2,695	
Impervious (Excluding Roofs going to LID)				9,138	
Pervious	31,665	20,657			
Total	32,490	32,490			
Climate (mm/year)					
Precipitation	862	862			
Evapotranspiration	497	497			
Surplus (infiltration + runoff)	365	365			
Infiltration	201	201			
Runoff	164	164			
Annual Water Balance (m^3/year)					
Precipitation	**28,006**	**28,006**	**0**	**28,006**	**0**
Evapotranspiration (Pervious)	15,738	10,267	-5,471	10,267	-5,471
Evapotranspiration (Impervious)	71	1,020	949	1,020	949
Evapotranspiration (pervious + impervious)	**15,809**	**11,287**	**-4,522**	**11,287**	**-4,522**
Infiltration	**6,357**	**4,147**	**-2,210**	**6,238**	**-119**
Runoff (Pervious)	5,201	3,393	-1,808	3,393	-1,808
Runoff (Impervious)	640	9,180	8,540	7,089	6,449
Runoff (pervious + impervious)	**5,841**	**12,573**	**6,732**	**10,482**	**4,641**
TOTAL	28,006	28,006	0	28,006	0

Based on the findings of the annual water balance, there will be an approximate 24% increase in runoff and an approximate 24% decrease in infiltration and evapotranspiration at the site with no LIDS on site.

However, with the infiltration trench there will only be an 16.5% increase in runoff at the site. Additionally, runoff can be further reduced via other LID features such as bioretention which temporary stores and treats runoff before infiltration and increases evapotranspiration. Bioretention's are made up of mulched ground cover and plants overlying a filter bed consisting of sand and organic material and are ideal for small storm events. Factsheets for the infiltration trenches and bioretention LID methods are provided in **Appendix E**.

3.2.2 LID SPECIFICATIONS

The following calculations were completed as per CVC and TRCA (2010) to assess the sizing requirements for the recommend infiltration trench at the site.

The following calculation was completed to assess the depth of the infiltration trench. It was determined that the recommended maximum depth of the stone reservoir is 1.2 meters.

$$d_{r\,max} = (i)(t_s/V_r)$$

Where:

i is the native soils infiltration (which is 9.6 mm/hr for this site based on the Guelph Permeameter testing and when divided by a 3.5 Factor of Safety, which presumes that the soils within 1.5 meters of the proposed depth is equal to or higher then the actual infiltration rate of 33.7 mm/hr for the native soils determined through field testing (CVC and TRCA, 2010));

V_r is the stone void space ratio (typically 0.4 for 50 mm clear stone);

T_s is the time to drain (typically 48 hours); and

d_{rmax} is the maximum stone reservoir depth (1200 mm or 1.2 meters).

The following calculation was completed to assess the area of the infiltration trench. It was determined that the recommended surface area footprint is 283 m^2.

$$A_f = ((WQV)/(d_r)(V_r))$$

Where:

WQV is the water quality volume (128 m^3, based on the maximum precipitation in one day in the past 5 years of 53 mm and the approximate proposed area of roofs at 2,695m^2 with 90% runoff); and

A_f is the surface area footprint (278 m^2).

As such is recommended that a 278 m^2 infiltration trench be installed on site up to a depth of 1.2 m. The site-specific location can be determined based on the proposed construction. Ideally the layout of this area will have little to no construction traffic compacting the soil during construction. The site has relatively similar elevations with gentle slopes towards the wetlands, and groundwater is interpreted to follow the local topography with flow generally in the direction of the wetland. It is recommended that the infiltration trench location be considered in a central location to reduce the length of piping for roof runoff towards the infiltration trench. It is noted that the site is located in a two (2) year time of travel wellhead protection area and therefore the infiltration trench should not receive surface water runoff from roads or parking lots.

4 CONCLUSIONS AND RECOMMENDATIONS

Based on the Guelph Permeameter testing this site's shallow silty sand unit has an unsaturated conductivity of approximately 9.36×10^{-6} m/s. It is interpreted that surface water flow at the site is to the north, towards a tributary to the Speed River. Based on the water balance for this site, there will be an approximate 24% increase in runoff following construction of the proposed development from the existing conditions of the site.

It is recommended that Low Impact Developments (LIDs) be considered to reduce the anticipated runoff increase at this site. It is recommended that a 278 m^2 and 1.2-meter-deep infiltration trench be considered on site to contain some roof runoff and increase the infiltration at the site. Additionally, it is recommended that bioretention LIDs be considered at the site to further decrease the runoff via increased evapotranspiration and infiltration.

5 LIMITATIONS

This Report was prepared for the Client, Exquisite Developments, in accordance with the professional services agreement, solely for their exclusive use to provide an assessment of current environmental conditions in association with the Site. The intended recipient is solely responsible for the disclosure of any information contained in this report. The content and opinions contained in the present report are based on the observations and/or information available to WSP at the time of preparation. If a third party makes use of, relies on, or makes decisions in accordance with this report, said third party is solely responsible for such use, reliance or decisions. WSP does not accept responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken by said third party based on this report. This limitations statement is considered an integral part of this report.

The Report summarizes WSP's review of available data in accordance with the principal components of the stated regulations, standards and guidelines and the scope, terms and conditions of the contract or proposal to which the Assignment was conducted. No other warranties are either expressed or implied with respect to the professional services provided under the terms of the contract or proposal and represented in this Report. Conditions may exist which were not detected given the nature of the inquiry WSP was retained to undertake with respect to the Site. Additional environmental studies and actions may be recommended.

The Report is based on data and information collected at the time of this Assessment, as stated in the Report. Site use or conditions change and the information and conclusions in the Report may no longer apply following the date of this Report. If any conditions become apparent that differ significantly from that presented in this Report, we request that we be notified to reassess the conclusions and recommendations provided herein. WSP disclaims any obligation to update this Report for conditions that may be identified after the date of this Report; however, WSP reserves the right to amend or supplement this report based on additional information, documentation or evidence.

In evaluating the Site, WSP has relied in good faith on information provided by others, as noted in the Report. WSP has assumed that the information provided is correct and WSP assumes no responsibility for the accuracy, completeness or workmanship of any such information.

The Report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment.

The conclusions are based on the Site conditions observed by WSP at the time the work was performed and may include information obtained at specific testing and/or sampling locations. It is recognized that overall conditions can only be extrapolated to an undefined limited area around these testing and sampling locations. The conditions that WSP interprets to exist between testing and sampling points may differ from those that actually exist. The accuracy of any extrapolation and interpretation beyond the sampling locations will depend on natural conditions, the history of Site development and changes through construction and other activities. In addition, analysis has been carried out for the identified chemical and physical parameters only, and it should not be inferred that other chemical species or physical conditions are not present. WSP cannot warrant against undiscovered environmental liabilities or adverse impacts off-Site.

The conclusions presented in this Report are based on Work undertaken by trained professional and technical staff and the reasonable and professional interpretation of the information considered. Conclusions presented in this report should not be construed as legal advice. WSP makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in the Report, including, but not limited to, ownership of any property, or the application of any law to the findings of the Assessment.

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- Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

FIGURES

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wsp

78 - 82 EASTVIEW ROAD, GUELPH ONTARIO HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

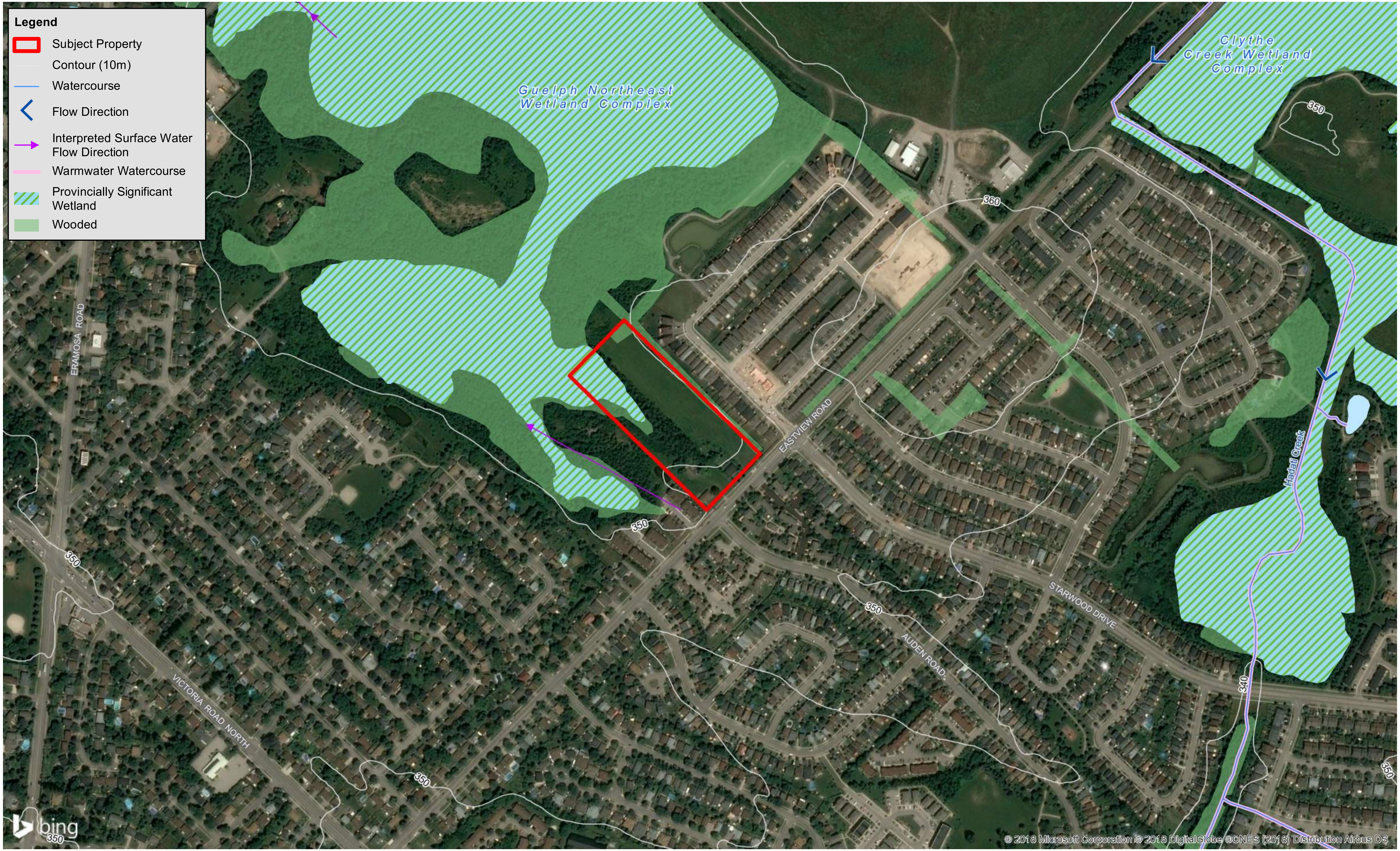
Site Location

0 100 200
metres
1:10,000
N

Date: December 2018

Project No: 17M-01526-00

Figure No: 1



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78 - 82 EASTVIEW ROAD, GUELPH ONTARIO HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

Natural Features

0 50 100 150
metres
1:5,000
N

Date: December 2018
Project No: 17M-01526-00
Figure No: 3

78 - 82 EASTVIEW ROAD, GUELPH ONTARIO HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

Surficial Geology

Date: November 2018

Project No: 17M-01526-00

Figure No: 4

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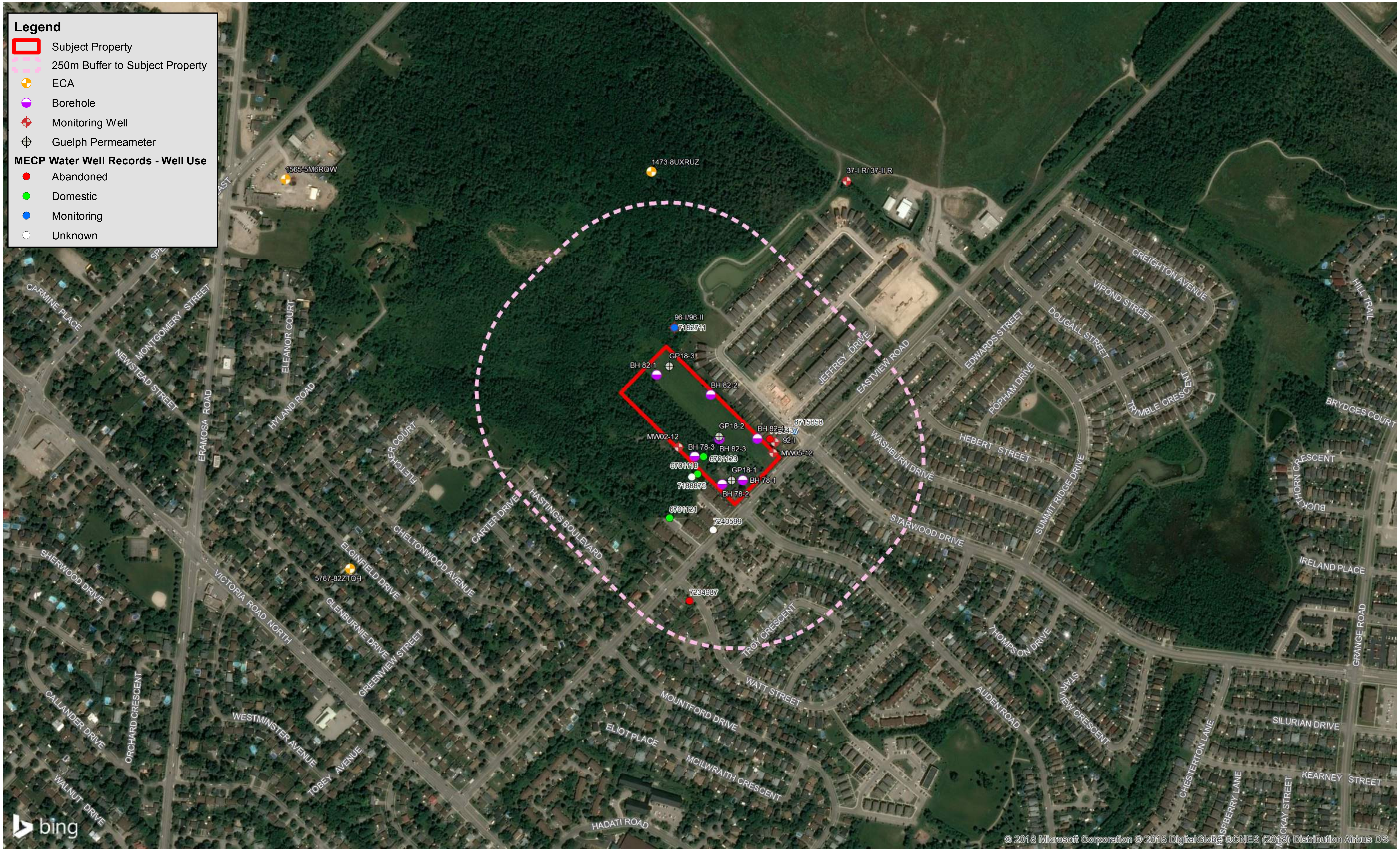

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78/82 EASTVIEW DEVELOPMENT ENVIRONMENTAL IMPACT STUDY

MECP Water Well Records

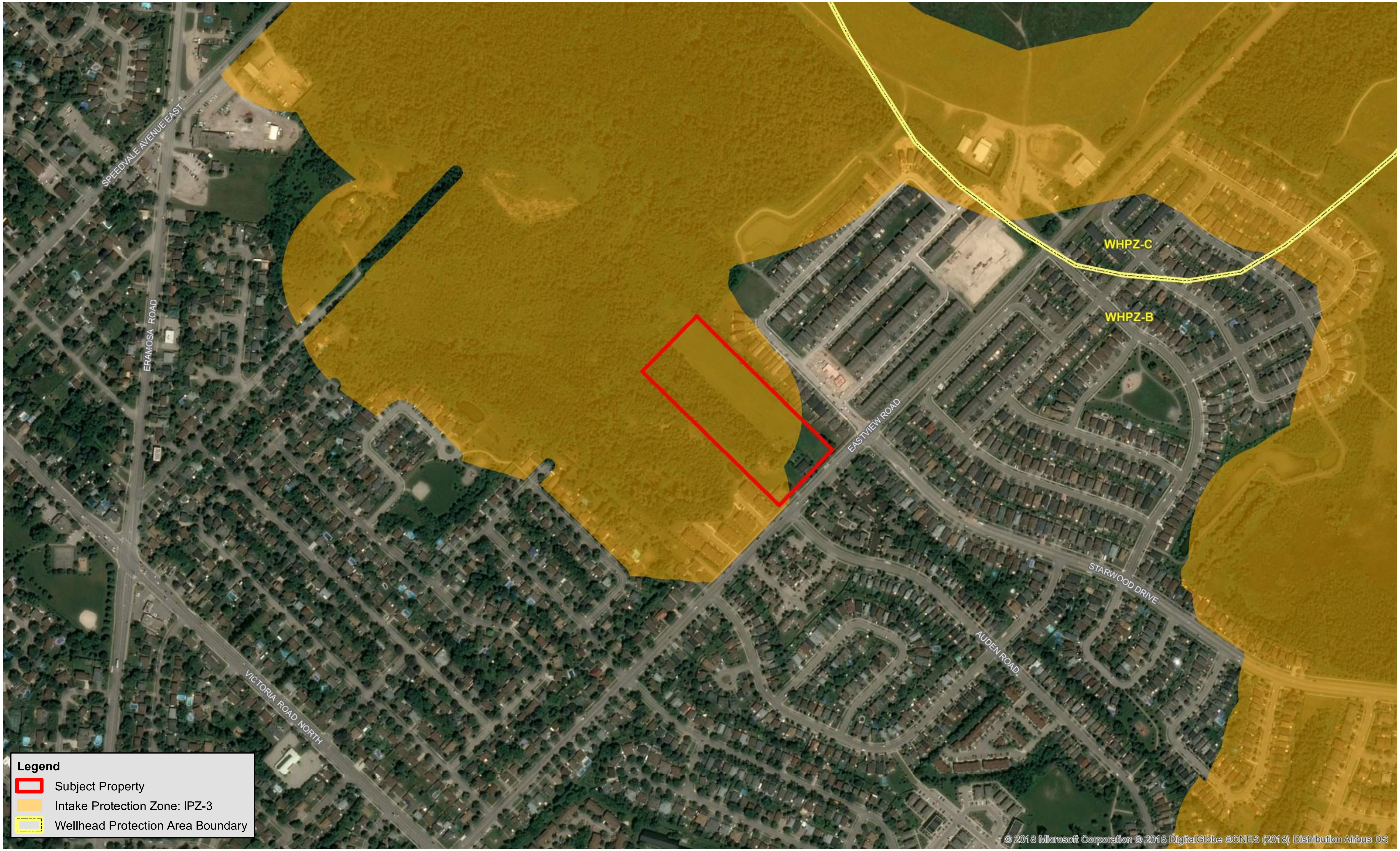
0 50 100 150
metres
1:6,000

N

Date: November 2018

Project No: 17M-01526-00

Figure No: 5



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78 - 82 EASTVIEW ROAD, GUELPH ONTARIO HYDROGEOLOGICAL ASSESSMENT AND WATER BALANCE

Wellhead Protection Area and Intake Protection Zone

Date: November 2018

Project No: 17M-01526-00

Figure No: 6

APPENDIX

A

CONCEPT PLAN

1 PROPOSED SITE PLAN

(BUILDABLE AREA)

scale: 1 : 400

R.3A-XX ZONING REQUIREMENTS

THE LEGAL DESCRIPTION & PROPERTY INFORMATION USED FOR THIS DRAWING HAS BEEN TRANSFERRED THE FOLLOWING SURVEYS PREPARED BY VAN HARTEN SURVEYING INC:

- PROJECT No. 18120–08, DATED: APRIL 24, 2008 (JEFFREY E. BUISMAN)
- PROJECT No. 18136–08, DATED: OCTOBER 15, 2008, (JEFFREY E. BUISMAN)
- PROJECT No. 17139–06, DATED NOVEMBER 11, 2008, (JAMES M. LAWS)
- PROJECT No. 19408–10, DATED: AUGUST 23, 2012, (JEFFREY E. BUISMAN) REGISTERED PLAN 61R–11921 (AUG 30, 2012)
- PROJECT No. 22659–15, DATED MARCH 5, 2015, (JAMES M. LAWS) REGISTERED PLAN 61R–20768 (FEB 18, 2016)

REGULATIONS	REQUIRED	PROVIDED	CONFORMS
MINIMUM LOT AREA	800 m²	15,433.5 m² (1.543 Ha)	✓
MINIMUM LOT AREA PER DWELLING UNIT	270 m²	302.6 m²	✓
MINIMUM LOT FRONTAGE	11.00 m	11.055 m (ARKELL RD.)	✓
MINIMUM FRONT YARD	6.00 m + ROAD ALLOWANCE	6.00 m + ROAD ALLOWANCE	✓
MINIMUM EXTERRIOR SIDE YARD	15 m	25.2 m (GORDON ST.)	✓
MINIMUM SIDE YARD	1/2 the building height, but not less than 3.00 m	AS NOTED	✓
MINIMUM REAR YARD	1/2 building height, but not less than 3.00m and 12.0m with a 10.0m buffer (min.) where abutting residential	4.520 m	✓
MAXIMUM BUILDING COVERAGE (% OF LOT AREA)	30 %	24.2 % (3736 m²)	✓
MAXIMUM BUILDING HEIGHT	3 Storeys	3 Storeys	✓
MAXIMUM BUILDING HEIGHT ABUTTING RESIENTIAL	2 Storeys – within 30 m of the Eastern property line abutting residential	2 Storeys	✓

PIN 71186–0066 & PIN 71186–0069 of the REGISTERED PLAN 61R–11921, DATED AUGUST 30, 2012. PLAN of SURVEY of PART 3 of PART of LOT 7, CONCESSION 8 of the REGISTERED PLAN 61R–20768, DATED FEBRUARY 18, 2016. Geographic Township of Puslinch, CITY OF GUELPH, COUNTY OF WELLINGTON. JEFFREY E. BUISMAN – ONTARIO LAND SURVEYOR. PREPARED by VAN HARTEN SURVEYORS INC.

ZONING CATEGORY: R.3A–XX XXX SPECIALIZED CLUSTER TOWNHOUSE ZONE

REGULATIONS	REQUIRED	PROVIDED	CONFORMS
MINIMUM DISTANCE BETWEEN BUILDING FACES/WINDOWS	15.0 m	AS NOTED	✓
MINIMUM DISTANCE BETWEEN BUILDINGS	3.0 m	3.0 m	✓
MINIMUM PRIVATE AMENITY AREA	20 m² per UNIT	20 m² per UNIT	✓
MINIMUM COMMON AMENITY AREA	5 m² per DWELLING UNIT TOTAL = 255 m²	292.2 m²	✓
MINIMUM LANDSCAPED OPEN SPACE (% OF LOT AREA)	40 %	44.79 % (6,913.64 m²)	✓
BUFFER STRIP	3.0 m (SIDE & REAR YARDS)	3.0 m (SIDE & REAR YARDS)	✓
FENCES	SIDE YARD HEIGHT 1.9 m MAX	1.8 m HEIGHT BETWEEN PRIVATE AMENITY SPACES	✓
ACCESSORY BUILDINGS OR STRUCTURES	NOT APPLICABLE		✓
MAXIMUM NUMBER OF DWELLING UNITS IN A ROW	12	12	✓
GARBAGE, REFUSE STORAGE AND COMPOSTERS	YES	2.0 m² WITHIN EACH GARAGE + MUNICIPAL CURBSIDE COLLECTION	✓
MAXIMUM DENSITY OF SITE	50 UNITS/Ha = 76 max	51 UNITS PROVIDED	✓

OFF STREET PARKING

PARKING REGULATION	REQUIRED	PROVIDED	CONFORMS
PARKING RATIO	1 per UNIT = 55	110	✓
VISITOR PARKING RATIO:	20% OF REQUIRED PARKING = 11	18 (includes 2 Barrier Free Spaces)	✓
PARKING SPACE SIZE:	2.75 m X 5.50 m (EXTERIOR) 3.00 m X 6.00 m (INTERIOR)	2.75 m X 5.50 m (EXTERIOR) 3.00 m X 6.00 m (INTERIOR)	✓
ACCESSIBLE PARKING SPACE SIZE(S)	TYPE A: 3.40 m X 5.50 m TYPE B: 2.40 m X 5.50 m	TYPE A: 3.40 m X 5.50 m (with 2.0 m AISLE)	✓
ACCESSIBLE PARKING SPACES REQUIRED	51 to 200 SPCAES = 2	2 SPACES	✓
BICYCLE PARKING	1 SPACE pre 20 UNITS = 2.8	6 SPACES	✓

SITE STATISTICS

TOTAL GROSS FLOOR AREA:	BLDG 1	BLDG 2	BLDG 3	BLDG 4	BLDG 5	BLDG 6
TOTAL AREA OF ALL FLOORS ABOVE GRADE (1st floor + 2nd floor, including garages)	701.45 m²	1645.31 m²	1060.96 m²	1971.59 m²	1413.32 m²	1391.00 m²
TOTAL AREA OF ALL FLOORS ABOVE GRADE (1st floor + 2nd floor, and NOT including garages)	701.45 m² garage at basement level	1405.05 m²	915.39 m²	1683.20 m²	1219.24 m²	1391.00 m² garage at basement level

KEY PLAN

PART 3 of the PIN 71186–0066(LT); PART OF LOT 7, CONCESSION 8

PIN 71186–0069(LT); PART OF LOT 7, CONCESSION 8

🟦	BUILDABLE AREA
▨	SWM AREA

OPTION -04

2018-12-19

(On New Surevy)

57 UNITS

NOTE:

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LEGEND

- PROPERTY LINE
- SET BACK LINE
- FIRE ROUTE — CENTER LINE OF TRUCK ACCESS ROUTE with 12m TURNING RADIUS. (12.0 m)
- PROPOSED PRIVACY FENCE
- ▶ ENTRANCE / EXIT
- Ⓢ Rb–xx STANDARD TRAFFIC SIGNS c/w GALVANIZED POSTS
- B.F. ACCESS RAMP
- ▨ AREA OF NEW LANDSCAPING
- AREA OF NEW ASPHALT
- XXX
- FIRE WALL
- BARRIER CURB
- NEW CONCRETE WALKS
- ▦ NEW PAVING / DECKS
- XXX
- FH NEW FIRE HYDRANT
- EUP EXISTING UTILITY POLE
- NEW LIGHT STANDARD
- T TRANSFORMER PAD
- BF BICYCLE PARKING
- STANDARD PARKING SPACE: 5.5m x 2.75m (5.500 m; 2.750 m)
- TYPICAL AISLE WIDTH: 7.0m
- DESIGNATED ACCESSIBLE PARKING SPACE (D.A.P.): TYPE A: 3.4m x 5.5m TYPE B: 2.4m x 5.5m + 2.0m ACCESS AISLE (5.500 m; 2.0m; VARIES)

No.	REVISIONS	date

No.	REVISIONS	date
02		
01	XXX	YYYY MM.DD

PROJECT NORTH

TRUE NORTH

ONTARIO ASSOCIATION OF ARCHITECTS
BARRY R. JOHNSON
LICENCE
3984

scale	1 : 400
drawn	CRT
last worked on	2018.12.19
by	CRT
checked	BRJ
print date	2018.MM.DD

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client
EXQUISITE HOMES
221 HELEN AVE
MARKHAM ONTARIO

project
SITE DESIGN
NEW TOWNHOUSES
GUELPH
EASTVIEW STREET
GUELPH ONTARIO

drawing title
PROPOSED
SITE PLAN

reference

project no. 18–138

client reference number

sheet no. A-101

APPENDIX

B

GEOTECHNICAL REPORTS

PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED SUBDIVISION DEVLOPMENT 78 EASTVIEW ROAD GUELPH (GEOPRO, 2017);
PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED SUBDIVISION DEVLOPMENT 82 EASTVIEW ROAD GUELPH (GEOPRO, 2016)

GeoPro Consulting Limited

Geotechnical-Hydrogeology-Environmental-Materials-Inspection

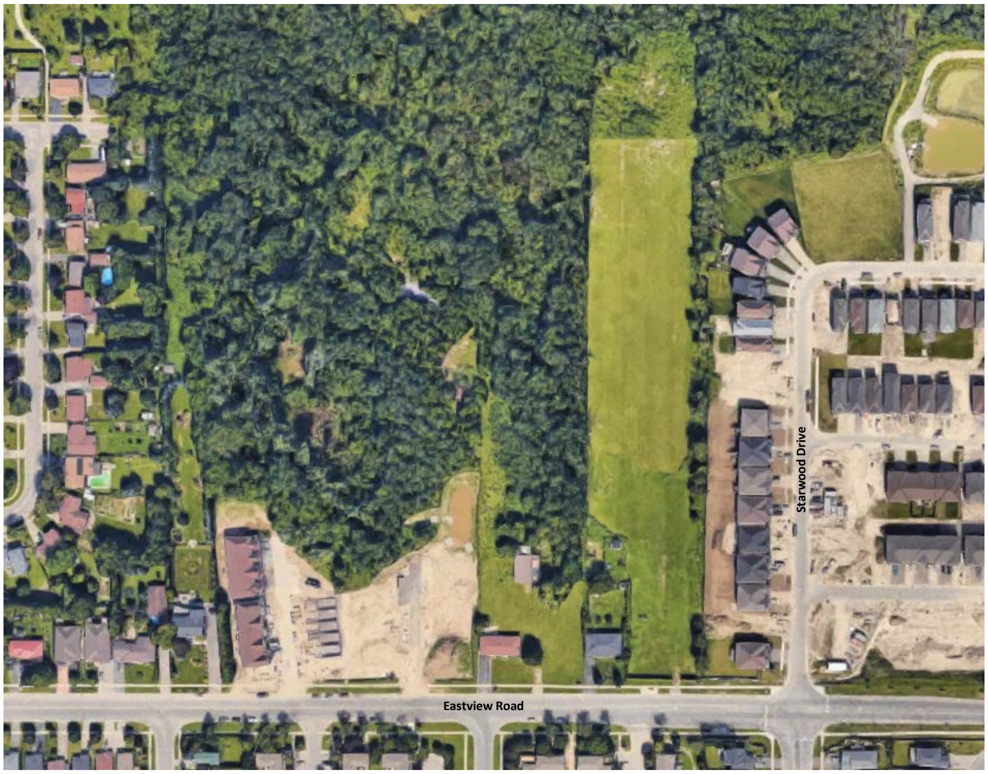
Preliminary Geotechnical Investigation

Proposed Subdivision Development

78 Eastview Road, Guelph, Ontario

Prepared For:

Best Homes Canada



GeoPro Project No.: 16-1696G2

Report Date: September 13, 2017

Table of Contents

1. INTRODUCTION

GeoPro Consulting Limited (GeoPro) was retained by Best Homes Canada (the Client) to undertake a preliminary geotechnical investigation to support the acquisition due diligence for the proposed subdivision development located at 78 Eastview Road, Guelph, Ontario.

The purpose of this preliminary geotechnical investigation was to obtain preliminary information on the existing subsurface conditions by means of a limited number of boreholes, in-situ tests and laboratory tests of soil samples to provide required preliminary geotechnical design information. Based on GeoPro's interpretation of the data obtained, preliminary geotechnical comments and recommendations related to the general planning and project concept design purposes are provided.

The report is prepared with the condition that the design will be in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice. Further, the recommendations and opinions in this report are applicable only to the proposed project as described above. On-going liaison and communication with GeoPro prior to and during the detailed design stage and construction phase of the project is strongly recommended to confirm that the preliminary recommendations in this report are applicable and/or correctly interpreted and implemented. Also, any queries concerning the geotechnical aspects of the proposed project shall be directed to GeoPro for further elaboration and/or clarification.

This report is provided on the basis of the terms of reference presented in our approved proposal prepared based on our understanding of the project. If there are any changes in the preliminary design and planning features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations can be relied upon.

This report deals with geotechnical issues only. The geo-environmental (chemical) aspects of the subsurface conditions, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, were not investigated and were beyond the scope of this assignment.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice in Ontario.

This report has been prepared for the Client. Third party use of this report without GeoPro's consent is prohibited. The limitation conditions presented in this report form an integral part of the report and they must be considered in conjunction with this report.

2. FIELD AND LABORATORY WORK

The field work for the geotechnical investigation was carried out on August 21, 2017, during which time three (3) boreholes (Boreholes BH1 to BH3) were advanced at the locations shown on the Borehole Location Plan, Drawing No. 1. The boreholes were drilled to depths ranging from 2.4 m to 4.0 m below the existing ground surface.

The boreholes were advanced using a continuous flight auger drilling equipment supplied by a drilling specialist subcontracted to GeoPro. Samples were retrieved with a 51 mm (2 inches) O.D. split-barrel (split spoon) sampler driven with a hammer weighing 624 N and dropping 760 mm (30 inches) in accordance with the Standard Penetration Test (SPT) method. The number of blows required to drive the sampler 300 mm (12 inches) depth into the undisturbed soil (SPT N values) gives an indication of the compactness condition or consistency of the sampled soil material.

Groundwater condition observations were made in the boreholes during drilling and upon completion of drilling. The boreholes were backfilled and sealed upon completion of drilling.

The field work for this investigation was monitored by a member of our engineering staff, who also determined the approximate borehole locations in the field, logged the boreholes and cared for the recovered samples. The boreholes were located and staked in the field by GeoPro according to the underground utility conditions and the accessibility of the drill rig.

All soil samples obtained during this investigation were brought to our laboratory for further examination. These soil samples will be stored for a period of three (3) months after the day of issuing draft report, after which time they will be discarded unless we are advised otherwise in writing. Geotechnical classification testing (including water content, grain size distribution and Atterberg Limits, when applicable) were carried out on selected soil samples.

The ground surface elevations of the as-drilled borehole locations were not available at the time of preparing the report. Therefore, the stratigraphy of each borehole location was referenced to the current grade level. The borehole locations plotted on the Borehole Location Plan, Drawing No. 1, were based on the measurement of site features and should be considered to be approximate. Contractors performing the work should confirm the elevations prior to construction.

3. SITE AND SUBSURFACE CONDITIONS

The borehole locations are shown on Drawing No. 1. Notes on sample descriptions are presented in Enclosure No. 1A. An explanation of terms used in the borehole logs is presented in Enclosure No. 1B. The subsurface conditions in the boreholes (Boreholes BH1 to BH3) are presented in the individual borehole logs (Enclosure Nos. 2 to 4 inclusive). The detailed descriptions of the major soil strata encountered in the boreholes drilled at the site are provided as follows.

3.1 Soil Conditions

Topsoil

Topsoil with thickness ranging from 180 mm to 260 mm was encountered surficially in all boreholes.

Fill Materials

Fill materials consisting of sandy silt was encountered below the topsoil in Borehole BH1, and extended to a depth of about 1.1 m below the existing ground surface. SPT N values ranging from 7 to 17 blows per 300 mm penetration indicated a loose to compact compactness.

Probable Fill Materials

Probable fill materials consisting of silty sand and sandy silt to sand and silt were encountered below the topsoil in Boreholes BH2 and BH3, and extended to depths ranging from about 1.8 m to 2.3 m below the existing ground surface. SPT N values ranging from 7 to 10 blows per 300 mm penetration indicated a loose to compact compactness.

Sandy Silt to Sand and Silt

Sandy silt to sand and silt deposits was encountered below the (probable) fill materials and within sandy silt till deposit in Borehole BH1 and BH2, and extended to depths ranging from 2.1 m to 3.3 m below the existing ground surface. Borehole BH2 was terminated in these deposits. SPT N values ranging form 14 to greater than 100 blows per 300 mm penetration indicated a compact to very dense compactness.

Sandy Silt Till

Sandy silt till deposit was encountered below the probable fill materials, sandy silt, and sandy silt to sand and silt deposits in Boreholes BH1 and BH3, and extended to depths ranging form 2.9 m to 4.0 m below the existing ground surface. Boreholes BH1 and BH3 were terminated in this deposit. SPT N values ranging from 32 blows per 300 mm penetration to 71 blows per 230 mm penetration indicated a dense to very dense compactness.

3.2 Groundwater Conditions

The groundwater conditions were observed in the boreholes during and immediately upon completion of drilling. Summary of the observations are shown in the borehole logs and in the following table:

Borehole No.	Borehole Depth (m)	Depth of Water Encountered during Drilling (mBGS)	Water Level upon Completion of Drilling (mBGS)	Cave-in Depth upon Completion of Drilling (mBGS)
BH1	4.0	2.4	Dry	Open
BH2	2.4	-	-	2.3
BH3	3.1	-	-	2.8

Note: mBGS = meter below ground surface

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to weather events.

4. PRELIMINARY GEOTECHNICAL INFORMATION

This report contains the findings of GeoPro's preliminary geotechnical investigation, together with the preliminary geotechnical engineering recommendations and comments. These preliminary recommendations and comments are based on factual information and are intended only for use by the design engineers. The number of boreholes are not sufficient for the detailed design and are not sufficient to determine all the factors that may affect construction methods and costs. Once the actual development plans are available, the information in this report should be reviewed by the geotechnical engineer from GeoPro and an additional detailed geotechnical investigation shall be carried out, compatible with the actual proposed development plans for the site.

It is understood that the proposed development consists of residential subdivision. Based on the results of this preliminary geotechnical investigation, the native soils encountered at the site are generally considered to be suitable for supporting the proposed development. The following preliminary geotechnical information is provided for the planning and preliminary design of subdivision development at the site.

- The existing fill materials and probable fill materials were encountered below the topsoil and extended to depths ranging from 1.1 m to 2.3 m below the existing ground surface at the site. The topsoil and fill materials are considered to be not suitable for supporting the residential houses (with or without basement) and any other settlement sensitive structures. The topsoil, existing fill materials and probable fill materials are also considered to be not suitable for supporting the engineered fill.

- Depending upon the final site grading scheme and proposed basement elevations, the areas should be brought up to the underside of the footings, as required, using engineered fill. The materials proposed for use as engineered fill should be approved by qualified geotechnical personnel from GeoPro at the source, prior to hauling to the site. Most of the native soils at the site would be suitable for reuse as engineered fill. Imported materials meeting the requirements of OPSS Select Subgrade Material (SSM) would also be suitable for use as engineered fill. Details regarding placement and compaction requirements for engineered fill, if utilized at the site, can be provided once the actual development plans are available, as part of the final geotechnical recommendations for the project.

- The native subsoils at the site are considered to be suitable for supporting conventional residential houses (with or without basements). A preliminary allowable bearing pressure of 150 kPa may be assumed for conventional shallow spread and/or strip footings bearing in the native, undisturbed subsoils, at depths approximately ranging from 1.1 m to 2.3 m below the existing ground surface (or deeper as required for basements). Footings founded on approved engineered fill, if utilized at the site, may be designed using a preliminary allowable bearing pressure of 150 kPa.

- The type of foundation drainage system required (perimeter drains and/or under slab drains; damp-proofing or water-proofing) depends upon the proposed founding elevations, soil types in the area and actual stabilized groundwater levels. Based on the results of this preliminary investigation, it is anticipated that conventional foundation drainage (i.e. perimeter drains and damp-proofing) would be adequate at most locations. Basements founded in sandy/silty soils below the groundwater level (if any) may require water-proofing and/or under slab drains. In any event, the type of foundation drainage should be confirmed by the geotechnical engineer once the site grading plans are available, as part of final design process.

- All exterior footings and footings in unheated areas should be protected with a minimum of 1.4 m of earth cover for frost protection.

- Based on the results of this preliminary investigation, for shallow excavations above the prevailing groundwater tables, groundwater control during excavation can probably be handled, if required, by pumping from properly constructed and filtered sumps located at the base of the excavation. However, more significant groundwater seepage will be expected from the sandy/silty deposits below the prevailing groundwater tables. Some form of positive groundwater control will be required to maintain the stability of the base and side slopes of the excavations in these areas, in addition to pumping from sumps. In this regard, further investigation of the native soil types and stabilized groundwater levels should be carried out as part of the follow-up, site specific geotechnical investigation at the site. The need for and type of groundwater control measures can then be reviewed by the geotechnical engineer as part of the final design process.

- It is anticipated that the trench excavations for underground servicing would consist of conventional temporary open cuts with side slopes not steeper than 1 horizontal to 1 vertical. However, some local flattening of side slopes may be required in some areas in looser soil zones or where significant water seepage is encountered. Conventional bedding thicknesses are anticipated for underground services founded within the native competent subsoils at the site. Additional bedding thicknesses may be required for services founded in wet sandy/silty soils, depending upon the excavation depths and success of the contractor's groundwater control measures. It should be noted that cobbles and boulders may be encountered throughout the soils at the site.

- The majority of the subsoils above the local water table are generally near their estimated optimum water contents for compaction and should be suitable for reuse as trench backfill, provided they are free of significant amounts of topsoil, organics and other deleterious materials. Excavated subsoils from below the local water table (i.e. for deeper excavations, if required) would likely require some drying prior to placement.

- Complete removal of any existing septic systems, wells, old foundations, etc. would likely be required as part of the site redevelopment.

- Based on the subsoil conditions encountered, conventional asphaltic (flexible) pavement designs are considered to be appropriate for a proposed residential development at the site. Typical pavement designs are as follows:

MATERIAL		THICKNESS OF PAVEMENT ELEMENTS (mm)	
		Collector Residential Streets	**Local Residential Streets**
Asphaltic Material (OPSS 1150)	HL 3 Surface Course	40	40
	HL 8 Binder Course	65	50
Granular Material (OPSS 1010)	Granular A Base	150	150
	Granular B Type I Subbase	400	300
Prepared and Approved Subgrade			

Prior to placing the granular subbase material, the exposed soil subgrade should be heavily proof-rolled in conjunction with inspection by qualified geotechnical personnel. Deleterious, organic, softened or loosened native subsoils or any fills will require subexcavation and replacement with approved material (i.e. controlled fill), as directed by geotechnical personnel.

5. CLOSURE

The preliminary geotechnical recommendations provided in this report are not sufficient for final design or construction purposes. Once the actual residential development plans are available, the information in this report should be reviewed by the geotechnical engineer and an additional detailed geotechnical and hydrogeological investigation carried out, compatible with the actual proposed development plans for the site. In this regard, GeoPro would be pleased to provide further geotechnical and hydrogeological services if site development plans proceed forward.

We appreciate the opportunity to be of service to you and trust that this report provides sufficient preliminary geotechnical engineering information to facilitate the planning and preliminary concept design of this project. We look forward to providing you with continuing service during the detailed design stage. Please do not hesitate to contact our office should you wish to discuss, in further detail, any aspects of this project.

Yours very truly,

***GEOPRO* CONSULTING LIMITED**

Tim Yu

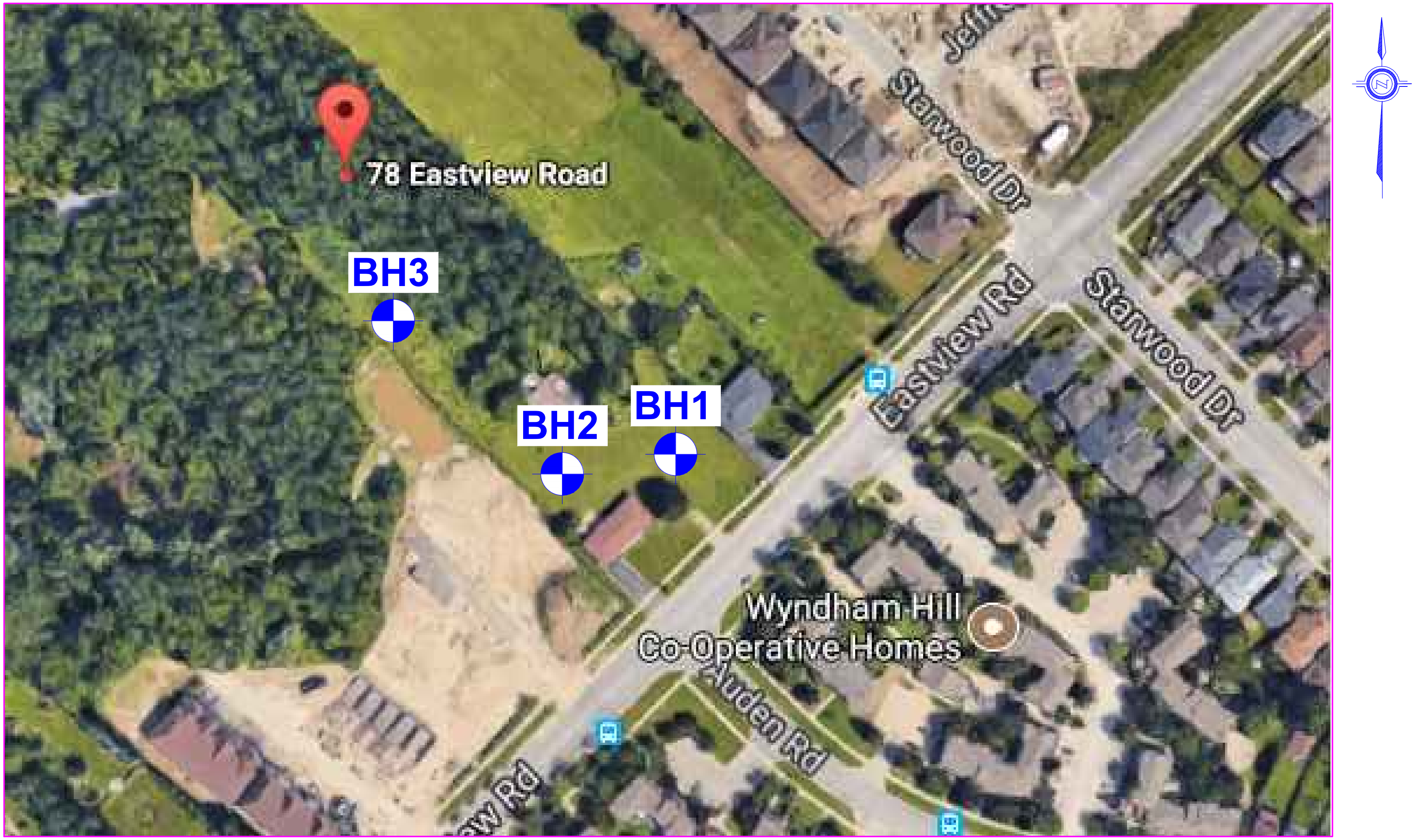
David B. Liu

LICENSED PROFESSIONAL ENGINEER
D. B. LIU
100107874
PROVINCE OF ONTARIO

Tim Yu, B.Eng., EIT
Geotechnical Group

David B. Liu, P.Eng. Principal

DRAWINGS



Legend:

⊕ Borehole Location

Client: **Best Homes Canada**

Project No.: **16-1696G2**

Drawing No.: **1**

Drawn: **TY**

Approved: **DL**

Title: **Borehole Location Plan**

Date: **September 2017**

Scale: **N.T.S.**

Project: **Preliminary Geotechnical Investigation for Due Diligence**
78 Eastview Road, Guelph, Ontario

Original Size: **Letter**

Rev: **JY**

GeoPro Consulting Limited

ENCLOSURES

GeoPro Consulting Limited

Geotechnical-Hydrogeology-Environmental-Materials-Inspection

Enclosure 1A: Notes on Sample Descriptions

1. Each soil stratum is described according to the *Modified Unified Soil Classification System*. The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined according to Canadian Foundation Engineering Manual, 4th Edition. Different soil classification systems may be used by others. Please note that a description of the soil stratums is based on visual and tactile examination of the samples augmented with field and laboratory test results, such as a grain size analysis and/or Atterberg Limits testing. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Enclosure 1B: Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
NR	No recovery
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

PM – Samples advanced by manual pressure
WR – Samples advanced by weight of sampler and rod
WH – Samples advanced by static weight of hammer

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60^o cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Piezo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60 degree conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurement of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

Textural Classification of Soils (ASTM D2487)

Classification	Particle Size
Boulders	> 300 mm
Cobbles	75 mm - 300 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm(*)

(*) Canadian Foundation Engineering Manual (4th Edition)

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Compactness Condition (Formerly Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

GeoPro CONSULTING LIMITED

LOG OF BOREHOLE BH1

1 OF 1

PROJECT: Geotechnical Investigation for Proposed Residential Development
CLIENT: Best Homes
PROJECT LOCATION: 78 Eastview Road, Guelph, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger- Auto Hammer
FIELD ENGINEER: SD
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 115 mm
DATE: 2017-08-21
REF. NO.: 16-1696G2
ENCL. NO.: 2

SOIL PROFILE			SAMPLES			GROUND WATER	ELEVATION	DYNAMIC PENETRATION TEST ○ SPT ⩘ Cone blows/0.3m 20 40 60 80 / SHEAR STRENGTH (kPa) ● Unconfined ✕ Field Vane & Sensitivity ▲ Quick Triaxial ⊠ Penetrometer + Lab Vane 20 40 60 80	Plastic Limit w_P Natural Moisture Content w Liquid Limit w_L / WATER CONTENT (%) 10 20 30 40	UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3m						
0.0	**TOPSOIL:** (250 mm)										
0.3	**FILL:** sandy silt, trace clay, trace gravel, trace organics, trace rootlets, containing cobbles and boulders, containing rock fragments, dark brown, moist, loose to compact		1 2A	SS SS	7 17			○ ○			
1.1	**SANDY SILT:** trace clay, trace gravel, containing cobbles and boulders, brown, moist to wet, compact		2B 3	SS SS	 14			○			
2.1	**SANDY SILT TILL:** trace clay, trace gravel, containing cobbles and boulders, layers of sandy silt, brown, moist, dense --- auger grinding		4	SS	32			○			
2.9	**SANDY SILT TO SAND AND SILT:** trace clay, trace gravel, containing cobbles and boulders, brown, moist, very dense		5A	SS	76			○			
3.3	**SANDY SILT TILL:** trace clay, trace gravel, containing cobbles and boulders, layers of sand, brown, moist, very dense		5B 6	SS SS	 71 / 230 mm			> >100 ○			
4.0	**END OF BOREHOLE DUE TO AUGER REFUSAL ON PROBABLE COBBLES AND BOULDERS** Notes: 1) Water was encountered at a depth of 2.4 m below ground surface (mBGS) during drilling. 2) Borehole was open and dry upon completion of drilling.										

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, ✕ 3: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

01 - GEOPRO SOIL LOG GEOPRO 16-1696BG BH LOG PROJECT DATA 20170913 -QC - TY.GPJ 2017-09-13 14:39

LOG OF BOREHOLE BH2

PROJECT: Geotechnical Investigation for Proposed Residential Development
CLIENT: Best Homes
PROJECT LOCATION: 78 Eastview Road, Guelph, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger- Auto Hammer
FIELD ENGINEER: SD
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 115 mm
DATE: 2017-08-21
REF. NO.: 16-1696G2
ENCL. NO.: 3

SOIL PROFILE		SAMPLES			GROUND WATER	ELEVATION	DYNAMIC PENETRATION TEST ○ SPT, Cone, blows/0.3m (20, 40, 60, 80) SHEAR STRENGTH (kPa) ● Unconfined ✕ Field Vane & Sensitivity ▲ Quick Triaxial ⊠ Penetrometer + Lab Vane (20, 40, 60, 80)	Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L WATER CONTENT (%) (10, 20, 30, 40)	UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3m						
0.0	**TOPSOIL:** (260 mm)									
0.3	**PROBABLE FILL:** silty sand, trace gravel, containing cobbles and boulders, dark brown to brown, moist, loose to compact	1	SS	8			○			
		2	SS	10			○			
1.8	--- auger grinding **SANDY SILT TO SAND AND SILT:** trace clay, trace gravel, containing cobbles and boulders, brown, moist, very dense	3	SS	50 / 150 mm			> >100 ○			
2.4	**END OF BOREHOLE DUE TO AUGER REFUSAL ON PROBABLE COBBLES AND BOULDERS** Note: 1) Borehole caved at a depth of 2.3 m below ground surface (mBGS) during drilling.									

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + ^{3}, × ^{3}: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

LOG OF BOREHOLE BH3

PROJECT: Geotechnical Investigation for Proposed Residential Development
CLIENT: Best Homes
PROJECT LOCATION: 78 Eastview Road, Guelph, Ontario
DATUM: Geodetic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger- Auto Hammer
FIELD ENGINEER: SD
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 115 mm
DATE: 2017-08-21
REF. NO.: 16-1696G2
ENCL. NO.: 4

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3m	GROUND WATER	ELEVATION	DYNAMIC PENETRATION TEST (SPT blows/0.3m)	WATER CONTENT (%)	UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** (180 mm)									
0.2	**PROBABLE FILL:** sandy silt to sand and silt, trace clay, trace gravel, trace organics, trace rootlets, containing cobbles and boulders, dark brown to brown, moist, loose to compact	1	SS	7			○			
		2	SS	10			○			
		3	SS	9			○			
2.3	**SANDY SILT TILL:** some gravel, trace to some clay, containing cobbles and boulders, brown, moist, very dense --- auger grinding	4	SS	73			○			
3.1	**END OF BOREHOLE DUE TO AUGER REFUSAL ON PROBABLE COBBLES AND BOULDERS** Note: 1) Borehole caved at a depth of 2.8 m below ground surface (mBGS) during drilling.									

DYNAMIC PENETRATION TEST: ○ SPT, Cone, blows/0.3m — 20 40 60 80
SHEAR STRENGTH (kPa): ● Unconfined, ✕ Field Vane & Sensitivity, ▲ Quick Triaxial, ⊠ Penetrometer, + Lab Vane — 20 40 60 80
Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L
WATER CONTENT (%): 10 20 30 40

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ▲ ε=3% Strain at Failure

01 - GEOPRO SOIL LOG GEOPRO 16-1696BG BH LOG PROJECT DATA 20170913 -QC - TY.GPJ 2017-09-13 14:39

LIMITATIONS TO THE REPORT

This report is intended solely for the Client named. The report is prepared based on the work has been undertaken in accordance with normally accepted geotechnical engineering practices in Ontario.

The comments and recommendations given in this report are based on information determined at the limited number of the test hole and test pit locations. The boundaries between the various strata as shown on the borehole logs are based on non-continuous sampling and represent an inferred transition between the various strata and their lateral continuation rather than a precise plane of geological change. Subsurface and groundwater conditions between and beyond the test holes and test pits may differ significantly from those encountered at the test hole and test pit locations. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole and test pit locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The report reflects our best judgment based on the information available to GeoPro Consulting Limited at the time of preparation. Unless otherwise agreed in writing by GeoPro Consulting Limited, it shall not be used to express or imply warranty as to any other purposes. No portion of this report shall be used as a separate entity, it is written to be read in its entirety. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated.

The design recommendations given in this report are applicable only to the project designed and constructed completely in accordance with the details stated in this report. Otherwise, our responsibility is limited to interpreting the subsurface information at the borehole or test pit locations.

Should any comments and recommendations provided in this report be made on any construction related issues, they are intended only for the guidance of the designers. The number of test holes and test pits may not be sufficient to determine all the factors that may affect construction activities, methods and costs. Such as, the thickness of surficial topsoil or fill layers may vary significantly and unpredictably; the amount of the cobbles and boulders may vary significantly than what described in the report; unexpected water bearing zones/layers with various thickness and extent may be encountered in the fill and native soils. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and make their own conclusions as to how the subsurface conditions may affect their work and determine the proper construction methods.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. GeoPro Consulting Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

GeoPro Consulting Limited

Geotechnical-Hydrogeology-Environmental-Materials-Inspection

Preliminary Geotechnical Investigation

Proposed Subdivision Development

82 Eastview Road, Guelph, Ontario

Prepared For:

Best Homes Canada

GeoPro Project No.: 16-1696-01

Report Date: December 19, 2016

Professional, Proficient, Proactive

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GeoPro
CONSULTING LIMITED

Table of Contents

1. INTRODUCTION

GeoPro Consulting Limited (GeoPro) was retained by Best Homes Canada (the Client) to undertake a preliminary geotechnical investigation to support the acquisition due diligence for the proposed subdivision development located at 82 Eastview Road, Guelph, Ontario.

The purpose of this preliminary geotechnical investigation was to obtain preliminary information on the existing subsurface conditions by means of a limited number of boreholes, in-situ tests and laboratory tests of soil samples to provide required preliminary geotechnical design information. Based on GeoPro's interpretation of the data obtained, preliminary geotechnical comments and recommendations related to the general planning and project concept design purposes are provided.

The report is prepared with the condition that the design will be in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice. Further, the recommendations and opinions in this report are applicable only to the proposed project as described above. On-going liaison and communication with GeoPro prior to and during the detailed design stage and construction phase of the project is strongly recommended to confirm that the preliminary recommendations in this report are applicable and/or correctly interpreted and implemented. Also, any queries concerning the geotechnical aspects of the proposed project shall be directed to GeoPro for further elaboration and/or clarification.

This report is provided on the basis of the terms of reference presented in our approved proposal prepared based on our understanding of the project. If there are any changes in the preliminary design and planning features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations can be relied upon.

This report deals with geotechnical issues only. The geo-environmental (chemical) aspects of the subsurface conditions, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, were not investigated and were beyond the scope of this assignment.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice in Ontario.

This report has been prepared for the Client. Third party use of this report without GeoPro's consent is prohibited. The limitation conditions presented in this report form an integral part of the report and they must be considered in conjunction with this report.

2. FIELD AND LABORATORY WORK

The field work for the geotechnical investigation was carried out on December 8, 2016, during which time four (4) boreholes (Boreholes BH1 to BH4) were advanced at the locations shown on the Borehole Location Plan, Drawing No. 1. The boreholes were drilled to depths ranging from 3.2 m to 7.8 m below the existing ground surface.

The boreholes were advanced using a continuous flight auger drilling equipment supplied by a drilling specialist subcontracted to GeoPro. Samples were retrieved with a 51 mm (2 inches) O.D. split-barrel (split spoon) sampler driven with a hammer weighing 624 N and dropping 760 mm (30 inches) in accordance with the Standard Penetration Test (SPT) method. The number of blows required to drive the sampler 300 mm (12 inches) depth into the undisturbed soil (SPT N values) gives an indication of the compactness condition or consistency of the sampled soil material.

Groundwater condition observations were made in the boreholes during drilling and upon completion of drilling. The boreholes were backfilled and sealed upon completion of drilling.

The field work for this investigation was monitored by a member of our engineering staff, who also determined the approximate borehole locations in the field, logged the boreholes and cared for the recovered samples. The boreholes were located and staked in the field by GeoPro according to the underground utility conditions and the accessibility of the drill rig.

All soil samples obtained during this investigation were brought to our laboratory for further examination and geotechnical classification testing (including water contents, grain size distributions and Atterberg limits, when applicable) on selected soil samples. The results of grain size analyses of the selected soil samples are shown on Figure 1.

The ground surface elevations of the as-drilled borehole locations were not available at the time of preparing the report. Therefore, the stratigraphy of each borehole location was referenced to the current grade level. The borehole locations plotted on the Borehole Location Plan, Drawing No. 1, were based on the measurement of site features and should be considered to be approximate. Contractors performing the work should confirm the elevations prior to construction.

3. SITE AND SUBSURFACE CONDITIONS

The borehole locations are shown on Drawing No. 1. Notes on sample descriptions are presented in Enclosure No. 1A. An explanation of terms used in the borehole logs is presented in Enclosure No. 1B. The subsurface conditions in the boreholes (Boreholes BH1 to BH4) are presented in the individual borehole logs (Enclosure Nos. 2 to 5 inclusive). The detailed descriptions of the major soil strata encountered in the boreholes drilled at the site are provided as follows.

3.1 Soil Conditions

Topsoil

Topsoil with thickness ranging from 460 mm to 610 mm was encountered surficially in all boreholes.

Fill Materials

Fill materials consisting of organic silt were encountered below the topsoil in Boreholes BH3 and BH4, and extended to depths ranging from about 0.7 m to 1.8 m below the existing ground surface. It is noted that fill materials were observed in composition of organic silt and topsoil in Borehole BH4. SPT N values ranging from 4 to 23 blows per 300 mm penetration indicated a very loose to compact relative density. The in-situ moisture contents measured in the fill samples ranged from approximately 16% to 32%.

Reworked Sandy Silt

Reworked sandy silt deposit was encountered below the topsoil in Boreholes BH1 and BH2, and extended to a depth of about 0.7 m below the existing ground surface. SPT N value of 8 blows per 300 mm penetration indicated a loose relative density. The natural moisture content measured in the soil sample was approximately 14%.

Sandy Silt

Sandy silt deposit was encountered below the silty sand in Borehole BH2, and extended to a depth of about 2.9 m below the existing ground surface. SPT N value of 71 blows per 300 mm penetration indicated a very dense relative density. The natural moisture content measured in the soil sample was approximately 13%.

Upper Layer Sand and Gravel

Upper layer sand and gravel deposit was encountered below the silty sand in Borehole BH3, and extended to a depth of about 2.9 m below the existing ground surface. SPT N values ranging from 32 to 63 blows per 300 mm penetration indicated a dense to very dense relative density. The natural moisture contents measured in the soil samples ranged from approximately 6% to 7%.

Fine Sand and Silt to Fine Sandy Silt

Fine sand and silt to fine sandy silt deposits were encountered below the sandy silt in Borehole BH2, and extended to a depth of about 3.2 m below the existing ground surface. Borehole BH2 was terminated in these deposits. SPT N value of greater than 100 blows per 300 mm penetration indicated a very dense relative density. The natural moisture content measured in the soil sample was approximately 9%.

Silty Sand

Silty sand deposit was encountered below the sandy silt till and fill materials in Boreholes BH2 to BH4, and extended to depths ranging from about 1.4 m to 3.4 m below the existing ground surface. SPT N values ranging from 15 to 35 blows per 300 mm penetration indicated a compact to dense relative density. The natural moisture contents measured in the soil samples ranged from approximately 5% to 32%.

Silty Sand Till

Silty sand till deposit was encountered below the upper layer sand and gravel in Borehole BH3, and extended to a depth of about 4.0 m below the existing ground surface. SPT N value of 43 blows per 300 mm penetration indicated a dense relative density. The natural moisture content measured in the soil sample was approximately 7%.

Clayey Silt

Clayey silt deposit was encountered below the silty sand in Borehole BH4, and extended to a depth of about 4.0 m below the existing ground surface. SPT N value of 27 blows per 300 mm penetration indicated a very stiff consistency. The natural moisture content measured in the soil sample was approximately 11%.

Sand and Silt

Sand and silt deposit was encountered below the reworked sandy silt in Borehole BH1, and extended to a depth of about 4.7 m below the existing ground surface. Borehole BH1 was terminated in this deposit. SPT N values ranging from 12 to greater than 100 blows per 300 mm penetration indicated a compact to very dense relative density. The natural moisture contents measured in the soil samples ranged from approximately 8% to 11%.

Sandy Gravel

Sandy gravel deposit was encountered below the clayey silt in Borehole BH4, and extended to a depth of about 5.5 m below the existing ground surface. Borehole BH4 was terminated in this deposit. SPT N value of 27 blows per 300 mm penetration indicated a compact relative density. The natural moisture content measured in the soil sample was approximately 8%.

Lower Layer Sand and Gravel

Lower layer sand and gravel deposit was encountered below the silty sand till in Borehole BH3, and extended to a depth of about 5.6 m below the existing ground surface. SPT N value of 67 blows per 300 mm penetration indicated a very dense relative density. The natural moisture content measured in the soil sample was approximately 8%.

Sandy Silt Till

Sandy silt till deposit was encountered below the reworked sandy silt and lower layer sand and gravel in Boreholes BH2 and BH3, and extended to depths ranging from about 1.4 m to 7.1 m below the existing ground surface. SPT N values ranging from 30 to greater than 100 blows per 300 mm penetration indicated a compact to very dense relative density. The natural moisture contents measured in the soil samples ranged from approximately 6% to 7%.

Silt

Silt deposit was encountered below the sandy silt till in Borehole BH3, and extended to a depth of about 7.8 m below the existing ground surface. Borehole BH3 was terminated in this deposit. SPT N value of greater than 100 blows per 300 mm penetration indicated a very dense relative density. The natural moisture content measured in the soil sample was approximately 16%.

3.2 Groundwater Conditions

The groundwater conditions were observed in the boreholes during and immediately upon completion of drilling. Summary of the observations are shown in the borehole logs and in the following table:

Borehole No.	Borehole Depth (m)	Depth of Water Encountered during Drilling (mBGS)	Water Level upon Completion of Drilling (mBGS)	Cave-in Depth upon Completion of Drilling (mBGS)
BH1	4.7	2.1	3.4	Open
BH2	3.2	-	Dry	Open
BH3	7.8	3.0	3.4	2.4
BH4	5.5	4.0	4.0	Open

Note: mBGS = meter below ground surface

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to weather events.

4. PRELIMINARY GEOTECHNICAL INFORMATION

This report contains the findings of GeoPro's preliminary geotechnical investigation, together with the preliminary geotechnical engineering recommendations and comments. These preliminary recommendations and comments are based on factual information and are intended only for use by the design engineers. The number of boreholes are not sufficient for the detailed design and are not sufficient to determine all the factors that may affect construction methods and costs. Once the actual development plans are available, the information in this report should be reviewed by the geotechnical engineer from GeoPro and an additional detailed geotechnical investigation shall be carried out, compatible with the actual proposed development plans for the site.

It is understood that the proposed development consists of residential subdivision. Based on the results of this preliminary geotechnical investigation, the native soils encountered at the site are generally considered to be suitable for supporting the proposed development. The following preliminary geotechnical information is provided for the planning and preliminary design of subdivision development at the site.

- The existing fill materials and fill/organic silt mixture were encountered below the topsoil and extended to depths ranging from 0.7 m to 1.8 m below the existing ground surface at the site. Topsoil and/or reworked sandy silt deposit were encountered in Boreholes BH1 and BH2; whereas topsoil and/or fill materials consisting of organic silt were encountered in Boreholes BH3 and BH4. The topsoil/reworked soils and existing fill materials are considered to be not suitable for supporting the residential houses (with or without basement) and any other settlement sensitive structures. The topsoil/reworked soils and existing fill materials are also considered to be not suitable for supporting the engineered fill.

- Depending upon the final site grading scheme and proposed basement elevations, the areas should be brought up to the underside of the footings, as required, using engineered fill. The materials proposed for use as engineered fill should be approved by qualified geotechnical personnel from GeoPro at the source, prior to hauling to the site. Most of the native soils at the site would be suitable for reuse as engineered fill. Imported materials meeting the requirements of OPSS Select Subgrade Material (SSM) would also be suitable for use as engineered fill. Details regarding placement and compaction requirements for engineered fill, if utilized at the site, can be provided once the actual development plans are available, as part of the final geotechnical recommendations for the project.

- The native subsoils at the site are considered to be suitable for supporting conventional residential houses (with or without basements). A preliminary allowable bearing pressure of 150 kPa may be assumed for conventional shallow spread and/or strip footings bearing in the native, undisturbed subsoils, at depths approximately ranging from 0.7 m to 1.8 m below the existing ground surface (or deeper as required for basements). Footings founded on approved engineered fill, if utilized at the site, may be designed using a preliminary allowable bearing pressure of 150 kPa.

- The type of foundation drainage system required (perimeter drains and/or under slab drains; damp-proofing or water-proofing) depends upon the proposed founding elevations, soil types in the area and actual stabilized groundwater levels. Based on the results of this preliminary investigation, it is anticipated that conventional foundation drainage (i.e. perimeter drains and damp-proofing) would be adequate at most locations. Basements founded in sandy/silty soils below the groundwater level (if any) may require water-proofing and/or under slab drains. In any event, the type of foundation drainage should be confirmed by the geotechnical engineer once the site grading plans are available, as part of final design process.

- All exterior footings and footings in unheated areas should be protected with a minimum of 1.4 m of earth cover for frost protection.

- Based on the results of this preliminary investigation, for shallow excavations above the prevailing groundwater tables, groundwater control during excavation can probably be handled, if required, by pumping from properly constructed and filtered sumps located at the base of the excavation. However, more significant groundwater seepage will be expected from the sandy/silty deposits below the prevailing groundwater tables. Some form of positive groundwater control will be required to maintain the stability of the base and side slopes of the excavations in these areas, in addition to pumping from sumps. In this regard, further investigation of the native soil types and stabilized groundwater levels should be carried out as part of the follow-up, site specific geotechnical investigation at the site. The need for and type of groundwater control measures can then be reviewed by the geotechnical engineer as part of the final design process.

- It is anticipated that the trench excavations for underground servicing would consist of conventional temporary open cuts with side slopes not steeper than 1 horizontal to 1 vertical. However, some local flattening of side slopes may be required in some areas in looser soil zones or where significant water seepage is encountered. Conventional bedding thicknesses are anticipated for underground services founded within the native competent subsoils at the site. Additional bedding thicknesses may be required for services founded in wet sandy/silty soils, depending upon the excavation depths and success of the contractor's groundwater control measures. It should be noted that cobbles and boulders may be encountered throughout the soils at the site.

- The majority of the subsoils above the local water table are generally near their estimated optimum water contents for compaction and should be suitable for reuse as trench backfill, provided they are free of significant amounts of topsoil, organics and other deleterious materials. Excavated subsoils from below the local water table (i.e. for deeper excavations, if required) would likely require some drying prior to placement.

- Complete removal of any existing septic systems, wells, old foundations, etc. would likely be required as part of the site redevelopment.

- Based on the subsoil conditions encountered, conventional asphaltic (flexible) pavement designs are considered to be appropriate for a proposed residential development at the site. Typical pavement designs are as follows:

MATERIAL		**THICKNESS OF PAVEMENT ELEMENTS (mm)**	
		Collector Residential Streets	**Local Residential Streets**
Asphaltic Material (OPSS 1150)	HL 3 Surface Course	40	40
	HL 8 Binder Course	65	50
Granular Material (OPSS 1010)	Granular A Base	150	150
	Granular B Type I Subbase	400	300
Prepared and Approved Subgrade			

Prior to placing the granular subbase material, the exposed soil subgrade should be heavily proof-rolled in conjunction with inspection by qualified geotechnical personnel. Deleterious, organic, softened or loosened native subsoils or any fills will require subexcavation and replacement with approved material (i.e. controlled fill), as directed by geotechnical personnel.

5. CLOSURE

The preliminary geotechnical recommendations provided in this report are not sufficient for final design or construction purposes. Once the actual residential development plans are available, the information in this report should be reviewed by the geotechnical engineer and an additional detailed geotechnical and hydrogeological investigation carried out, compatible with the actual proposed development plans for the site. In this regard, GeoPro would be pleased to provide further geotechnical and hydrogeological services if site development plans proceed forward.

We appreciate the opportunity to be of service to you and trust that this report provides sufficient preliminary geotechnical engineering information to facilitate the planning and preliminary concept design of this project. We look forward to providing you with continuing service during the detailed design stage. Please do not hesitate to contact our office should you wish to discuss, in further detail, any aspects of this project.

Yours very truly,

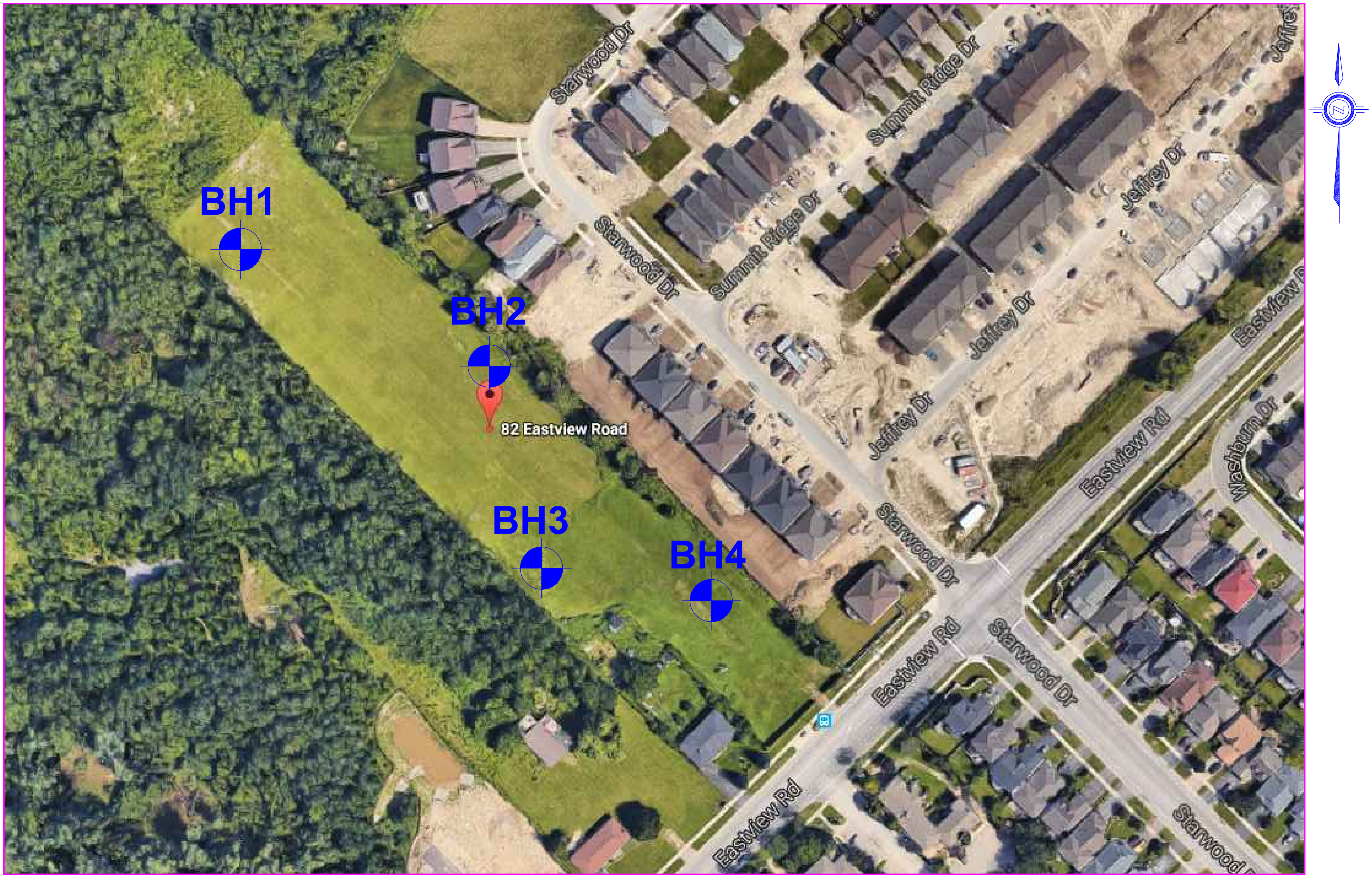
***GEOPRO* CONSULTING LIMITED**

Tim Yu, B.Eng., EIT
Geotechnical Group

LICENSED PROFESSIONAL ENGINEER
D. B. LIU
100107874
PROVINCE OF ONTARIO

David B. Liu, P.Eng. Principal

DRAWINGS



Legend:

⊕ Borehole Location

Client:	**Best Homes Canada**	Project No.: **16-1696**	Drawing No.: **1**
Drawn: **GH**	Approved: **DL**	Title:	**Borehole Location Plan**
Date: **December 2016**	Scale: **N.T.S.**	Project:	**Preliminary Geotechnical Investigation for Due Diligence** **82 Eastview Road, Guelph, Ontario**
Original Size: **Letter**	Rev: **JY**		***GeoPro* Consulting Limited**

ENCLOSURES

Enclosure 1A: Notes on Sample Descriptions

1. Each soil stratum is described according to the *Modified Unified Soil Classification System*. The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined according to Canadian Foundation Engineering Manual, 4th Edition. Different soil classification systems may be used by others. Please note that a description of the soil stratums is based on visual and tactile examination of the samples augmented with field and laboratory test results, such as a grain size analysis and/or Atterberg Limits testing. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

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Enclosure 1B: Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
NR	No recovery
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

PM – Samples advanced by manual pressure
WR – Samples advanced by weight of sampler and rod
WH – Samples advanced by static weight of hammer

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60^{o} cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Piezo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60 degree conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurement of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

Textural Classification of Soils (ASTM D2487)

Classification	Particle Size
Boulders	> 300 mm
Cobbles	75 mm - 300 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm(*)

(*) Canadian Foundation Engineering Manual (4th Edition)

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency Value	Undrained Shear Strength (kPa)	SPT "N"
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

LOG OF BOREHOLE BH1

1 OF 1

PROJECT: Due Diligence Preliminary Geotechnical Investigation
CLIENT: Best Homes Canada
PROJECT LOCATION: 82 Eastview Road, Guelph, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

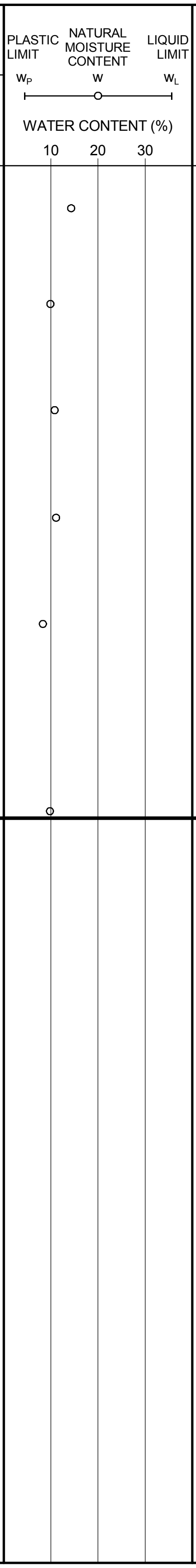
DRILLING DATA
Method: Continuous Flight Auger - Auto Hammer
Diameter: 155/205mm
Date: Dec/08/2016

REF. NO.: 16-1696
ENCL NO.: 2

SOIL PROFILE			SAMPLES								
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** (460 mm)		1	SS	8					
0.5	**REWORKED SANDY SILT:** trace gravel, trace rootlets, brown, moist, loose									
0.7	**SAND AND SILT:** trace clay, trace to some gravel, trace rootlets, brown, moist to wet, compact to very dense		2	SS	32					
			3	SS	12					
			4	SS	38					
			5	SS	93					
			6	SS	50/100 mm					
4.7	**END OF BOREHOLE** Notes: 1) Water encountered at a depth of 2.1 m below ground surface (mBGS) during drilling. 2) Water was at a depth of 3.4 mBGS upon completion of drilling. 3) Borehole was open upon completion of drilling.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100



GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH2

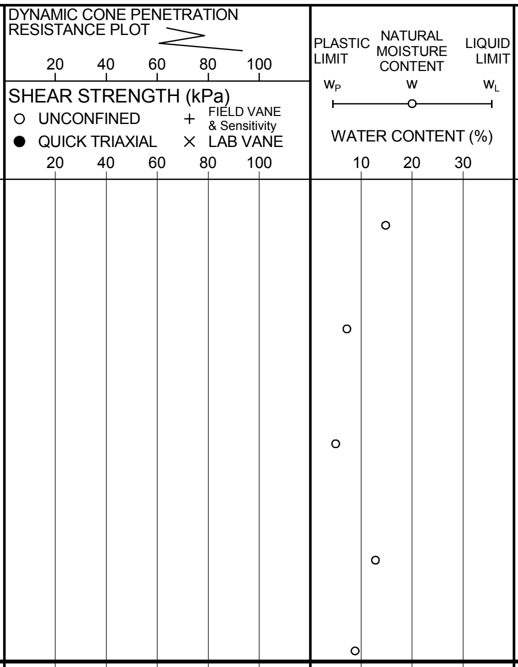
1 OF 1

PROJECT: Due Diligence Preliminary Geotechnical Investigation
CLIENT: Best Homes Canada
PROJECT LOCATION: 82 Eastview Road, Guelph, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Continuous Flight Auger - Auto Hammer
Diameter: 155/205mm
Date: Dec/08/2016

REF. NO.: 16-1696
ENCL NO.: 3

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m					
0.0	**TOPSOIL:** (610 mm)		1	SS	3					
0.6	**REWORKED SANDY SILT:** trace gravel, trace rootlets, brown, moist									
0.7	**SANDY SILT TILL:** trace to some clay, trace gravel, brown, moist, compact		2	SS	30					
1.4	**SILTY SAND:** some gravel, containing cobbles and boulders, brown, moist, dense		3	SS	35					
2.1	**SANDY SILT:** trace gravel, containing cobbles and boulders, brown, moist to wet, very dense		4	SS	71					
2.9	**FINE SAND AND SILT TO FINE SANDY SILT:** trace gravel, containing cobbles and boulders, brown, moist, very dense		5	SS	50/140 mm					
3.2	**END OF BOREHOLE DUE TO AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDERS** Notes: 1) Borehole was open and dry upon completion of drilling.									



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

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LOG OF BOREHOLE BH3

1 OF 1

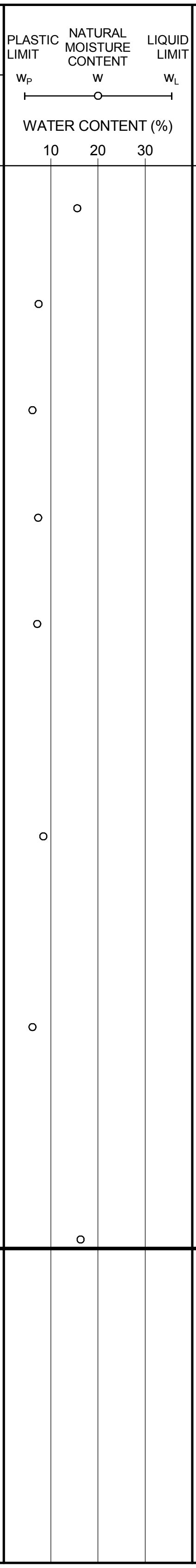
PROJECT: Due Diligence Preliminary Geotechnical Investigation
CLIENT: Best Homes Canada
PROJECT LOCATION: 82 Eastview Road, Guelph, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Continuous Flight Auger - Auto Hammer
Diameter: 155/205mm
Date: Dec/08/2016

REF. NO.: 16-1696
ENCL NO.: 4

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** (460 mm)	1	SS	4						
0.5	**FILL:** organic silt, trace rootlets, brown, moist, very loose									
0.7	**SILTY SAND:** trace gravel, containing zones of sandy silt with some clay, brown, moist, compact	2	SS	15						
1.4	**SAND AND GRAVEL:** trace to some silt, brown, moist to wet, very dense to dense	3 4	SS SS	63 32						
2.9	**SILTY SAND TILL:** some clay, some gravel, brown, moist to wet, dense	5	SS	43						
4.0	**SAND AND GRAVEL:** some silt, containing cobbles and boulders, brown, wet, very dense	6	SS	67						
5.6	**SANDY SILT TILL:** trace gravel, containing cobbles and boulders, grey, moist, very dense	7	SS	50/130 mm						
7.1	**SILT:** trace sand, some clay, grey, moist to wet, very dense	8	SS	50/130 mm						
7.8	**END OF BOREHOLE** Notes: 1) Water encountered at a depth of 3.0 m below ground surface (mBGS) during drilling. 2) Water was at a depth of 3.4 mBGS upon completion of drilling. 3) Borehole was caved in at a depth of 2.4 mBGS upon completion of drilling.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED; + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL; × LAB VANE; 20 40 60 80 100



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH4

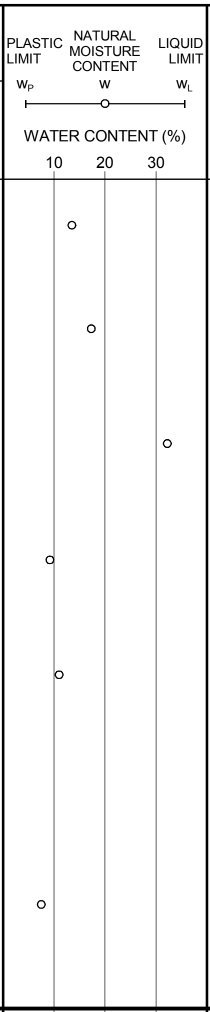
1 OF 1

PROJECT: Due Diligence Preliminary Geotechnical Investigation
CLIENT: Best Homes Canada
PROJECT LOCATION: 82 Eastview Road, Guelph, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Continuous Flight Auger - Auto Hammer
Diameter: 155/205mm
Date: Dec/08/2016
REF. NO.: 16-1696
ENCL NO.: 5

SOIL PROFILE		SAMPLES		
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m
0.0	**TOPSOIL:** (610 mm)	1	SS	22
0.6	**FILL:** organic silt, mixed with topsoil, trace sand, trace to some clay, dark brown, moist, compact	2	SS	11
		3	SS	23
1.8	**SILTY SAND:** some gravel, brown, moist to wet, compact	4	SS	23
		5	SS	27
3.4	**CLAYEY SILT:** sandy, trace gravel, brown, moist, very stiff			
4.0	**SANDY GRAVEL:** some silt, containing cobbles and boulders, brown, wet, compact	6	SS	27
5.5	**END OF BOREHOLE DUE TO AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDERS** Notes: 1) Water encountered at a depth of 4.0 m below ground surface (mBGS) during drilling. 2) Water was at a depth of 4.0 mBGS upon completion of drilling. 3) Borehole was open upon completion of drilling.			

GROUND WATER CONDITIONS | ELEVATION | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m³) | REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

FIGURES

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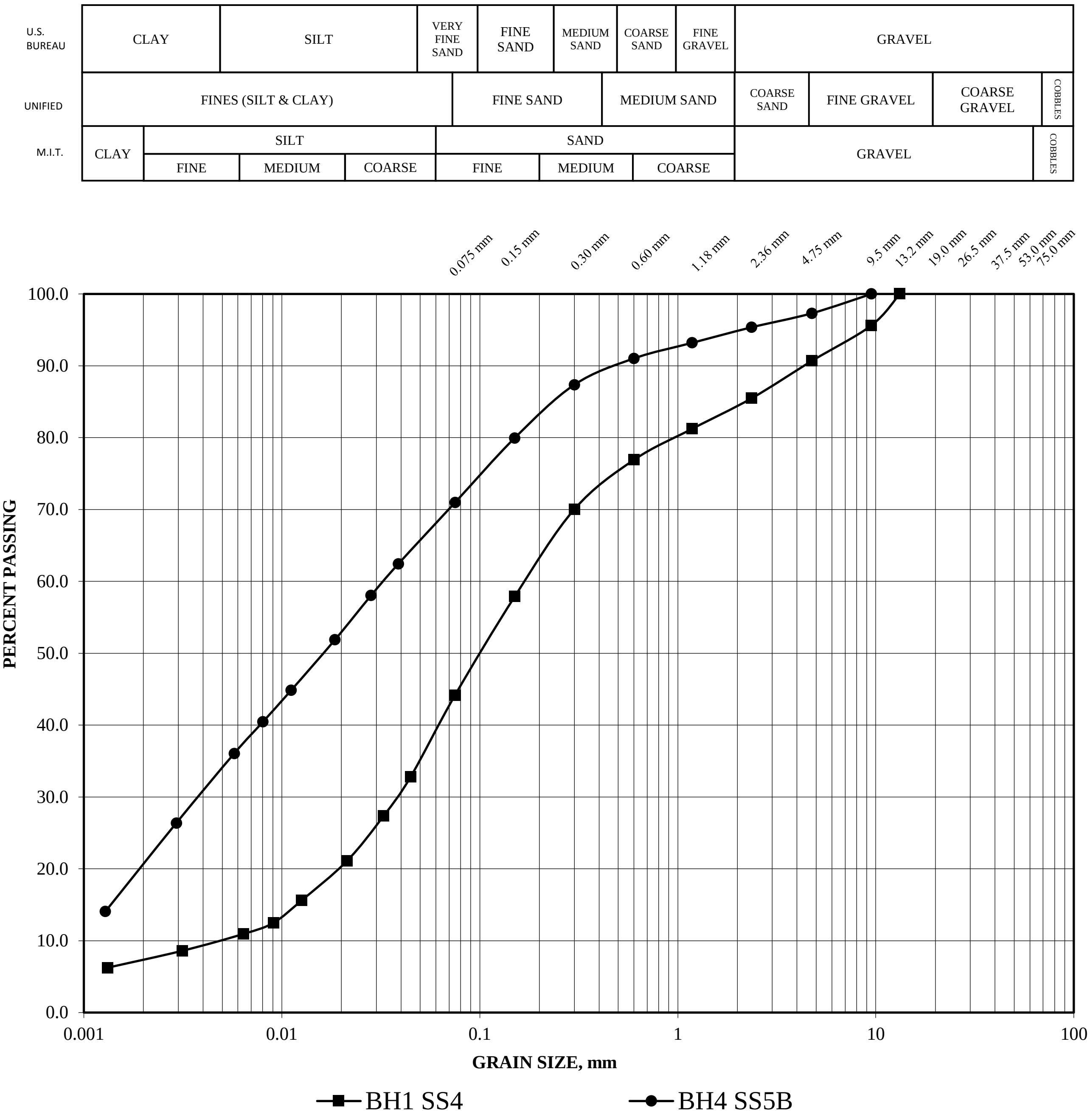
Figure 1

GRAIN SIZE DISTRIBUTION



Project No.	16-1696
Project Name	82 Eastview Road, Guelph, Ontario

LIMITATIONS TO THE REPORT

This report is intended solely for the Client named. The report is prepared based on the work has been undertaken in accordance with normally accepted geotechnical engineering practices in Ontario.

The comments and recommendations given in this report are based on information determined at the limited number of the test hole and test pit locations. The boundaries between the various strata as shown on the borehole logs are based on non-continuous sampling and represent an inferred transition between the various strata and their lateral continuation rather than a precise plane of geological change. Subsurface and groundwater conditions between and beyond the test holes and test pits may differ significantly from those encountered at the test hole and test pit locations. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole and test pit locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The report reflects our best judgment based on the information available to GeoPro Consulting Limited at the time of preparation. Unless otherwise agreed in writing by GeoPro Consulting Limited, it shall not be used to express or imply warranty as to any other purposes. No portion of this report shall be used as a separate entity, it is written to be read in its entirety. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated.

The design recommendations given in this report are applicable only to the project designed and constructed completely in accordance with the details stated in this report.

Should any comments and recommendations provided in this report be made on any construction related issues, they are intended only for the guidance of the designers. The number of test holes and test pits may not be sufficient to determine all the factors that may affect construction activities, methods and costs. Such as, the thickness of surficial topsoil or fill layers may vary significantly and unpredictably; the amount of the cobbles and boulders may vary significantly than what described in the report; unexpected water bearing zones/layers with various thickness and extent may be encountered in the fill and native soils. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and make their own conclusions as to how the subsurface conditions may affect their work and determine the proper construction methods.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. GeoPro Consulting Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

APPENDIX

C

GUELPH PERMEAMETER CALCULATIONS

Appendix C: Guelph Permeameter Calculations

78 - 82 Eastview Road Guelph

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi(H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi(H_2/a^*)}$$

Two Head Method:

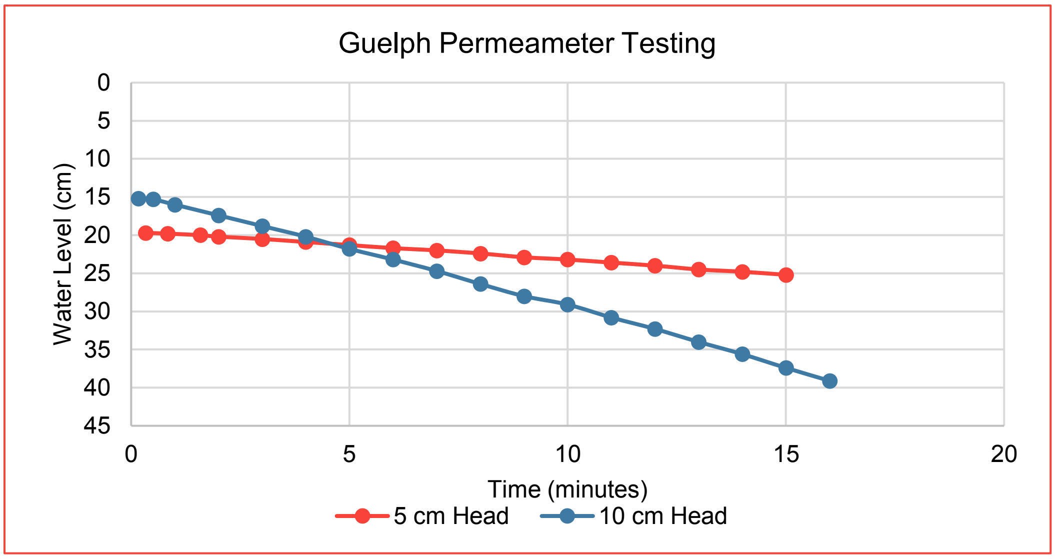
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\big(2H_1 H_2 (H_2 - H_1) + a^2 (H_1 C_2 - H_2 C_1)\big)$$

Borehole ID: GP18-1S
Depth (cm): 20



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1515.11	cm^3		calculated
R_1	0.006	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.028	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	1.71E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	5.76E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$	3.16E-03	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	1.30E-05	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi(H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi(H_2/a^*)}$$

Two Head Method:

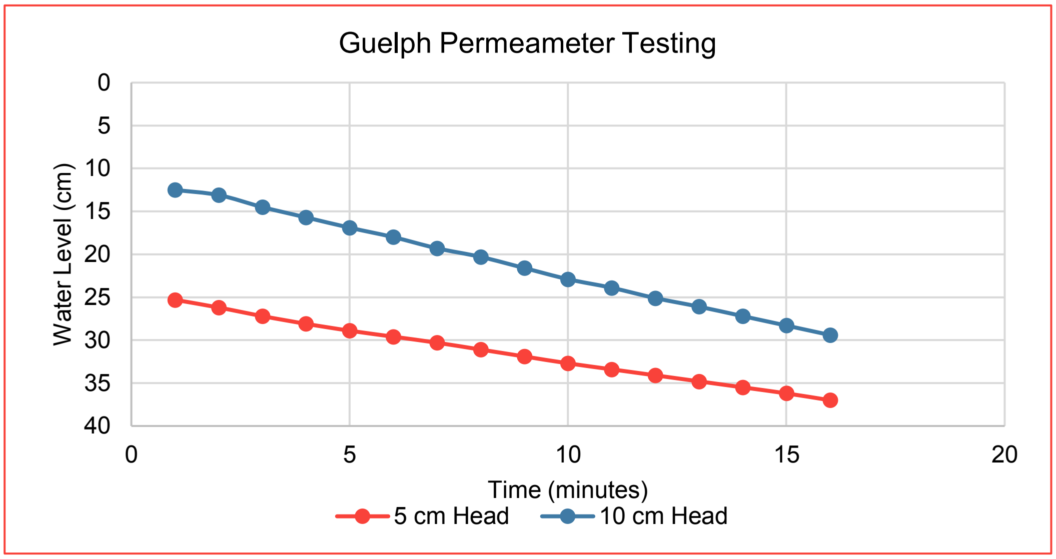
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)$$

Borehole ID: GP18-1D
Depth (cm): 55



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1515.11	cm^3		calculated
R_1	0.012	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.018	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	3.75E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	3.73E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$	3.57E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	3.68E-06	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

Appendix C: Guelph Permeameter Calculations

78 - 82 Eastview Road Guelph

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi (H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi (H_2/a^*)}$$

Two Head Method:

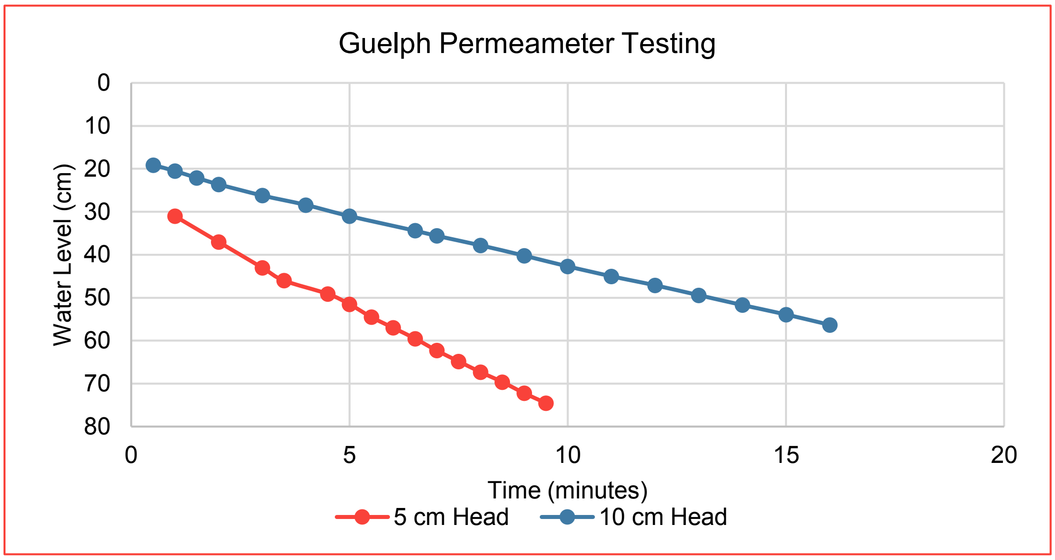
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)$$

Borehole ID: GP18-2S
Depth (cm): 25



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1515.11	cm^3		calculated
R_1	0.080	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.038	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	2.46E-03	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	7.79E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$		cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	1.62E-05	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

Appendix C: Guelph Permeameter Calculations
78 - 82 Eastview Road Guelph

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi (H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi (H_2/a^*)}$$

Two Head Method:

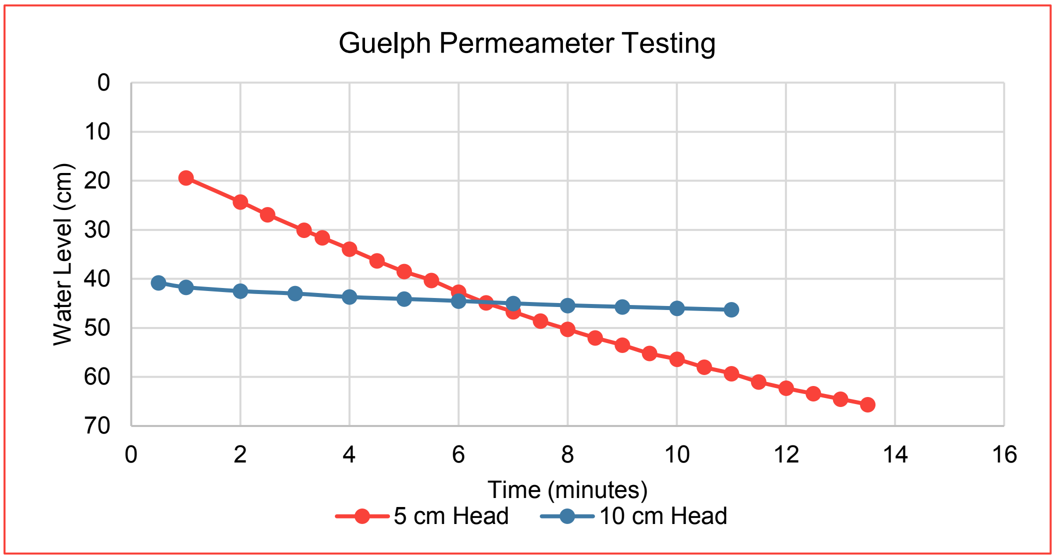
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)$$

Borehole ID: GP18-2D
Depth (cm): 65



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1515.11	cm^3		calculated
R_1	0.037	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.005	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	1.13E-03	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	1.02E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$		cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	6.14E-06	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

Appendix C: Guelph Permeameter Calculations

78 - 82 Eastview Road Guelph

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi (H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi (H_2/a^*)}$$

Two Head Method:

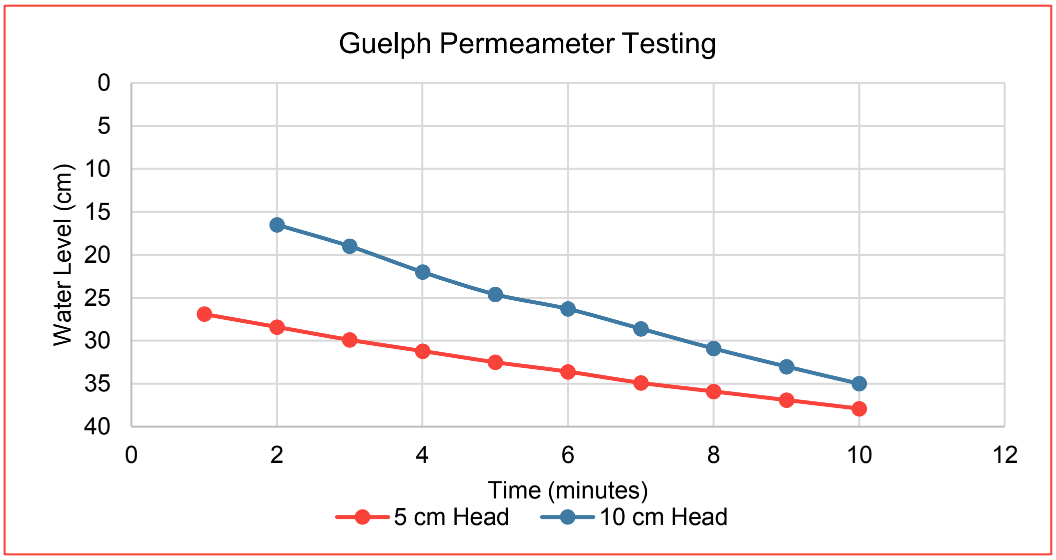
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)$$

Borehole ID: GP18-3S
Depth (cm): 20



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1514.73	cm^3		calculated
R_1	0.017	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.036	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	5.12E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	7.21E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$	2.06E-03	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	1.10E-05	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

Unsaturated Hydraulic Conductivity Calculations (Soil Moisture, 2012)

One Head Method:

$$K_{fs5} = \frac{C_1 R_1 X}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi (H_1/a^*)} \qquad K_{fs10} = \frac{C_2 R_2 X}{2\pi H_2^2 + \pi a^2 C_2 + 2\pi (H_2/a^*)}$$

Two Head Method:

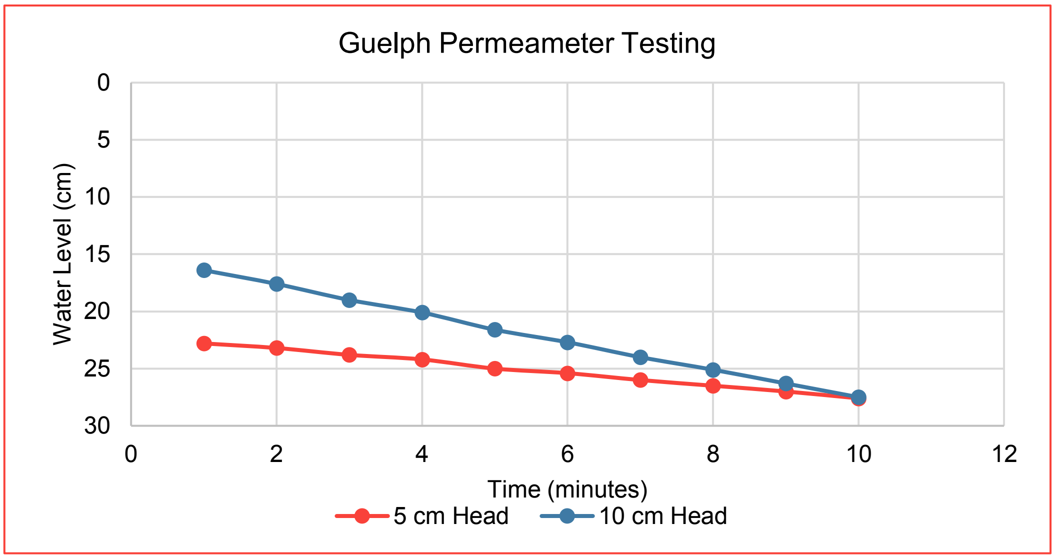
$$K_{fs5/10} = \frac{X(H_1 C_2 R_2 - H_2 C_1 R_1)}{C_3}$$

Additional Calculations

$$C_1 = \left(\frac{(H_1/a)}{1.992 + 0.091(H_1/a)}\right)^{0.683} \qquad C_2 = \left(\frac{(H_2/a)}{1.992 + 0.091(H_2/a)}\right)^{0.683}$$

$$C_3 = \pi\big(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\big)$$

Borehole ID: GP18-3D
Depth (cm): 70



Parameter	Value	Unit	Definition	Notes
C_1	0.84		shape factor	calculated
C_2	1.29		shape factor	calculated
C_3	1515.11	cm^3		calculated
R_1	0.009	cm/s	steady state rate of fall of water (cm/s)	From Test
R_2	0.019	cm/s	steady state rate of fall of water (cm/s)	From Test
X	35.22		combined reservoir constant	From Guelph Permeameter
H_1	5	cm	first head (cm)	From Test
H_2	10	cm	second head (cm)	From Test
a	3	cm	borehole radius (cm)	From Test
a*	0.04	cm^{-1}	macroscopic capillary length	Approximated based on Soil Type (Soil Moisture, 2012)
K_{fs5}	2.73E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
K_{fs10}	3.95E-04	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
$K_{fs5/10}$	1.18E-03	cm/s	soil unsaturated hydraulic conductivity (cm/s)	calculated
Average K_{fs}	6.15E-06	m/s	soil unsaturated hydraulic conductivity (cm/s)	calculated

References:

Soil Moisture, 2012. Guelph Permeameter Operating Instructions.

APPENDIX

D

WATER BALANCE SUPPORTING DOCUMENTS

Table 3.1: Hydrologic Cycle Component Values

	Water Holding Capacity mm	Hydrologic Soil Group	Precipitation mm	Evapo-transpiration mm	Runoff mm	Infiltration* mm
Urban Lawns/Shallow Rooted Crops (spinach, beans, beets, carrots)						
Fine Sand	50	A	940	515	149	276
Fine Sandy Loam	75	B	940	525	187	228
Silt Loam	125	C	940	536	222	182
Clay Loam	100	CD	940	531	245	164
Clay	75	D	940	525	270	145
Moderately Rooted Crops (corn and cereal grains)						
Fine Sand	75	A	940	525	125	291
Fine Sandy Loam	150	B	940	539	160	241
Silt Loam	200	C	940	543	199	199
Clay Loam	200	CD	940	543	218	179
Clay	150	D	940	539	241	160
Pasture and Shrubs						
Fine Sand	100	A	940	531	102	307
Fine Sandy Loam	150	B	940	539	140	261
Silt Loam	250	C	940	546	177	217
Clay Loam	250	CD	940	546	197	197
Clay	200	D	940	543	218	179
Mature Forests						
Fine Sand	250	A	940	546	79	315
Fine Sandy Loam	300	B	940	548	118	274
Silt Loam	400	C	940	550	156	234
Clay Loam	400	CD	940	550	176	215
Clay	350	D	940	549	196	196

Notes: Hydrologic Soil Group A represents soils with low runoff potential and Soil Group D represents soils with high runoff potential. The evapotranspiration values are for mature vegetation. Streamflow is composed of baseflow and runoff.

* *This is the total infiltration of which some discharges back to the stream as base flow. The infiltration factor is determined by summing a factor for topography, soils and cover.*

<u>Topography</u>	Flat Land, average slope < 0.6 m/km	0.3
	Rolling Land, average slope 2.8 m to 3.8 m/km	0.2
	Hilly Land, average slope 28 m to 47 m/km	0.1
<u>Soils</u>	Tight impervious clay	0.1
	Medium combinations of clay and loam	0.2
	Open Sandy loam	0.4
<u>Cover</u>	Cultivated Land	0.1
	Woodland	0.2

APPENDIX

E

LID FACT SHEETS

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

SOAKAWAYS, INFILTRATION TRENCHES AND CHAMBERS

GENERAL DESCRIPTION

Soakaways are rectangular or circular excavations lined with geotextile fabric and filled with clean granular stone or other void forming material that receive runoff from a perforated pipe inlet and allow it to infiltrate into the native soil. They typically service individual lots and receive only roof and walkway runoff but can also be designed to receive overflows from rainwater harvesting systems. Soakaways can also be referred to as infiltration galleries, dry wells or soakaway pits.

Infiltration trenches are rectangular trenches lined with geotextile fabric and filled with clean granular stone or other void forming material. Like soakaways, they typically service an individual lot and receive only roof and walkway runoff. This design variation on soakaways is well suited to sites where available space for infiltration is limited to narrow strips of land between buildings or properties, or along road rights-of-way. They can also be referred to as infiltration galleries or linear soakaways.

Infiltration chambers are another design variation on soakaways. They include a range of proprietary manufactured modular structures installed underground, typically under parking or landscaped areas that create large void spaces for temporary storage of stormwater, allowing it to infiltrate into the underlying native soil. Structures typically have open bottoms, perforated side walls and optional underlying granular stone reservoirs. They can be installed individually or in series in trench or bed configurations. They can infiltrate roof, walkway, parking lot and road runoff with adequate pretreatment. Due to the large volume of underground void space they create in comparison to a soakaway of the same dimensions, and the modular nature of their design, they are well suited to sites where available space for other types of BMPs is limited, or where it is desirable for the facility to have little or no surface footprint (e.g., high density development contexts). They can also be referred to as infiltration tanks.

DESIGN GUIDANCE

MONITORING WELLS

Capped vertical non-perforated pipes connected to the inlet and outlet pipes are recommended to provide a means of inspecting and flushing them out as part of routine maintenance. A capped vertical standpipe consisting of an anchored 100 to 150 mm diameter perforated pipe with a lockable cap installed to the bottom of the facility is also recommended for monitoring the length of time required to fully drain the facility between storms. Manholes and inspection ports should be installed in infiltration chambers to provide access for monitoring and maintenance activities.

PRE-TREATMENT

It is important to prevent sediment and debris from entering infiltration facilities because they could contribute to clogging and failure of the system. The following pretreatment devices are options:

- Leaf screens: Leaf screens are mesh screens installed either on the building eavestroughs or roof downspouts and are used to remove leaves and other large debris from roof runoff.

- In-ground devices: Devices placed between a conveyance pipe and the facility (e.g., oil and grit separators, sedimentation chamber or goss traps), that can be designed to remove both large and fine particulate from runoff. A number of proprietary stormwater filter designs are available

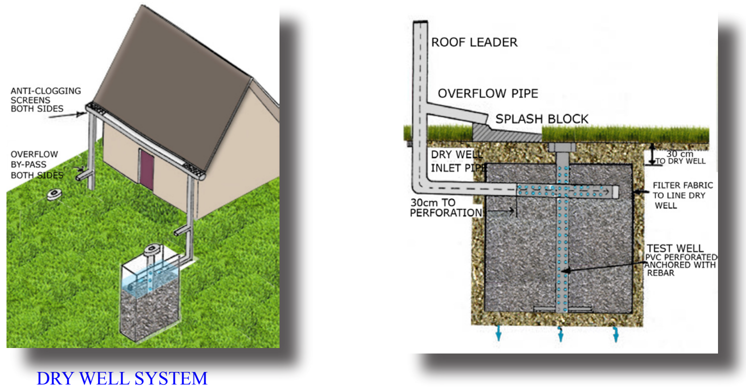
- Vegetated filter strips or grass swales: Road and parking lot runoff can be pretreated with vegetated filter strips or grass swales prior to entering the infiltration practice

FILTER MEDIA

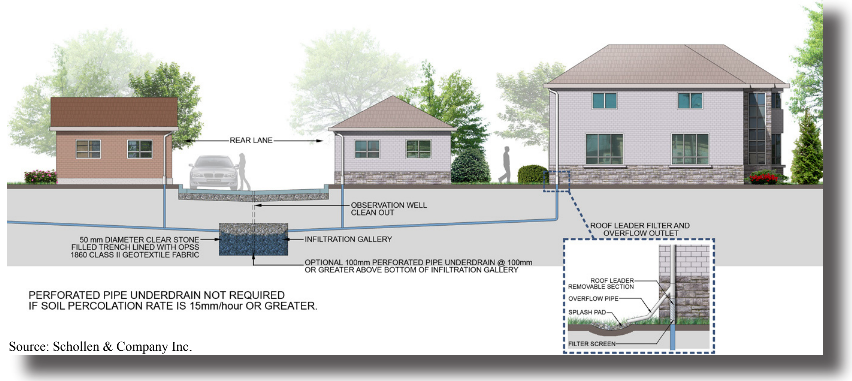
- Stone reservoir: Soakaways and infiltration trenches should be filled with uniformly-graded, washed stone that provides 30 to 40% void space. Granular material should be 50 mm clear stone
- Geotextile: A non-woven needle punched, or woven monofilament geotextile fabric should be installed around the stone reservoir of soakaways and infiltration trenches with a minimum overlap at the top of 300 mm. Woven slit film and non-woven heat bonded fabrics should not be used as they are prone to clogging. Specification of geotextile fabrics should consider the apparent opening size (AOS) for non-woven fabrics, or percent open area (POA) for woven fabrics, which affect the long term ability to maintain water flow. Other factors that need consideration include maximum forces to be exerted on the fabric, and the load bearing ratio, texture (i.e., grain size distribution) and permeability of the native soil in which they will be installed.


Source: Lanark Consultants Ltd


Source: Pennsylvania Department of Environmental Protection



DRY WELL SYSTEM



Source: Schollen & Company Inc.

INFILTRATION TRENCH BELOW A LANEWAY

Source: Schollen & Company Inc.

INFILTRATION CHAMBER SYSTEM UNDER A PARKING LOT

GEOMETRY AND SITE LAYOUT

Soakaways and infiltration chambers can be designed in a variety of shapes, while infiltration trenches are typically rectangular excavations with a bottom width generally between 600 and 2400 mm. Facilities should have level or nearly level bed bottoms.

CONVEYANCE AND OVERFLOW

Inlet pipes to soakaways and infiltration trenches are typically perforated pipe connected to a standard non-perforated pipe or eavestrough that conveys runoff from the source area to the facility. The inlet and overflow outlet to the facility should be installed below the maximum frost penetration depth to prevent freezing. The overflow outlet can simply be the perforated pipe inlet that backs up when the facility is at capacity and discharges to a splash pad and pervious area at grade or can be a pipe that is at the top of the gravel layer and is connected to a storm sewer. Outlet pipes must have capacity equal to or greater than the inlet.

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Soakaways, Infiltration Trenches and Chambers	Yes	Yes	Partial, depends on soil infiltration rate

CONSTRUCTION CONSIDERATIONS

SOIL DISTURBANCE AND COMPACTION: Before site work begins, locations of facilities should be clearly marked. Only vehicular traffic used for construction of the infiltration facility should be allowed close to the facility location.

EROSION AND SEDIMENT CONTROL: Infiltration practices should never serve as a sediment control device during construction. Construction runoff should be directed away from the proposed facility location. After the site is vegetated, erosion and sediment control structures can be removed.

COMMON CONCERNS

RISK OF GROUNDWATER CONTAMINATION

Most pollutants in urban runoff are well retained by infiltration practices and soils and therefore, have a low to moderate potential for groundwater contamination. To minimize risk of groundwater contamination the following management approaches are recommended:

- infiltration practices should not receive runoff from high traffic areas where large amounts of de-icing salts are applied (e.g., busy highways), nor from pollution hot spots;
- prioritize infiltration of runoff from source areas that are comparatively less contaminated such as roofs, low traffic roads and parking areas; and,
- apply sedimentation pretreatment practices (e.g., oil and grit separators) before infiltration of road or parking area runoff.

RISK OF SOIL CONTAMINATION

Available evidence from monitoring studies indicates that small distributed stormwater infiltration practices do not contaminate underlying soils, even after 10 years of operation.

ON PRIVATE PROPERTY

Property owners or managers will need to be educated on their routine maintenance needs, understand the long-term maintenance plan, and be subject to a legally binding maintenance agreement. An incentive program such as a storm sewer user fee based on the area of impervious cover on a property that is directly connected to a storm sewer could be used to encourage property owners or managers to maintain existing practices. Alternatively, infiltration practices could be located in an expanded road right-of-way or “stormwater easement” so that municipal staff can access the facility in the event it fails to function properly.

WINTER OPERATION

Soakaways, infiltration trenches and chambers will continue to function during winter months if the inlet pipe and top of the facility is located below the local maximum frost penetration depth.

OPERATION AND MAINTENANCE

Maintenance typically consists of cleaning out leaves, debris and accumulated sediment caught in pretreatment devices, inlets and outlets annually or as needed. Inspection via an monitoring well should be performed to ensure the facility drains within the maximum acceptable length of time (typically 72 hours) at least annually and following every major storm event (>25 mm). If the time required to fully drain exceeds 72 hours, drain via pumping and clean out the perforated pipe underdrain, if present. If slow drainage persists, the system may need removal and replacement of granular material and/or geotextile fabric.

SITE CONSIDERATIONS

Wellhead Protection

Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.

Site Topography

Facilities cannot be located on natural slopes greater than 15%.

Water Table

The bottom of the facility should be vertically separated by one (1) metre from the seasonally high water table or top of bedrock elevation.

Soil

Soakaways, infiltration trenches and chambers can be constructed over any soil type, but hydrologic soil group A or B soils are best for achieving water balance and channel erosion control objectives. If possible, facilities should be located in portions of the site with the highest native soil infiltration rates. Designers should verify the soil infiltration rate at the proposed location and depth through field measurement of hydraulic conductivity under field saturated conditions.

Drainage Area

Typically are designed with an impervious drainage area to treatment facility area ratio of between 5:1 and 20:1. A maximum ratio of 10:1 is recommended for facilities receiving road or parking lot runoff.

Pollution Hot Spot Runoff

To protect groundwater from possible contamination, runoff from pollution hot spots should not be treated by soakaways, infiltration trenches or chambers.

Setback from Buildings

Facilities should be setback a minimum of four (4) metres from building foundations.

Proximity to Underground Utilities

Local utility design guidance should be consulted to define the horizontal and vertical offsets. Generally, requirements for underground utilities passing near the practice will be no different than for utilities in other pervious areas. However, the designer should consider the need for long term maintenance when locating infiltration facilities near other underground utilities.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.4 OF THE CVC/TRCA LID SWM GUIDE

BIORETENTION

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

As a stormwater filter and infiltration practice, bioretention temporarily stores, treats and infiltrates runoff. Depending on native soil infiltration rate and physical constraints, the system may be designed without an underdrain for full infiltration, with an underdrain for partial infiltration, or with an impermeable liner and underdrain for filtration only (i.e., a biofilter). The primary component of the practice is the filter bed which is a mixture of sand, fines and organic material. Other elements include a mulch ground cover and plants adapted to the conditions of a stormwater practice. Bioretention is designed to capture small storm events or the water quality storage requirement. An overflow or bypass is necessary to pass large storm event flows. Bioretention can be adapted to fit into many different development contexts and provide a convenient area for snow storage and treatment.

DESIGN GUIDANCE

SOIL CHARACTERISTICS

Bioretention can be constructed over any soil type, but hydrologic soil group A and B are best for achieving water balance goals. If possible, bioretention should be sited in the areas of the development with the highest native soil infiltration rates. Bioretention in soils with infiltration rates less than 15 mm/hr will require an underdrain. Designers should verify the native soil infiltration rate at the proposed location and depth through measurement of hydraulic conductivity under field saturated conditions.

GEOMETRY & SITE LAYOUT

Key geometry and site layout factors include:

- The minimum footprint of the filter bed area is based on the drainage area. Typical drainage areas to bioretention are between 100 m2 to 0.5 hectares. The maximum recommended drainage area is 0.8 hectares. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 15:1.
- Bioretention can be configured to fit into many locations and shapes. However, cells that are narrow may concentrate flow as it spreads throughout the cell and result in erosion.
- The filter bed surface should be level to encourage stormwater to spread out evenly over the surface.

PRE-TREATMENT

Pretreatment prevents premature clogging by capturing coarse sediment particles before they reach the filter bed. Where the runoff source area produces little sediment, such as roofs, bioretention can function effectively without pretreatment. To treat parking area or road runoff, a two-cell design that incorporates a forebay is recommended. Pretreatment practices that may be feasible, depending on the method of conveyance and the availability of space include:

- *Two-cell design (channel flow):* Forebay ponding volume should account for 25% of the water quality storage requirement and be designed with a 2:1 length to width ratio.
- *Vegetated filter strip (sheet flow):* Should be a minimum of three (3) metres in width. If smaller strips are used, more frequent maintenance of the filter bed can be anticipated.
- *Gravel diaphragm (sheet flow):* A small trench filled with pea gravel, which is perpendicular to the flow path between the edge of the pavement and the bioretention practice will promote settling out of sediment and maintain sheet flow into the facility. A drop of 50-150 mm into the gravel diaphragm can be used to dissipate energy and promote settling.
- *Rip rap and/or dense vegetation (channel flow):* Suitable for small bioretention cells with drainage areas less than 100 square metres.

GRAVEL STORAGE LAYER

- DEPTH: Should be a minimum of 300 mm deep and sized to provide the required storage volume. Granular material should be 50 mm diameter clear stone.
- PEA GRAVEL CHOKING LAYER: A 100 mm deep layer of pea gravel (3 to 10 mm diameter clear stone) should be placed on top of the coarse gravel storage layer as a choking layer separating it from the overlying filter media bed.

FILTER MEDIA

- COMPOSITION: To ensure a consistent and homogeneous bed, filter media should come pre-mixed from an approved vendor.
- DEPTH: Recommended depth is between 1.0 and 1.25 m. However in constrained applications, pollutant removal benefits may be achieved in beds as shallow as 500 mm. If trees are to be included in the design, bed depth must be at least 1.0 m.
- MULCH: A 75 mm layer of mulch on the surface of the filter bed enhances plant survival, suppresses weed growth and pretreats runoff before it reaches the filter bed.

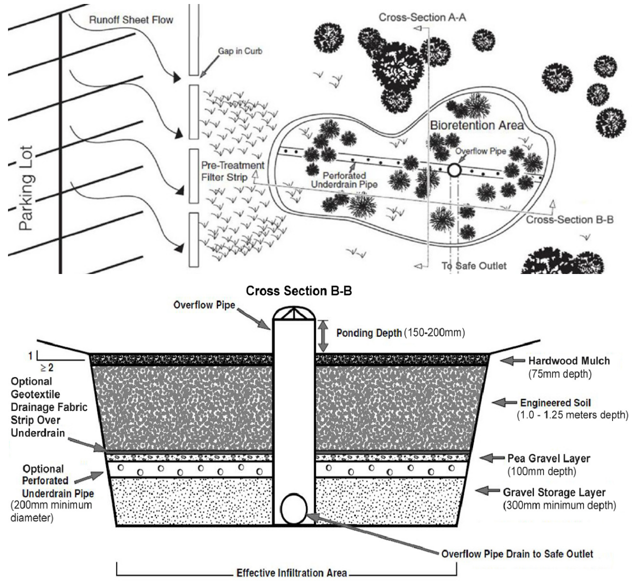
CONVEYANCE AND OVERFLOW

Bioretention can be designed to be inline or offline from the drainage system. Inline bioretention accepts all flow from a drainage area and conveys larger event flows through an overflow outlet. Overflow structures must be sized to safely convey larger storm events out of the facility. The invert of the overflow should be placed at the maximum water surface elevation of the bioretention area, which is typically 150-250 mm above the filter bed surface.

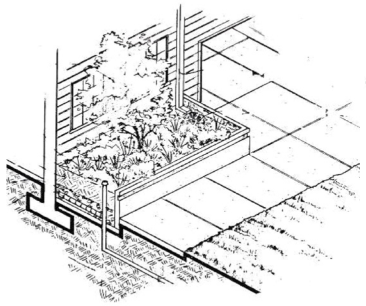
Offline bioretention practices use flow splitters or bypass channels that only allow the required water quality storage volume to enter the facility. This may be achieved with a pipe, weir, or curb opening sized for the target flow, but in conjunction, create a bypass channel so that higher flows do not pass over the surface of the filter bed. Using a weir or curb opening minimizes clogging and reduces maintenance frequency.

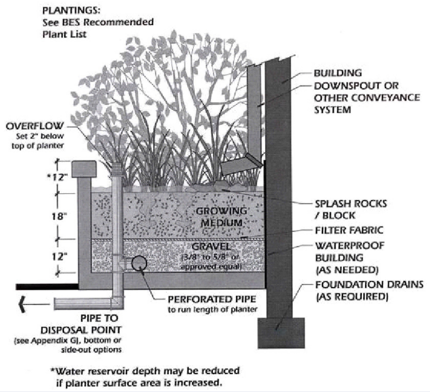
Source: Minnesota Businesses for Clean Water

Source: City of Portland



Source: Wisconsin Department of Natural Resources





Source: City of Portland

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefits
Bioretention with no underdrain	Yes	Yes - size for water quality storage requirement	Partial - based on available storage volume and infiltration rates
Bioretention with underdrain	Partial - based on available storage volume beneath the underdrain and soil infiltration rate	Yes - size for water quality storage requirement	Partial - based on available storage volume beneath the underdrain and soil infiltration rate
Bioretention with underdrain and impermeable liner	Partial - some volume reduction through evapotranspiration	Yes - size for water quality storage requirement	Partial - some volume reduction through evapotranspiration

UNDERDRAIN

- Only needed where native soil infiltration rate is less than 15 mm/hr (hydraulic conductivity of less than 1x10-6 cm/s).
- Should consist of a perforated pipe embedded in the coarse gravel storage layer at least 100 mm above the bottom.
- A strip of geotextile filter fabric placed between the filter media and pea gravel choking layer over the perforated pipe is optional to help prevent fine soil particles from entering the underdrain.
- A vertical standpipe connected to the underdrain can be used as a cleanout and monitoring well.

MONITORING WELLS

A capped vertical stand pipe consisting of an anchored 100 to 150 mm diameter perforated pipe with a lockable cap installed to the bottom of the facility is recommended for monitoring drainage time between storms.

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Filter Media Composition	Filter Media Soil Mixture to contain: • 85 to 88% sand • 8 to 12% soil fines • 3 to 5% organic matter (leaf compost) Other Criteria: • Phosphorus soil test index (P-Index) value between 10 to 30 ppm • Cationic exchange capacity (CEC) greater than 10 meq/100 g • Free of stones, stumps, roots and other large debris • pH between 5.5 to 7.5 • Infiltration rate greater than 25 mm/hr	Recommended depth is between 1.0 and 1.25 metres.
Mulch Layer	Shredded hardwood bark mulch	A 75 mm layer on the surface of the filter bed
Geotextile	Material specifications should conform to Ontario Provincial Standard Specification (OPSS) 1860 for Class II geotextile fabrics. Should be woven monofilament or non-woven needle punched fabrics. Woven slit film and non-woven heat bonded fabrics should not be used as they are prone to clogging. For further guidance see CVC/TRCA LID SWM Planning and Design Guide, Table 4.5.5.	Strip over the perforated pipe underdrain (if present) between the filter media bed and gravel storage layer (stone reservoir)
Gravel	Washed 50 mm diameter clear stone should be used to surround the underdrain and for the gravel storage layer Washed 3 to 10 mm diameter clear stone should be used for pea gravel choking layer.	Volume based on dimensions, assuming a void space ratio of 0.4.
Underdrain	Perforated HDPE or equivalent, minimum 100 mm diameter, 200 mm recommended.	• Perforated pipe for length of cell. • Non-perforated pipe as needed to connect with storm drain system. • One or more caps. • T's for underdrain configuration

CONSTRUCTION CONSIDERATIONS

Ideally, bioretention sites should remain outside the limit of disturbance until construction of the bioretention begins to prevent soil compaction by heavy equipment. Locations should not be used as sediment basins during construction, as the concentration of fines will prevent post-construction infiltration. To prevent sediment from clogging the surface of a bioretention cell, stormwater should be diverted away from the bioretention until the drainage area is fully stabilized.

For further guidance regarding key steps during construction, see the CVC/TRCA LID SWM Planning and Design Guide, Section 4.5.2 - Construction Considerations)

OPERATION AND MAINTENANCE

Bioretention requires routine inspection and maintenance of the landscaping as well as periodic inspection for less frequent maintenance needs or remedial maintenance. Generally, routine maintenance will be the same as for any other landscaped area; weeding, pruning, and litter removal. Regular watering may be required during the first two years until vegetation is established.

For the first two years following construction the facility should be inspected at least quarterly and after every major storm event (> 25 mm). Subsequently, inspections should be conducted in the spring and fall of each year and after major storm events. Inspect for vegetation density (at least 80% coverage), damage by foot or vehicular traffic, channelization, accumulation of debris, trash and sediment, and structural damage to pretreatment devices.

Trash and debris should be removed from pretreatment devices, the bioretention area surface and inlet and outlets at least twice annually. Other maintenance activities include reapplying mulch, pruning, weeding replacing dead vegetation and repairing eroded areas as needed. Remove accumulated sediment on the bioretention area surface when dry and exceeding 25 mm depth.

SITE CONSIDERATIONS

Wellhead Protection

Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.

Available Space

Reserve open areas of about 10 to 20% of the size of the contributing drainage area.

Site Topography

Contributing slopes should be between 1 to 5%. The surface of the filter bed should be flat to allow flow to spread out. A stepped multi-cell design can also be used.

Available Head

If an underdrain is used, then 1 to 1.5 metres elevation difference is needed between the inflow point and the downstream storm drain invert.

Water Table

A minimum of one (1) metre separating the seasonally high water table or top of bedrock elevation and the bottom of the practice is necessary.

Soils

Bioretention can be located over any soil type, but hydrologic soil group A and B soils are best for achieving water balance benefits. Facilities should be located in portions of the site with the highest native soil infiltration rates. Where infiltration rates are less than 15 mm/hr (hydraulic conductivity less than 1x10-6 cm/s) an underdrain is required. Native soil infiltration rate at the proposed facility location and depth should be confirmed through measurement of hydraulic conductivity under field saturated conditions.

Drainage Area & Runoff Volume

Typical contributing drainage areas are between 100 m2 to 0.5 hectares. The maximum recommended contributing drainage area is 0.8 hectares. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 15:1.

Pollution Hot Spot Runoff

To protect groundwater from possible contamination, runoff from pollution hot spots should not be treated by bioretention facilities designed for full or partial infiltration. Facilities designed with an impermeable liner (filtration only facilities) can be used to treat runoff from pollution hot spots.

Proximity to Underground Utilities

Designers should consult local utility design guidance for the horizontal and vertical clearances required between storm drains, ditches, and surface water bodies.

Overhead Wires

Check whether the future tree canopy height in the bioretention area will interfere with existing overhead phone and power lines.

Setback from Buildings

If an impermeable liner is used, no setback is needed. If not, a four (4) metre setback from building foundations should be applied.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.5 OF THE CVC/TRCA LID SWM GUIDE