

**Stormwater Management Report
642 Dunlop Street West**

City of Barrie

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WMI File # 21-710

Prepared by

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1.0 Background

1.1 General

WMI & Associates Limited has been retained by 2507517 Ontario Inc. to prepare a Stormwater Management Report in support of a proposed mixed use commercial and residential condominium development located at 642 Dunlop Street West, in the City of Barrie.

This report presents an investigation of existing storm services and drainage patterns, and a detailed design of stormwater management controls in support of a site plan application for the subject development. Fundamental servicing and drainage objectives, as detailed in this report, are based on pre-consultation information from the City of Barrie and the Lake Simcoe Region Conservation Authority (LSRCA) as well as typical standards and best management practices as set out by the City, LRSOA, and the Ministry of the Environment, Conservation and Parks (MECP). Municipal drainage plans and plan and profile drawings of the surrounding municipal roads obtained from the City of Barrie were also referenced to determine area-wide drainage patterns and the extent municipal infrastructure.

1.2 Site Description

The subject site comprises 0.81ha and is located at the north-east corner of Miller Drive and Dunlop Street West, near the western limits of the City. The site is presently occupied by a mechanic shop and a used car lot, and consists of two small single story buildings, asphalt and gravel parking areas, and outdoor storage areas for decommissioned vehicles and vehicle parts. An equipment rental facility is located to the east of the site, and a gas station / convenience store is located to west, opposite Miller Drive. The lands immediately to the north are undeveloped wetland, and to the south (opposite Dunlop Street West) lies undeveloped wooded lands.

The development proposed is a 5-storey building containing 72 residential units, 6 ground floor commercial units, and a ground floor parking garage. The site will be accessed via driveways onto Miller Drive and Dunlop Street West, and will also include an asphalt parking lot, landscaped areas, and an undeveloped wetland buffer area to the north.

The property is legally referred to as Part of Lot 25, Concession 7, in the City of Barrie, Geographic Township of Vespra. This report is based upon a Site Plan prepared by N Architecture Inc. dated March 13, 2023. The Site Plan, and Site Location Plan are contained in **Appendix A** for reference.

2.0 Stormwater Management

2.1 Existing Topography and Pre-Development Drainage

The site is generally flat, with prevailing slopes in the order of 0.5-2.0%. It is drained via overland sheet flow onto Miller Drive to the west, Dunlop Street to the south, and directly to the undeveloped wetland to the north. There is no storm drainage infrastructure within the site.

Both Dunlop Street West and Miller Drive and their associated drainage systems across the frontage of the subject site were recently reconstructed as a part of the Dunlop Street West and Miller Drive Improvements project. The drainage systems consist of open ditches and culverts. Dunlop Street Drainage is generally directed towards the east through the roadside ditches and driveway culverts, and outlets are provided to the south via road crossing culverts. Miller Drive is drained by roadside ditches that drain southward prior to outletting westward through a culvert crossing Miller Drive near the Dunlop St. West/ Miller Drive intersection. It is noted that the ditches & culverts in the area all have minimal grade, as is consistent with the flat / low-lying topography that characterizes the area.

The storm drainage plan for the Dunlop St. West & Tiffin Street / Miller Drive Intersection Improvements project (drawing STM1, dated March 15, 2017) details the area-wide drainage patterns, catchment areas, and runoff coefficients in the area of the subject site. From this, it is determined that the south/east portion of the site is captured by the Dunlop Street West ditches, and the north/ west portion of the site is captured by the northern undeveloped wetland area and the Miller Drive ditches. It is noted that the drainage area delineations as detailed on this drawing generally correlates with the observed pre-development drainage based on actual site topography.

At the site level, the south-east drainage area comprises 0.21ha and is herein referred to as catchment PRE-E. The western draining portion, herein referred to as catchment PRE-W, comprises 0.60ha.

Refer to the Pre-Development Drainage Plan, **Figure 2**, contained in **Appendix A**. A copy of the Drainage Plan from the Dunlop St. West and Tiffin St. / Miller Dr. Intersection Improvements project is also contained in **Appendix A**.

2.2 Subsurface Conditions

A Geotechnical Investigation has been conducted for the subject site, by Terraprobe Inc., dated April 6, 2022. This investigation consisted of 8 exploratory boreholes extending to depths of 1.8m to 12.6m below the ground surface. The Investigation reveals that the surficial soils generally consists of topsoil and asphalt/ gravel to depths of 0.13 – 0.45m. This is underlain by a layer of peat and/ or sand / silty sand earth fill, extending to depths ranging from 0.5 to 1.5m below grade. The underlying native soils were found to be a sand/ silty sand, which extends to the bottom of each borehole. Groundwater table depths were initially

determined to be between 1.5-1.9m below the ground surface, as measured from the 5 standpipes.

A Hydrogeological Assessment has also been conducted by Terraprobe Inc. dated March 16, 2023, to further assess the local groundwater conditions, soils permeability, and to provide a water balance assessment. The results of the groundwater monitoring program indicates that the seasonal high water table is in the order of 0.8-1.2m below the existing ground surface. The provided water balance assessment reveals that natural infiltration is reduced by 958m³/year in the post-development condition, without onsite mitigation.

From information provided in the Hydrogeological Assessment and from supplemental information provided by Terraprobe, it is estimated that the shallow soils exhibit a percolation rate in the order of 5min/cm (120mm/hr) – 12min/cm (50mm/hr).

Based on data from the Soils Map of Simcoe County, published by the Canada Department of Agriculture (1959), the soil deposit identified at the site location is Muck, and the surrounding soils are identified as either Tioga or Vasey Sandy Loam. As is substantiated by the geotechnical investigation, predominant native soil deposit is a sand/ silty sand material. The properties of these soils are generally consistent with that of the sandy loam material that is predominant on the lands surrounding the subject site. The Tioga and Vasey Sandy Loam soils are considered to have good to imperfect drainage characteristics, and both are classified within Hydrologic Soils Group 'AB'.

The Geotechnical Investigation, Hydrogeological Assessment, and supplemental percolation rate information (all provided by Terraprobe Inc.) are all contained in **Appendix C** for reference.

2.3 Design Criteria Guidelines

The stormwater management design principles for the proposed development incorporates the policies and criteria of the Ministry of the Environment, Conservation and Parks (MECP), the Lake Simcoe Region Conservation Authority (LSRCA), the Lake Simcoe Protection Plan (LSPP), the City of Barrie, and as well utilizes best management practices (BMPs) for the design of various aspects of the stormwater management system. The BMPs employed through this design utilize various Low Impact Development (LID) design initiatives in order to minimize the environmental impact of the proposed development. Design guidance for such systems is based on the Low Impact Development Stormwater Management Planning & Design Guide (LID Manual) prepared by the Credit Valley Conservation (CVC) and the Toronto and Region Conservation Authority (TRCA), Version 1.0, dated 2010.

The stormwater management design criteria for the development are summarized as follows:

- The City of Barrie rainfall intensity-duration-frequency (IDF) curves will be used to determine the peak design flows and runoff volumes for each of the design storm events analyzed.

- Quantity controls are to be provided to attenuate peak flows to pre-development levels for all storm events.
- Quality control will be provided at an 'enhanced' level as defined by The Ministry of the Environment's (MOE) Stormwater Management Planning and Design Manual (March 2003).
- Phosphorus reduction, water balance controls, and runoff volume control will be achieved to the greatest extent possible on-site as per LSRCA guidelines.
- Erosion and sediment control measures will be implemented prior to and during the construction of the development and maintained until the site is stabilized.

2.4 Post-Development Drainage

Drainage patterns in the post-development condition will strive to match pre-development conditions as closely as possible, in particular with regard to maintaining the site's split drainage characteristic as described previously.

The east portion of the site, catchment A1, comprises 0.27ha and consists of the proposed parking lot as well as landscape islands and walkway areas. Runoff from this area is to drain through an internal storm sewer system, and outlet to the Dunlop Street West ditch. Catchment A2 comprises 0.05ha of perimeter landscape areas along the frontage of Dunlop Street West that will drain directly to the adjacent municipal ditch via sheet flow, as this narrow perimeter area cannot reasonably be captured by internal drainage systems.

The north / west portion of the site, catchment B1, comprises 0.25ha and consists of the undeveloped / wetland buffer area, as well as a portion of the driveway and perimeter landscaping along the western site boundary. The undeveloped northern portion of catchment B1 lands will continue to sheet flow northward towards the wetland, and the driveway and perimeter landscape area will sheet flow into the Miller Drive ditch, as this area cannot be captured internally due to topographical constraints.

The building rooftop, consisting of catchments B2a, B2b, and B2c comprises 0.24ha. It is proposed to drain northward into the wetland buffer, via a storm sewer outlet.

Tables 1a & 1b below summarize the uncontrolled pre- and post-development peak flows for various return period design storms.

Table 1a: Pre- & Post-Development Uncontrolled Peak Flows- 'A' Catchments

Catchment I.D.	A (ha)	T _c (min.)	C	Q _{25mm} (m ³ /s)	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)
PRE-E	0.21	10.0	0.52	0.009	0.025	0.033	0.038	0.049	0.060	0.068
A1	0.27	10.0	0.91	0.031	0.057	0.074	0.086	0.111	0.123	0.135
A2	0.05	10.0	0.36	0.001	0.004	0.005	0.006	0.008	0.010	0.011
A1+A2	0.32	10.0	0.82	0.030	0.061	0.080	0.093	0.119	0.144	0.160

Table 1b: Pre- & Post-Development Uncontrolled Peak Flows- 'B' Catchments

Catchment I.D.	A (ha)	T _c (min.)	C	Q _{25mm} (m ³ /s)	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)
PRE-W	0.60	10.0	0.26	0.007	0.036	0.047	0.055	0.071	0.085	0.098
B1	0.25	10.0	0.20	0.002	0.012	0.015	0.018	0.023	0.027	0.031
B2a	0.07	10.0	0.95	0.009	0.015	0.020	0.023	0.029	0.032	0.035
B2b	0.07	10.0	0.95	0.009	0.015	0.020	0.023	0.029	0.032	0.035
B2c	0.10	10.0	0.95	0.012	0.022	0.029	0.033	0.041	0.046	0.050
B2 total	0.24	10.0	0.95	0.030	0.053	0.069	0.080	0.099	0.109	0.120
B1+B2	0.32	10.0	0.87	0.033	0.064	0.084	0.098	0.126	0.146	0.160

Since the total post-development flow from A1+A2 exceeds the pre-development PRE-E peak flow, quantity controls are required for runoff to the south/east that is to outlet to the Dunlop Street West drainage system. Similarly, since B1+B2 flows are greater than the PRE-W peak flows, quantity controls are required for runoff to the north/west, that is to outlet ultimately to the Miller Drive drainage system.

Refer to **Figure 3** for the Post-Development Drainage Plan, contained in **Appendix A**. Supporting storm drainage calculations are contained in **Appendix B**.

2.5 Quantity Control

To achieve the release rates equivalent to pre-development levels, peak flow attenuation is proposed for catchment A1 (parking Lot) and B2 (rooftop) runoff. The allowable release rates to Dunlop Street West ('A' areas) and to Miller Drive ('B' areas) are described in Table 2 below.

Table 2: Allowable Release Rates

Description	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)
Allowable Flow: 'A' catchments = PRE-E - A2 (uncontrolled)	0.021	0.028	0.032	0.041	0.050	0.057
Allowable Flow: 'B' catchments = PRE-W - B1 (uncontrolled)	0.0025	0.0033	0.0039	0.0068	0.0124	0.0163

Attenuation within Area A1 is to be achieved primarily by the internal storm sewer system and via underground storage chambers and parking lot surface ponding within catchment A1.

Table 2a below provides a summary of the post-development peak flows (controlled release for A1, uncontrolled for catchment A2) and maximum storage volumes required to achieve the allowable release rates.

Table 3a: Area 'A' Post-Development Controlled Peak Flows and Max Storage Volumes

Catchment I.D.	Area (ha)	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)	Max Storage Required (m ³)
A1	0.27	0.0125	0.0196	0.0250	0.0258	0.0260	0.0263	106
A2*	0.05	0.004	0.005	0.006	0.008	0.010	0.011	-
Total = A1 + A2*		0.017	0.025	0.031	0.034	0.036	0.038	

*uncontrolled flow

In comparing PRE-E flows from Table 1a to the total flows from Table 3a, it can be seen that the total post-development release rates for all storm events can be achieved to pre-development levels.

Attenuation from catchment B2 is to be achieved by detention storage within the western and central plateau areas of the building rooftop. The outlet will be a storm sewer connection that will discharge to the north of the parking lot.

Table 3b below provides a summary of the post-development peak flows (controlled release for B2a and B2b, uncontrolled for catchments B1 and B2c) and maximum storage volumes required to achieve the allowable release rates.

Table 3b: Area 'B' Post-Development Controlled Peak Flows and Max Storage Volumes

Catchment I.D.	Area (ha)	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)	Max Storage Required (m ³)
B1*	0.25	0.012	0.015	0.018	0.023	0.027	0.031	-
B2a	0.07	0.0012	0.0016	0.0019	0.0024	0.0027	0.0030	40
B2b	0.07	0.0012	0.0016	0.0019	0.0024	0.0027	0.0030	40
B2c*	0.10	0.022	0.029	0.033	0.041	0.046	0.050	-
Total = B1* + B2a + B2b + B2c*		0.036	0.047	0.055	0.069	0.078	0.087	

*uncontrolled flow

In comparing PRE-W flows from Table 1b to the total flows from Table 3b, it can be seen that the total post-development release rate for all storm events can be achieved to pre-development levels.

For supporting storm drainage and stormwater management design details refer to section 3.0. For detailed calculations, refer to **Appendix B**.

2.6 Quality Control & Low Impact Development Initiatives

In determining the best approach to providing quality controls and the associated LIDs for the proposed development, various factors were considered, including the following:

- Existing land characteristics and uses (soils, topography, treatment area, location, etc.);
- The sources of stormwater runoff and degree of contamination from each (i.e. road/parking areas vs. rooftop);
- The magnitude of the increase in impervious area from pre-development conditions;
- Utilizing an 'integrated treatment train' approach to treat stormwater runoff;
- Ability to utilize landscaped areas and providing water balance and nutrient uptake benefits;
- Consideration of water table elevations, and spatial separation from LID's (i.e. 1.0m minimum vertical separation);
- The nature of the native soils and the ability to infiltrate;
- Local requirements and maintenance considerations;
- Constructability / practicality considerations;
- Considerations associated with the natural heritage features.

Through this investigation it has been determined that the native soils are generally conducive to support infiltration, as the infiltration rate is greater than a typical minimum of 15mm/hr that is required for infiltration-based LIDs. However the groundwater table elevation is determined to be relatively shallow, which is a factor which inhibits the ability to utilize infiltration based LIDs.

In consideration of these factors, as well as local requirements, the chosen form of LIDs are mechanical systems applied to parking lot runoff that are to consist of an oil/grit separator and an underground storage tank. It is also noted that clean runoff from the building rooftop is to discharge via a separate sewer outlet to the northern wetland, to provide groundwater recharge benefits. These LIDs will provide the maximum possible quality control benefit within the confines of site constraints as it promotes the treatment train approach to stormwater management.

It is further noted that the proposed development and the application of the LID features represents a significant improvement over the existing condition, in terms of environmental impacts from stormwater since the current use is a mechanic shop and used car lot with an outdoor storage yard for car parts that discharges directly to both the wetland and existing municipal drainage systems without stormwater controls.

The following sub-sections describe how various forms of stormwater quality control will be implemented within the proposed development.

2.6.1 Total Suspended Solids Removal Initiatives

The appropriate level of quality control for this site was determined to be at an 'Enhanced' level as defined by the MOE's Stormwater Management Planning & Design Manual (2003), which equates to providing 80% total suspended solids (TSS) removal efficiency.

Stormwater quality control is to be provided for the proposed parking area (the primary source of contaminated runoff) through the use of an oil/grit separator (OGS) designed to Environmental Treatment Verification (ETV) standards, and through the sediment capture system (isolator rows) that are integrated into the proposed Stormtech underground storage tank system. Pre-treatment will be provided via the goss traps affixed to the catchbasin structures, and by providing deep depth sumps (0.9m depth) within each catchbasin. These initiatives working together as a part of the treatment train will provide a sufficient level of suspended solids removal from the proposed parking lot.

Runoff from catchment B2 is not proposed to be quality controlled since this area mostly comprises rooftop and landscaped areas which produce uncontaminated runoff.

Refer to the quality control design calculations contained in **Appendix B**.

2.6.2 Water Balance Controls

Water Balance Controls are proposed in accordance with the requirements of the Lake Simcoe Protection Plan (LSPP), LSRCA guidelines, and in accordance with the findings of the Water Balance Assessment conducted by Terraprobe Inc. The Water Balance Assessment identifies that there is a net infiltration deficit of 958 m³ annually, but which is reduced to 918 m³ with the addition of the proposed LIDs.

The primary method of achieving water balance onsite is via the rooftop outlet that discharges to the wetland. Although retention is not explicitly provided due to the shallow groundwater table depth and space constraints (close proximity to the 'no touch' wetland buffer), the flow path across pervious surfaces consisting of a flat (0.1% slope) outlet swale and then sheet flow within the wetland and buffer areas will inherently provide infiltration and evapotranspiration benefits which promotes water balance.

Since the parking lot is within a critical sourcewater protection area (Q1 & Q2 area), it is noted that parking lot runoff is not proposed to be infiltrated.

Refer to **Appendix C** for supporting water balance calculations contained within Terraprobe's Hydrogeological Assessment.

2.6.3 Phosphorous Controls

In accordance with LSRCA policy for new development sites within the Lake Simcoe watershed, Stormwater Management Best Management Practices (BMPs) are to be implemented to achieve measurable reductions in phosphorous loading from post-development runoff.

Considering the proposed built form and inability to infiltrate parking lot runoff, the preferred mechanism for providing phosphorus control of parking lot runoff is by way of hydrodynamic separation and underground storage within the Stormtech system.

To quantify the effectiveness of the proposed BMPs, a pre- and post-development phosphorous loading analysis was undertaken using the Lake Simcoe Phosphorous Loading Development Tool (TP Tool), prepared for the Ontario Ministry of the Environment.

The Phosphorous removal efficiency of an underground storage facility is 25% as determined through studies referenced in support of the development of the TP Tool. Additionally, a 20% phosphorous removal efficiency is accredited for oil/grit separators with ETV treatment capability as per the LSRCA's Technical Guideline for Stormwater Management Submissions.

Considering the drainage characteristics of the site, the total post-development phosphorous export from the site without LIDs is determined to be 0.84kg/yr, whereas the total post-development phosphorous export from the site with the LIDs and using the above noted removal efficiencies is estimated to be 0.70kg/yr. In summary, the total phosphorous load

reduction from the site with the implementation of the proposed LIDs is determined to be 0.14kg/yr (0.84kg/yr – 0.70kg/yr), which is a reduction of 17% (0.14kg/yr / 0.84kg/yr).

Furthermore, in consideration of the pre-development phosphorous export of 0.77kg/yr, the total phosphorous reduction achieved with the implementation of the LIDs is 0.07kg/year, which is a 9% reduction over pre-development conditions (0.07kg/yr / 0.77kg/yr).

For supporting calculations, refer to the Phosphorous Loading Development Tool Project Development Summary contained in **Appendix B**.

2.6.4 Runoff Volume Controls

The objective of runoff volume control is to provide volume reduction through stormwater retention (via practices such as infiltration, evapotranspiration), and through runoff filtration is this order of priority. The targeted runoff retention volume for new development sites is 25mm of rainfall from all impervious surfaces as per LSRCA technical guidelines, which equates to a volume of 136m³ from the total impervious area of 5440m². The application of rate control BMPs is considered an 'additional technique to consider' on sites with constraints, as detailed in the LSRCA's Technical Guidelines for Stormwater Management Submissions.

The LSRCA's Technical Guidelines stipulates that if the 25mm retention volume requirement cannot be met, that alternatives 1, 2, 3 for 'sites with restrictions' shall be considered for implementation. Alternatives 1 & 2 speak to a reduced runoff retention volume requirement, from 12.5mm (alternative #1) to 5mm (alternative #2) over the total impervious area. Alternative #3 speaks to implementing runoff retention off site, within the same catchment area. As it is not permissible to infiltrate both rooftop and parking lot runoff due to the shallow groundwater table and sourcewater protection constraints, stormwater filtration, rate control BMPs, and rainwater harvesting are the remaining options.

Filtration is typically achieved via conveyance controls- such as via filtering a retained volume of runoff through check dams a swale. Filtration of rooftop runoff is achieved through a rock check dam located within the swale that outlets from the roof leader pipe. Due to the limited space available for the swale, the retention volume provided behind the check dam is minimal (~1.0m³).

Therefore, since only 1.0m³ of retention is able to be achieved, alternative #1 (targeted 12.5mm x 5440m² = 68m³), and alternative #2 (targeted 5mm x 5440m² = 27.2m³) is not able to be met. However, since the building rooftop is to drain north towards the adjacent wetland, alternative #3 can be met since the 'off site' wetland will provide this runoff retention similar to the pre-development conditions. Specifically, the volume of runoff retention from the developed portion of the site provided by the wetland is equal to 25mm of rainfall from the rooftop area (0.24ha) that drains into it, which is 60m³ (2400m² x 0.025m), which equates to a rainfall depth of 11mm over the overall impervious area of 5440m².

Furthermore, as discussed in sections 2.5 and 3.0, rate controlled detention of both rooftop and parking lot runoff is provided. Rainwater retention has also been explored as a means to

provide further runoff retention, however given the small area of vegetated landscape areas, it is not feasible to provide this form of runoff retention.

2.6.5 Other Contaminants

It is noted that other contaminants such as heavy metals, oil, grease, pesticides, bacteria and other nutrients could be present in stormwater runoff. These contaminants are not expected in significant concentrations due to the proposed land use (mixed use residential & commercial development), therefore are will not be a concern that warrants any additional/ specific form of quality controls that the proposed LID's within the treatment train will already provide.

The application of winter salt is expected to occur onsite to address safety concerns, however the need for significant application is inherently minimized due to the mild slopes/ grades within the proposed exterior parking lot, and the presence of the interior parking lot which will not be exposed to ice/ snow. Directing the building rooftop drainage to rear landscaped area will also inherently reduce the need for heavy salt application in exterior parking area/ walkways.

The temperature of stormwater is not expected to be a concern since the rooftop outlet to the stormwater storage/ infiltration basin will allow for direct infiltration during frequent storm events prior to runoff, otherwise temperature acclimation will be realized in runoff via the slow release through the orifice tube outlet.

3.0 SWM Control Design Details

3.1 Parking Lot Drainage System

Table 4a below highlights the Stage, Storage, Discharge characteristics of the proposed Stormtech system which services the parking lot.

Table 4a: Parking Lot Facility Stage, Storage, Discharge Characteristics

Storm Event (Year)	Total Area (ha)	"A1" SWM Inflow (m ³ /s) (Table 1a)	"A1" Post-Development Allowable (m ³ /s) (Table 2)	"A1" SWM Outflow (m ³ /s) (Table 3a)	SWM Storage Utilized (m ³)	Estimated Water Levels
2	0.27	0.057	0.021	0.0125	38.5	234.90
5		0.074	0.028	0.0196	46.7	235.35
10		0.086	0.032	0.0250	51.6	235.81
25		0.111	0.041	0.0258	77.9	235.87
50		0.123	0.050	0.0260	91.2	235.90
100		0.135	0.057	0.0263	106.3	235.93

The following details the predominant design features of the proposed drainage system(s) servicing the parking lot:

- All drainage from the parking lot will be routed through the OGS for attenuation prior to entering the underground detention tank.
- Storm runoff from catchment A1 will be detained within the underground pipes, manhole / catchbasin structures, and the Stormtech tank to a maximum depth of 1.38m above the base of the chambers during the 100-year storm event. The maximum storage utilized is 106.3m³.
- The base of the Stormtech system is to be lined with an impermeable thermoplastic liner, to prevent infiltration to the native soils.
- Detention from frequent storm events, (up the 5-year storm event), will be contained below the surface elevation of catchbasins, exclusively within the Stormtech chambers and storm pipes/ structures.
- Detention from infrequent storm events, (from the 10-year through to 100-year storm events), will be achieved via parking lot surface ponding, to a maximum depth of approximately 0.18m.
- Discharge from the Stormtech tank is controlled primarily by a 102mm dia. orifice plate affixed to a control manhole that is located to the south of the Stormtech tank.
- Emergency overflows will be provided via a rectangular curb cut at the south-east corner of the site, and through the eastern driveway for discharge directly to Dunlop Street West in the event of a clog in the storm drainage system.

3.2 Rooftop Drainage System

Table 4b below highlights the Stage, Storage, Discharge characteristics of the proposed rooftop detention system.

Table 4b: Rooftop Facility Stage, Storage, Discharge Characteristics

Storm Event (Year)	Total Area (ha)	"B2a/b" SWM Inflow (m ³ /s) (Table 1b)	"B2a/b" Post-Development Allowable (m ³ /s) (Table 2)	"B2a/b" SWM Outflow (m ³ /s) (Table 3b)	SWM Storage Utilized (m ³)	Estimated Rooftop Water Depth (cm)
2	0.00	0.015	0.0025	0.0012	16.2	2.3
5		0.020	0.0033	0.0016	22.2	3.2
10		0.023	0.0039	0.0019	26.1	3.7
25		0.029	0.0068	0.0024	32.6	4.7
50		0.032	0.0124	0.0027	36.2	5.2
100		0.035	0.0163	0.0030	39.9	5.7

The following details the predominant design features of the proposed drainage system(s) servicing the building rooftop:

- Storm runoff from catchments B2a and B2b, which comprises the western and central portions of the building rooftop will be detained within the stormwater storage/ infiltration basin to maximum depth of 0.057m above the roof deck during the 100-year storm event.
- Detention from all storm events, up to the 100-year storm event will be via Zurn Z105 Control Flow roof drains, which contain weirs that control the flow through each drain. The maximum rooftop storage volume for each section of roof is 33.9m³, with a maximum drawdown time of 23.5 hours.
- The emergency overflow outlet for each section of rooftop will be via scupper drains, in accordance with OBC criteria.
- All rooftop drainage will be directed through a single storm sewer outlet for discharge to the north of the parking lot via an outlet swale and overland flow path after exiting the pipe.

Refer to **Appendix B** for supporting calculations.

4.0 Sediment and Erosion Controls

Effective erosion and sediment control must be established prior to construction commencement and maintained until the site has been stabilized. Proactive measures will be required to limit the amount of sediment travelling downstream. Where site grading is required, exposure of the soil during construction should be minimized to avoid erosion and sedimentation.

Silt Fence: Silt fence will be placed on the down slope of all excavated material and along the limits of the wetland buffer to prevent sediment transport onto adjacent properties and the municipal roadways. Periodic inspections and repairs to the silt fence should be performed regularly, as well as after every rainfall event.

Storm Sewer Inlets: Filter cloth complete with clear stone cover at storm sewer inlet structures (i.e. catchbasins), will ensure sediment laden overland runoff during construction is filtered before entering the municipal storm sewer system.

Mud Mat: Mud tracking from construction traffic must be controlled through the use of a mud-mat consisting of large diameter rip-rap located at the construction site entrance/exit.

Monitoring and Inspection: Erosion monitoring and sediment removal should be undertaken every week, and after every rainfall event. All damaged or clogged control devices or fencing must be repaired immediately.

Refer to the Erosion and Sediment Control drawing contained within the engineering drawing set for further details.

5.0 Summary

This Stormwater Management Report demonstrates how the proposed 642 Dunlop Street West residential & commercial development can be serviced by existing municipal services, and that storm runoff will not impose any adverse effects on the surrounding environment or neighbouring properties.

Specifically:

- Storm drainage from the parking lot will be provided by an internal storm sewer that will discharge to the storm drainage ditches on Dunlop Street West. Drainage from the building rooftop will drain northward via a separate storm sewer towards the undeveloped/ wetland area.
- Peak flow attenuation will be provided by an underground storm detention tank for parking lot runoff, and by a rooftop detention storage system.
- An enhanced level of quality control (80% TSS removal minimum) will be provided by an oil/grit separator and the underground detention tank.
- Water balance will be provided to the greatest extent possible via the rooftop outlet to the adjacent wetland.
- Phosphorous controls are provided by the oil/grit separator and the underground detention tank.
- Runoff Volume controls are provided via runoff retention in the adjacent wetland.
- The use of silt fence, clear stone and filter cloth inlet protection, a construction mud mat, and vegetated buffers will provide adequate sediment and erosion controls during construction.

Based on the above we recommend that this report be accepted in support of site plan approval.

Should you have any questions, or require additional information, please contact the undersigned.

Prepared by:

WMI & Associates Limited

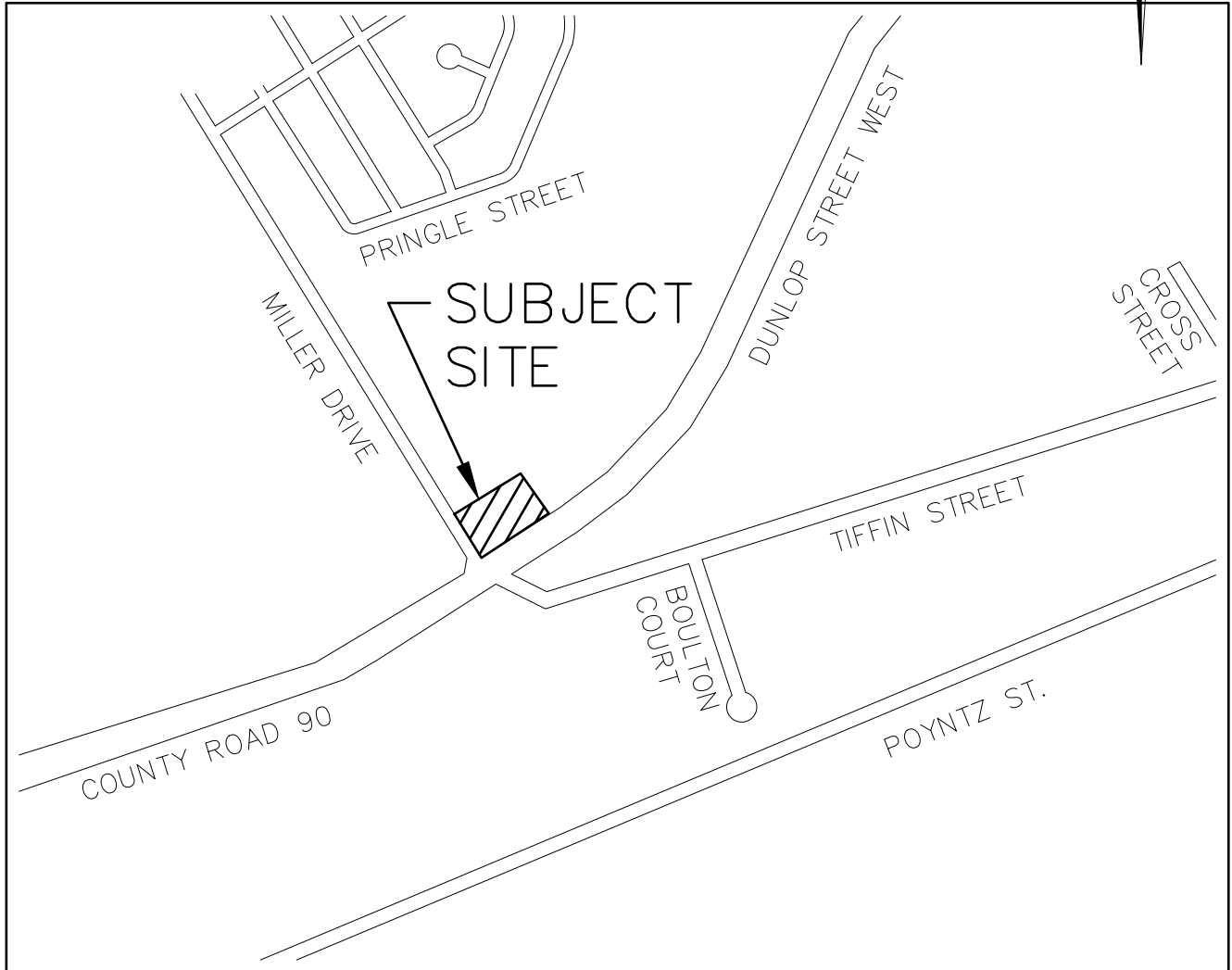
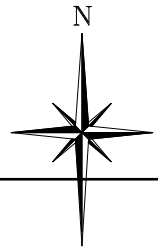


Jonathan P. Reimer, P.Eng.



APPENDIX A

FIGURES AND DRAWINGS



Drawing Title

SITE LOCATION PLAN

Project Title

642 DUNLOP STREET WEST

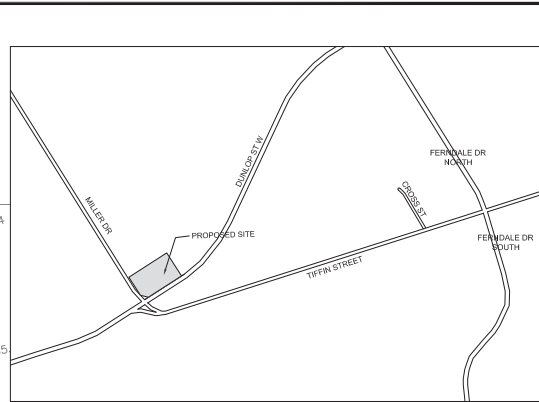
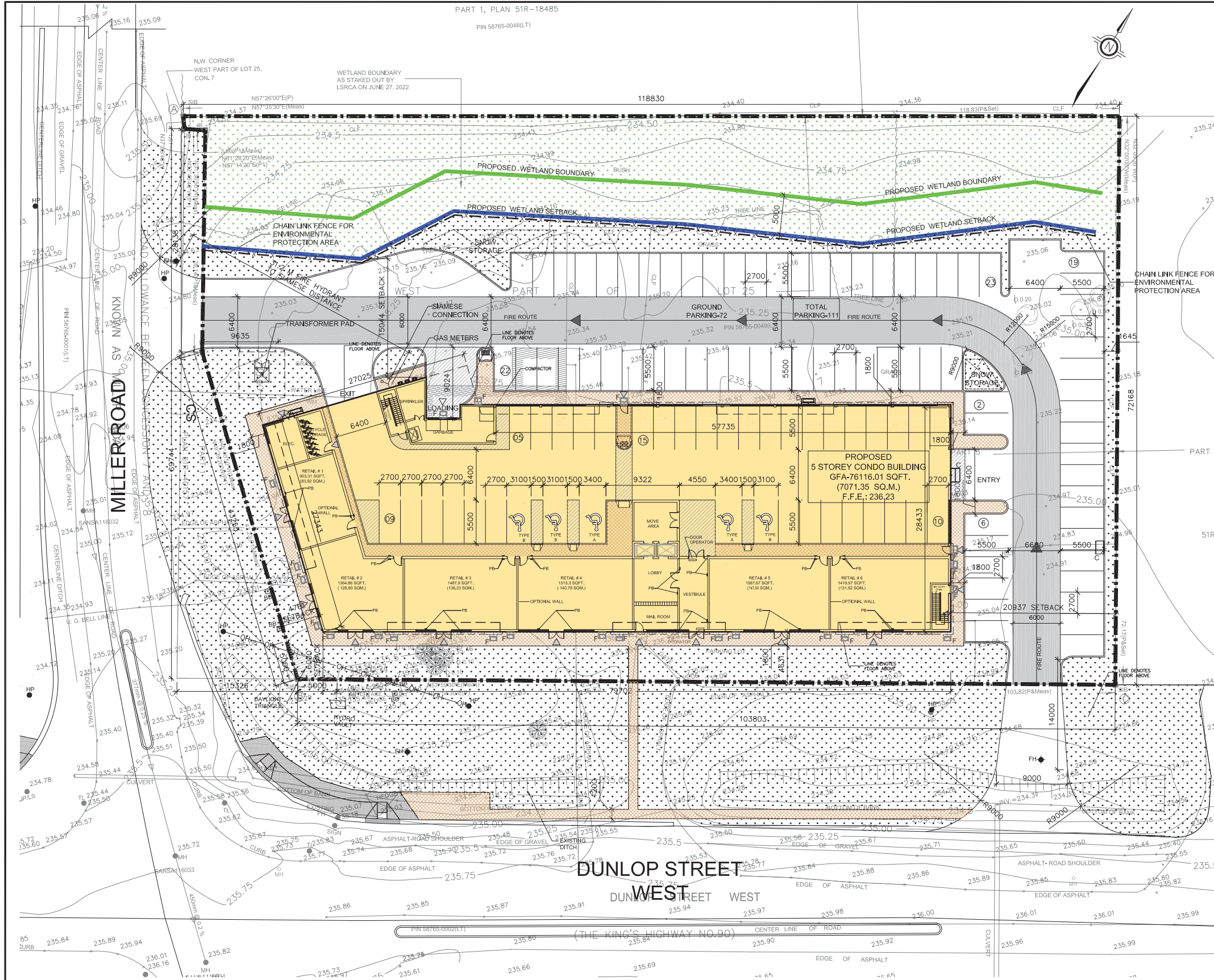


WMI & Associates Limited
119 Collier Street
Barrie, Ontario
L4M 1H5
705-797-2027
www.wmiengineering.ca

Drawn By	AB
Scale	N.T.S.

Checked By	JR
Project No.	21-710

Figure No.
FIG1



KEY PLAN

PROJECT STATISTICS		
ADDRESS 642 DUNLOP STREET WEST, BARRIE, ON.		
ZONING	GENERAL COMMERCIAL (C4)	
	REQUIRED	PROPOSED
LOT(SITE) AREA (m²)	450 SQ.M.	8084.59 SQ.M.
RETAIL		778.89 SQ.M.
RESIDENTIAL UNITS		6233.07 SQ.M.
AMENITY AREA		341.95 SQ.M.
TOTAL G.F.A.		7071.35 SQ.M.
COVERAGE	50% MAX	2420.23 SQ.M. (29.93%)
LANDSCAPE AREA		2811.57 SQ.M. (34.77%)
PAVED AREA		522.91 SQ.M. (6.47%)
BUILDING HEIGHT (MAX.)	14.0 M.	13.95 M.
TOTAL PARKING (MIN.)	128 CARS	111 CARS
BARRIER FREE PARKING (MIN.)	5 CARS	5 CARS
BICYCLE PARKING (MIN.)	4	7
LOADING SPACE (MIN.)	1	1
LOT FRONTAGE (MIN.)	15 M.	69.7 M.
DUNLOP STREET WEST		

BUILDING SETBACKS	REQUIRED	PROPOSED
FRONT YARD (WEST)	6.0M	4.76 M.
EXTERIOR SIDE YARD (SOUTH)	5.0M	6.0 M.
INTERIOR SIDE YARD (NORTH)	3.0M	15.04 M.
REAR YARD (EAST)	7.0M	20.93 M.

PARKING CALCULATIONS				
ITEM	AREA(SQM.)	RATE	REQUIRED	PROVIDED
RETAIL	769.15	1 PER 24 SQ.M.	32	16
RESIDENTIAL UNITS (72)	6233.07	1.25 PER DWELLING	90	90
H.C. PARKING (OVER 100 - 1+3% OF REQ. PARKING)			5	5
TOTAL			127	111
CAR SPACE SIZE		2.7X5.5M	2.7X5.5M	
H/C CAR SPACE SIZE (TYPE-A)		3.4X5.5M	3.4X5.5M	
(TYPE-B)		3.1X5.5M	3.1X5.5M	

- LEGEND
- NEW BUILDING
 - ASPHALT
 - LANDSCAPE
 - CONC. PAVEMENT, 150MM RAISED
 - HANDICAP PARKING
 - MAIN ENTRANCE
 - OVERHEAD DOOR
 - TREE
 - BARRIER CURB
 - BARRIER FREE CURB
 - PB BARRIER FREE PUSH BUTTON

MUNICIPAL ADDRESS & LEGAL DESCRIPTION:
PART 3 OF LOT 25 REGISTERED PLAN F45 CITY OF BARRIE COUNTY OF SIMCOE

APPLICANT:
n Architecture Inc., 9120 Leslie St., Suite 208, Richmond Hill, Ontario L4B 3J9
T: 905.597.5937 F: 1.866.340.5265
E: info@narchitecture.com
WEB: www.narchitecture.com

SURVEY INFORMATION TAKEN FROM:	
OMARI MWINYI SURVEYING LTD. 1905 PINE GROVE AVENUE PICKERING ONTARIO, L1V 6T1 Ph: (905) 728-8863 Fax: (905) 492 5485	
TOTAL UNITS:	
TWO BED ROOM UNITS	33
ONE BED ROOM UNITS	27
ACCESSIBLE ONE BED ROOM UNITS	06
ACCESSIBLE TWO BED ROOM UNITS	06
TOTAL	72

n Architecture Inc
PRINCIPAL: NITIN MALHOTRA, ARCHITECT.
9120 Leslie Street, Suite-208
Richmond Hill, Ontario. L4B 3J9
T : 4 1 6 . 2 5 6 . 9 7 4 1
E : info@narchitecture.com
www.narchitecture.com



13th MARCH 2023
NOT FOR CONSTRUCTION

No.	Date	Version	Dwn.
3.		ISSUED FOR SPA	SB
2.	13-JUN-2018	ISSUED FOR PRE-CONSULTATION	SB
1.	04-APR-2016	ISSUED FOR PRE-CONSULTATION	FK

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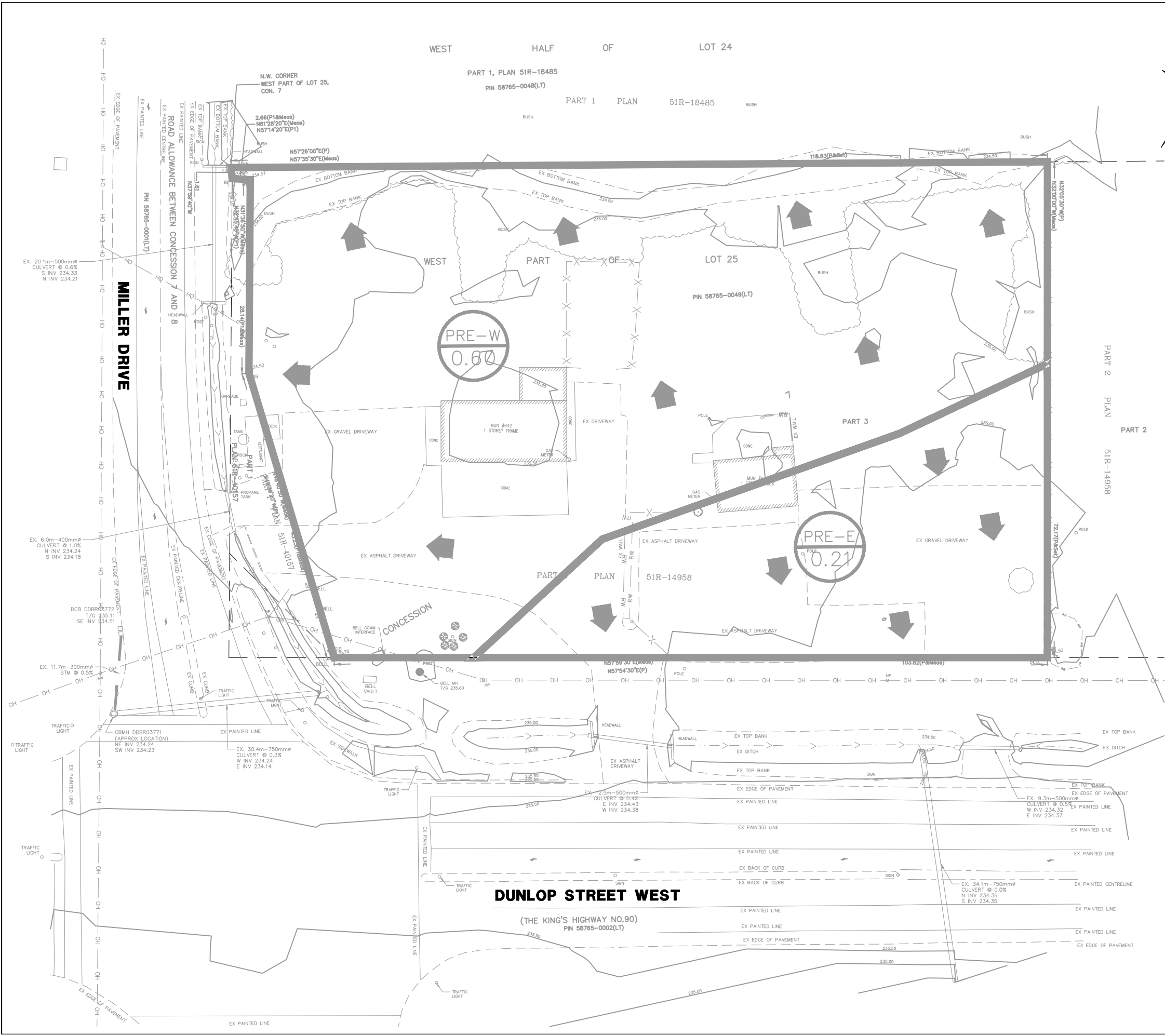
PROJECT:

**PROPOSED 5-STOREY CONDO BUILDING
642 DUNLOP STREET W
BARRIE, ON.**

DRAWING TITLE:	
SITE PLAN	
DRAWN BY: FK	DATE: DEC 02 2015
CHECKED BY: NM	SCALE: AS NOTED
PROJECT NO.:	DRAWING NO.:

15-48 A-1.0

1 SITE PLAN
A-1.0 SCALE: 1:250



Legend:

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0.00

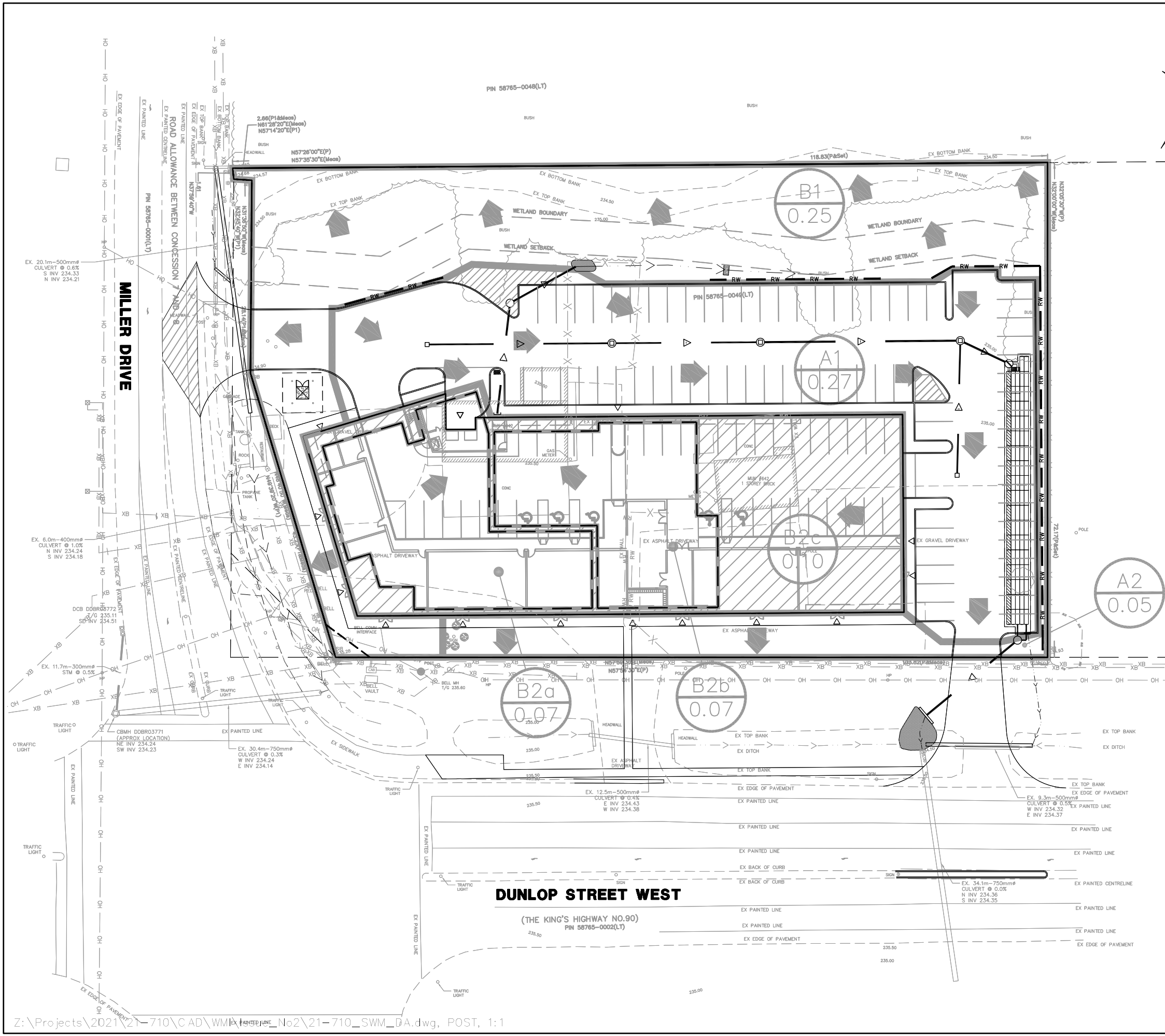
CATCHMENT IDENTIFICATION

CATCHMENT AREA (HA)

LIMITS OF CATCHMENT AREA

OVERLAND FLOW DIRECTION

Drawing Title PRE-DEVELOPMENT DRAINAGE PLAN		
Project Title 642 DUNLOP STREET WEST		
WMI & Associates Limited 119 Collier Street Barrie, Ontario L4M 1H5 705-797-2027 www.wmiengineering.ca		
Drawn By AB	Checked By JR	Figure No. FIG 2
Scale N.T.S.	Project No. 21-710	



Legend:

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0.00

CATCHMENT IDENTIFICATION

0.00

CATCHMENT AREA (HA)

LIMITS OF CATCHMENT AREA

OVERLAND FLOW DIRECTION

Drawing Title		
POST-DEVELOPMENT DRAINAGE PLAN		
Project Title		
642 DUNLOP STREET WEST		
 WMI & Associates Limited 119 Collier Street Barrie, Ontario L4M 1H5 705-797-2027 www.wmiengineering.ca		
Drawn By	AB	Checked By
Scale	N.T.S.	JR
Project No.		21-710
Figure No.		FIG 3



Figure 1: Symbols used in the catchment map.

Legend:

- EXISTING STORM MANHOLE
- EXISTING CATCH BASIN
- PROPOSED STORM MANHOLE
- PROPOSED CATCH BASIN

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SUB CATCHMENT	RUNOFF COEFFICIENT - C	RAINFALL INTENSITY										AREA (km ²)	Q (m ³ /s)	
		15-YEAR STORM					100-YEAR STORM						15-YEAR STORM	100-YEAR STORM
		A	B	C	(mm/hr)	A	B	C	(mm/hr)					
1	0.6	205.02	81.50	0.76	120.36	143.91	58.56	2.25	576	4.73	0.113	0.126		
2	0.67	174.97	13.89	0.76	117.76	147.44	33.89	0.27	676	48.34	2.44	2.366		
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95	0.67	174.97	13.89	0.76	117.76	147.44	33.89	0.27	676	48.34	2.44	2.366		
96	0.67	174.97	13.89											

DATE: 5-10-2017 CALCULATED BY: M.L. CHECKED BY: A.W.

Table 1: Storm Duration (min.)												
COLLECTOR DRAINAGE	STREET	DIRECTION	STATION ADDRESS	LENGTH (ft)	CONTRIBUTING CATCHMENT AREA		Q IN (cfs)	Q OUT (cfs)	Q FULL (cfs)	V FULL (m/s)	Percent Capacity	
					SUBCATCHMENT	AREA (sq. ft)						
DRIVEWAY ¹	DUNLOP ST W	EB	CARR 101	32.00	600-101	3.75	2.22	0.00	126	0.55	0.79	13.8%
DRIVEWAY ²	TYFFIN ST	S/EB	CARR 101	8	1	0.50	0.52	0.00	100	0.65	0.23	22.6%
DRIVEWAY ³	TYFFIN ST	S/EB	401	9.50	8	0.18	0.06	0.04	500	0.65	0.13	3.23
DRIVEWAY ⁴	TYFFIN ST	S/EB	401 + @ STA 19+30	18.50	—	—	—	2.14	500	—	—	—
DRIVEWAY ⁵	TYFFIN ST	S/EB	401 + @ STA 19+50	11.50	—	—	—	2.20	500	—	—	—
DRIVEWAY ⁶	TYFFIN ST	S/EB	516 17+125	14.00	—	—	—	3.25	500	—	—	—
DRIVEWAY ⁷	DUNLOP ST W	WB	402	31.00	—	—	—	0.68	750	—	—	—
DRIVEWAY ⁸	DUNLOP ST W	WB	403	16.00	4	1.12	0.11	0.00	500	0.28	0.09	39.29
DRIVEWAY ⁹	DUNLOP ST W	WB	403	10.00	—	—	—	0.16	500	—	—	—
HYD ACCESS	DUNLOP ST W	WB	516 17+180	9.00	—	—	—	0.00	600	—	—	—
DRIVEWAY ¹⁰	DUNLOP ST W	WB	404	12.00	—	—	—	0.16	500	—	—	—
HYD ACCESS	TYFFIN ST	N/WB	516 19+80	12.00	84-10	2.24	0.01	0.00	800	0.13	0.03	76.92
HYD ACCESS	TYFFIN ST	N/WB	516 19+100	9.00	—	—	—	0.00	500	—	—	—
HYD ACCESS	TYFFIN ST	N/WB	516 19+180	9.00	—	—	—	0.00	500	—	—	—
HYD ACCESS	TYFFIN ST	N/WB	516 19+300	9.00	—	—	—	0.00	600	—	—	—
DRIVEWAY	MILLER DR	WB	404	20.00	174	8.83	0.14	0.00	0.18	0.03	0.23	90.7%
HYD ACCESS	MILLER DR	WB	516 20+174	9.00	—	—	—	0.47	500	—	—	—

DATE: 3/15/2017 CALCULATED BY: M.L. CHECKED BY: A.H.

L = Storm Duration (min)												
CULVERT CROSSING	STREET	DIRECTION	STATION	LENGTH (ft)	CONCRETE SUBCATCHMENT AREA (sq ft)	I (in)	S (in)	D (in)	Q FULL INFE (cfs)	Q FULL OUT (cfs)	Percent Capacity	
ROAD	16400 17 th ST	---	115.11+00	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16400 17 th ST	---	115.11+10	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+00	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+10	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+20	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+30	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+40	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+50	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+60	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+70	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+80	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.12+90	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+00	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+10	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+20	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+30	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+40	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+50	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+60	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+70	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+80	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.13+90	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.14+00	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.14+10	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.14+20	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.14+30	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410 17 th ST	---	115.14+40	10.0	1	1.00	0.10	0.20	750	1.76	0.26	26.72
ROAD	16410											

DATE 3/15/2017 CALCULATED BY ML CHECKED BY LR

No	Issue / Revision	Date	Auth
1	ISSUED FOR GEOTECHNICAL QUOTATION	15/04/20	
2	ISSUED FOR 80% REVIEW - COB	15/08/11	
3	ISSUED FOR 90% REVIEW	16/11/06	
4	ISSUED FOR 100% REVIEW	17/02/03	
5	ISSUE FOR TENDER	17/03/15	



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**DUNLOP ST. W & TIFFIN ST./MILLER DR. INTERSECTION
IMPROVEMENTS**
STORM DRAINAGE PLAN
TIFFIN STREET/MILLER DRIVE & DUNLOP ST. W.

Drawn	Checked	Designed	Checked	Date	Drawing No
ML	DD	DD	GG	17/03/15	STM1
Date 1/2000	Project No RQ0479620	Revision No 5			

APPENDIX B

STORM DRAINAGE & STORMWATER MANAGEMENT CALCULATIONS



WMI & Associates Limited
119 Collier Street, Barrie, Ontario L4M 1H5
p (705) 797-2027 f (705) 797-2028

**RUNOFF COEFFICIENT CALCULATIONS
"C" SPREADSHEET**

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

RUNOFF COEFFICIENT NUMBERS

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade	0.22	0.35	0.55
	5 - 10% grade	0.3	0.45	0.6
	10 - 30% grade	0.4	0.65	0.7
Pasture Land	0 - 5% grade	0.1	0.28	0.4
	5 - 10% grade	0.15	0.35	0.45
	10 - 30% grade	0.22	0.4	0.55
Woodlot or Cutover	0 - 5% grade	0.08	0.25	0.35
	5 - 10% grade	0.12	0.3	0.42
	10 - 30% grade	0.18	0.35	0.52
Lakes and Wetlands		0.05	0.05	0.05
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.95	0.95	0.95
Gravel	(not used for proposed parking or storage areas)	0.4	0.5	0.6
Residential	Single Family	0.3	0.4	0.5
	Multiple (i.e. semi, townhouse, apartment, etc.)	0.5	0.6	0.7
Industrial	Light	0.55	0.65	0.75
	Heavy	0.65	0.75	0.85
Commercial		0.6	0.7	0.8
Unimproved Areas		0.1	0.2	0.3
Lawn	< 2% grade	0.05	0.11	0.17
	2 - 7% grade	0.1	0.16	0.22
	> 7% grade	0.15	0.25	0.35

Ref: Runoff Coefficient Numbers - Adapted from Design Chart 1.07, Ontario Ministry of Transportation, "MTO Drainage Management Manual", MTO. (1997)



<<< Elements Requiring Input Information

POST-DEVELOPMENT CONDITION - CATCHMENT A1

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.257		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade	0.013		
	2 - 7% grade			
	> 7% grade			

Total Area (ha) = 0.27

Runoff Coefficient, C = 0.91

POST-DEVELOPMENT CONDITION - CATCHMENT A2

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.015		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade			
	2 - 7% grade	0.034		
	> 7% grade			

Total Area (ha) = 0.05

Runoff Coefficient, C = 0.36

POST-DEVELOPMENT CONDITION - CATCHMENT B1

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade	0.093		
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade	0.103		
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.032		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade			
	2 - 7% grade	0.023		
	> 7% grade			

Total Area (ha) = 0.25

Runoff Coefficient, C = 0.20

POST-DEVELOPMENT CONDITION - CATCHMENT B2a

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.070		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade			
	2 - 7% grade			
	> 7% grade			

Total Area (ha) = 0.07

Runoff Coefficient, C = 0.95

POST-DEVELOPMENT CONDITION - CATCHMENT B2b

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.070		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade			
	2 - 7% grade			
	> 7% grade			

Total Area (ha) = 0.07

Runoff Coefficient, C = 0.95

POST-DEVELOPMENT CONDITION - CATCHMENT B2c

Land Cover		Hydrologic Soil Groups		
		A-AB	B-BC	C-D
Cultivated Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Pasture Land	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Woodlot or Cutover	0 - 5% grade			
	5 - 10% grade			
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)	0.100		
Gravel	(not used for proposed parking or storage areas)			
Residential	Single Family			
	Multiple (i.e. semi, townhouse, apartment, etc.)			
Industrial	Light			
	Heavy			
Commercial				
Unimproved Areas				
Lawn	< 2% grade			
	2 - 7% grade			
	> 7% grade			

Total Area (ha) = 0.10

Runoff Coefficient, C = 0.95

Z:\Projects\2021\21-710\Design\Storm\220310_C_CALC.S.xlsx]C CALC.S



MODIFIED RATIONAL METHOD CALCULATIONS
2-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	2-year	0.27	0.91	0.91	10	2	0.0125

NOTES: 2-year pre-development target is 0.021 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	83.1	0.0567	34.03	7.50	26.53	
12	75.2	0.0513	36.97	8.25	28.72	
14	68.9	0.0470	39.48	9.00	30.48	
16	63.6	0.0434	41.68	9.75	31.93	
18	59.2	0.0404	43.63	10.50	33.13	
20	55.4	0.0378	45.38	11.25	34.13	
22	52.1	0.0356	46.98	12.00	34.98	
24	49.3	0.0336	48.43	12.75	35.68	
26	46.8	0.0319	49.78	13.50	36.28	
28	44.5	0.0304	51.03	14.25	36.78	
30	42.5	0.0290	52.20	15.00	37.20	
32	40.7	0.0278	53.30	15.75	37.55	
34	39.0	0.0266	54.33	16.50	37.83	
36	37.5	0.0256	55.30	17.25	38.05	
38	36.1	0.0247	56.23	18.00	38.23	
40	34.9	0.0238	57.11	18.75	38.36	
42	33.7	0.0230	57.95	19.50	38.45	
44	32.6	0.0223	58.75	20.25	38.50	
46	31.6	0.0216	59.52	21.00	38.52	38.5
48	30.7	0.0209	60.26	21.75	38.51	
50	29.8	0.0203	60.97	22.50	38.47	
52	29.0	0.0198	61.66	23.25	38.41	
54	28.2	0.0192	62.32	24.00	38.32	
56	27.5	0.0187	62.96	24.75	38.21	
58	26.8	0.0183	63.58	25.50	38.08	
60	26.1	0.0178	64.18	26.25	37.93	
62	25.5	0.0174	64.76	27.00	37.76	
64	24.9	0.0170	65.32	27.75	37.57	
66	24.4	0.0166	65.87	28.50	37.37	
68	23.8	0.0163	66.40	29.25	37.15	
70	23.3	0.0159	66.92	30.00	36.92	



MODIFIED RATIONAL METHOD CALCULATIONS
5-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

Z:\Projects\2021\21-710\Design\Storm\230310_Modified_Rational_Method_Calcs(A,B,C).xlsx\A1(pkq)-2-YR

Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	5-year	0.27	0.91	0.91	10	2	0.0196

NOTES: 5-year pre-development target is 0.028 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	108.9	0.0743	44.60	11.76	32.84	
12	98.8	0.0674	48.54	12.94	35.61	
14	90.6	0.0618	51.93	14.11	37.82	
16	83.8	0.0572	54.91	15.29	39.62	
18	78.1	0.0533	57.56	16.46	41.09	
20	73.2	0.0500	59.94	17.64	42.30	
22	69.0	0.0471	62.12	18.82	43.31	
24	65.2	0.0445	64.12	19.99	44.13	
26	62.0	0.0423	65.97	21.17	44.80	
28	59.0	0.0403	67.69	22.34	45.35	
30	56.4	0.0385	69.30	23.52	45.78	
32	54.0	0.0369	70.82	24.70	46.12	
34	51.9	0.0354	72.25	25.87	46.37	
36	49.9	0.0341	73.60	27.05	46.55	
38	48.1	0.0328	74.89	28.22	46.66	
40	46.5	0.0317	76.11	29.40	46.71	46.7
42	44.9	0.0307	77.28	30.58	46.70	
44	43.5	0.0297	78.40	31.75	46.65	
46	42.2	0.0288	79.48	32.93	46.55	
48	41.0	0.0280	80.51	34.10	46.41	
50	39.8	0.0272	81.51	35.28	46.23	
52	38.7	0.0264	82.47	36.46	46.01	
54	37.7	0.0257	83.39	37.63	45.76	
56	36.8	0.0251	84.29	38.81	45.48	
58	35.9	0.0245	85.16	39.98	45.18	
60	35.0	0.0239	86.00	41.16	44.84	
62	34.2	0.0233	86.82	42.34	44.49	
64	33.4	0.0228	87.62	43.51	44.10	
66	32.7	0.0223	88.39	44.69	43.70	
68	32.0	0.0218	89.14	45.86	43.28	
70	31.4	0.0214	89.88	47.04	42.84	



MODIFIED RATIONAL METHOD CALCULATIONS
10-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	10-year	0.27	0.91	0.91	10	2	0.025

NOTES: 10-year pre-development target is 0.032 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	126.5	0.0864	51.82	15.00	36.82	
12	114.9	0.0784	56.44	16.50	39.94	
14	105.4	0.0719	60.42	18.00	42.42	
16	97.6	0.0666	63.92	19.50	44.42	
18	91.0	0.0621	67.04	21.00	46.04	
20	85.3	0.0582	69.86	22.50	47.36	
22	80.4	0.0549	72.43	24.00	48.43	
24	76.1	0.0519	74.80	25.50	49.30	
26	72.3	0.0493	76.98	27.00	49.98	
28	68.9	0.0470	79.02	28.50	50.52	
30	65.9	0.0450	80.93	30.00	50.93	
32	63.1	0.0431	82.73	31.50	51.23	
34	60.6	0.0414	84.42	33.00	51.42	
36	58.4	0.0398	86.03	34.50	51.53	
38	56.3	0.0384	87.56	36.00	51.56	51.6
40	54.3	0.0371	89.02	37.50	51.52	
42	52.6	0.0359	90.41	39.00	51.41	
44	50.9	0.0348	91.74	40.50	51.24	
46	49.4	0.0337	93.03	42.00	51.03	
48	48.0	0.0327	94.26	43.50	50.76	
50	46.6	0.0318	95.44	45.00	50.44	
52	45.4	0.0310	96.59	46.50	50.09	
54	44.2	0.0302	97.70	48.00	49.70	
56	43.1	0.0294	98.77	49.50	49.27	
58	42.0	0.0287	99.81	51.00	48.81	
60	41.0	0.0280	100.81	52.50	48.31	
62	40.1	0.0274	101.79	54.00	47.79	
64	39.2	0.0268	102.74	55.50	47.24	
66	38.4	0.0262	103.67	57.00	46.67	
68	37.6	0.0256	104.57	58.50	46.07	
70	36.8	0.0251	105.44	60.00	45.44	



MODIFIED RATIONAL METHOD CALCULATIONS
25-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	25-year	0.27	0.91	1.00	10	2	0.0258

NOTES: 25-year pre-development target is 0.041 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	148.2	0.1111	66.67	15.48	51.19	
12	134.7	0.1010	72.74	17.03	55.71	
14	123.8	0.0928	77.98	18.58	59.40	
16	114.7	0.0860	82.59	20.12	62.47	
18	107.0	0.0803	86.71	21.67	65.04	
20	100.5	0.0754	90.43	23.22	67.21	
22	94.8	0.0711	93.83	24.77	69.06	
24	89.8	0.0673	96.95	26.32	70.64	
26	85.3	0.0640	99.85	27.86	71.98	
28	81.4	0.0610	102.55	29.41	73.13	
30	77.8	0.0584	105.07	30.96	74.11	
32	74.6	0.0560	107.45	32.51	74.94	
34	71.7	0.0538	109.70	34.06	75.64	
36	69.0	0.0518	111.83	35.60	76.22	
38	66.6	0.0499	113.85	37.15	76.70	
40	64.3	0.0482	115.78	38.70	77.08	
42	62.2	0.0467	117.63	40.25	77.38	
44	60.3	0.0452	119.40	41.80	77.60	
46	58.5	0.0439	121.10	43.34	77.75	
48	56.8	0.0426	122.73	44.89	77.84	
50	55.2	0.0414	124.30	46.44	77.86	77.9
52	53.8	0.0403	125.82	47.99	77.83	
54	52.4	0.0393	127.29	49.54	77.75	
56	51.1	0.0383	128.71	51.08	77.63	
58	49.8	0.0374	130.09	52.63	77.46	
60	48.7	0.0365	131.42	54.18	77.24	
62	47.6	0.0357	132.72	55.73	76.99	
64	46.5	0.0349	133.98	57.28	76.71	
66	45.5	0.0341	135.21	58.82	76.38	
68	44.6	0.0334	136.40	60.37	76.03	
70	43.7	0.0328	137.57	61.92	75.65	



MODIFIED RATIONAL METHOD CALCULATIONS
50-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	50-year	0.27	0.91	1.00	10	2	0.026

NOTES: 50-year pre-development target is 0.05 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	164.2	0.1232	73.90	15.60	58.30	
12	149.2	0.1119	80.58	17.16	63.42	
14	137.1	0.1028	86.35	18.72	67.63	
16	127.0	0.0952	91.44	20.28	71.16	
18	118.5	0.0889	95.98	21.84	74.14	
20	111.2	0.0834	100.09	23.40	76.69	
22	104.9	0.0787	103.85	24.96	78.89	
24	99.4	0.0745	107.31	26.52	80.79	
26	94.5	0.0708	110.52	28.08	82.44	
28	90.1	0.0676	113.51	29.64	83.87	
30	86.2	0.0646	116.31	31.20	85.11	
32	82.6	0.0620	118.95	32.76	86.19	
34	79.4	0.0595	121.45	34.32	87.13	
36	76.4	0.0573	123.82	35.88	87.94	
38	73.7	0.0553	126.07	37.44	88.63	
40	71.2	0.0534	128.22	39.00	89.22	
42	68.9	0.0517	130.28	40.56	89.72	
44	66.8	0.0501	132.25	42.12	90.13	
46	64.8	0.0486	134.15	43.68	90.47	
48	63.0	0.0472	135.97	45.24	90.73	
50	61.2	0.0459	137.73	46.80	90.93	
52	59.6	0.0447	139.43	48.36	91.07	
54	58.1	0.0435	141.07	49.92	91.15	
56	56.6	0.0425	142.66	51.48	91.18	91.2
58	55.2	0.0414	144.20	53.04	91.16	
60	54.0	0.0405	145.70	54.60	91.10	
62	52.7	0.0396	147.15	56.16	90.99	
64	51.6	0.0387	148.56	57.72	90.84	
66	50.5	0.0379	149.94	59.28	90.66	
68	49.4	0.0371	151.28	60.84	90.44	
70	48.4	0.0363	152.59	62.40	90.19	



MODIFIED RATIONAL METHOD CALCULATIONS
100-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
A1	100-year	0.27	0.91	1.00	10	2	0.0263

NOTES: 100-year pre-development target is 0.057 m³/s

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	180.2	0.1351	81.07	15.78	65.29	
12	164.1	0.1231	88.61	17.36	71.25	
14	151.0	0.1132	95.13	18.94	76.19	
16	140.1	0.1051	100.87	20.51	80.35	
18	130.9	0.0981	106.00	22.09	83.90	
20	122.9	0.0922	110.63	23.67	86.96	
22	116.0	0.0870	114.86	25.25	89.61	
24	109.9	0.0825	118.75	26.83	91.92	
26	104.6	0.0784	122.35	28.40	93.94	
28	99.8	0.0748	125.70	29.98	95.72	
30	95.4	0.0716	128.85	31.56	97.29	
32	91.5	0.0686	131.80	33.14	98.66	
34	88.0	0.0660	134.59	34.72	99.88	
36	84.7	0.0635	137.24	36.29	100.94	
38	81.7	0.0613	139.75	37.87	101.88	
40	79.0	0.0592	142.15	39.45	102.70	
42	76.4	0.0573	144.44	41.03	103.41	
44	74.1	0.0555	146.63	42.61	104.02	
46	71.9	0.0539	148.73	44.18	104.55	
48	69.8	0.0523	150.76	45.76	105.00	
50	67.9	0.0509	152.71	47.34	105.37	
52	66.1	0.0495	154.59	48.92	105.67	
54	64.4	0.0483	156.41	50.50	105.91	
56	62.8	0.0471	158.16	52.07	106.09	
58	61.3	0.0459	159.87	53.65	106.22	
60	59.8	0.0449	161.52	55.23	106.29	
62	58.5	0.0439	163.12	56.81	106.32	106.3
64	57.2	0.0429	164.68	58.39	106.30	
66	56.0	0.0420	166.20	59.96	106.24	
68	54.8	0.0411	167.68	61.54	106.13	
70	53.7	0.0403	169.12	63.12	106.00	



MODIFIED RATIONAL METHOD CALCULATIONS
2-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	2-year	0.07	0.95	0.95	10	5	0.0012

NOTES: 2-year pre-development target is 0.0013 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	83.1	0.0154	9.21	0.72	8.49	
15	66.1	0.0122	10.99	0.90	10.09	
20	55.4	0.0102	12.28	1.08	11.20	
25	48.0	0.0089	13.29	1.26	12.03	
30	42.5	0.0078	14.13	1.44	12.69	
35	38.3	0.0071	14.84	1.62	13.22	
40	34.9	0.0064	15.46	1.80	13.66	
45	32.1	0.0059	16.01	1.98	14.03	
50	29.8	0.0055	16.50	2.16	14.34	
55	27.8	0.0051	16.95	2.34	14.61	
60	26.1	0.0048	17.37	2.52	14.85	
65	24.6	0.0046	17.75	2.70	15.05	
70	23.3	0.0043	18.11	2.88	15.23	
75	22.2	0.0041	18.45	3.06	15.39	
80	21.2	0.0039	18.77	3.24	15.53	
85	20.2	0.0037	19.07	3.42	15.65	
90	19.4	0.0036	19.35	3.60	15.75	
95	18.6	0.0034	19.62	3.78	15.84	
100	17.9	0.0033	19.88	3.96	15.92	
105	17.3	0.0032	20.13	4.14	15.99	
110	16.7	0.0031	20.36	4.32	16.04	
115	16.2	0.0030	20.59	4.50	16.09	
120	15.6	0.0029	20.81	4.68	16.13	
125	15.2	0.0028	21.02	4.86	16.16	
130	14.7	0.0027	21.23	5.04	16.19	
135	14.3	0.0026	21.42	5.22	16.20	
140	13.9	0.0026	21.62	5.40	16.22	
145	13.6	0.0025	21.80	5.58	16.22	
150	13.2	0.0024	21.98	5.76	16.22	16.2
155	12.9	0.0024	22.16	5.94	16.22	
160	12.6	0.0023	22.33	6.12	16.21	



MODIFIED RATIONAL METHOD CALCULATIONS
5-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where,

C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	5-year	0.07	0.95	0.95	10	10	0.0016

NOTES: 5-year pre-development target is 0.0017 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	108.9	0.0201	12.07	0.96	11.11	
20	73.2	0.0135	16.22	1.44	14.78	
30	56.4	0.0104	18.76	1.92	16.84	
40	46.5	0.0086	20.60	2.40	18.20	
50	39.8	0.0074	22.06	2.88	19.18	
60	35.0	0.0065	23.28	3.36	19.92	
70	31.4	0.0058	24.33	3.84	20.49	
80	28.5	0.0053	25.25	4.32	20.93	
90	26.1	0.0048	26.08	4.80	21.28	
100	24.2	0.0045	26.83	5.28	21.55	
110	22.6	0.0042	27.52	5.76	21.76	
120	21.2	0.0039	28.16	6.24	21.92	
130	20.0	0.0037	28.76	6.72	22.04	
140	18.9	0.0035	29.32	7.20	22.12	
150	18.0	0.0033	29.85	7.68	22.17	
160	17.1	0.0032	30.34	8.16	22.18	22.2
170	16.4	0.0030	30.82	8.64	22.18	
180	15.7	0.0029	31.27	9.12	22.15	
190	15.1	0.0028	31.70	9.60	22.10	
200	14.5	0.0027	32.11	10.08	22.03	
210	14.0	0.0026	32.51	10.56	21.95	
220	13.5	0.0025	32.89	11.04	21.85	
230	13.0	0.0024	33.25	11.52	21.73	
240	12.6	0.0023	33.61	12.00	21.61	
250	12.3	0.0023	33.95	12.48	21.47	
260	11.9	0.0022	34.28	12.96	21.32	
270	11.6	0.0021	34.60	13.44	21.16	
280	11.3	0.0021	34.92	13.92	21.00	
290	11.0	0.0020	35.22	14.40	20.82	
300	10.7	0.0020	35.51	14.88	20.63	
310	10.4	0.0019	35.80	15.36	20.44	



MODIFIED RATIONAL METHOD CALCULATIONS
10-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCF Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	10-year	0.07	0.95	0.95	10	10	0.0019

NOTES: 10-year pre-development target is 0.0020 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	126.5	0.0234	14.03	1.14	12.89	
20	85.3	0.0158	18.91	1.71	17.20	
30	65.9	0.0122	21.90	2.28	19.62	
40	54.3	0.0100	24.09	2.85	21.24	
50	46.6	0.0086	25.83	3.42	22.41	
60	41.0	0.0076	27.29	3.99	23.30	
70	36.8	0.0068	28.54	4.56	23.98	
80	33.4	0.0062	29.65	5.13	24.52	
90	30.7	0.0057	30.64	5.70	24.94	
100	28.5	0.0053	31.54	6.27	25.27	
110	26.6	0.0049	32.37	6.84	25.53	
120	24.9	0.0046	33.14	7.41	25.73	
130	23.5	0.0043	33.86	7.98	25.88	
140	22.3	0.0041	34.53	8.55	25.98	
150	21.2	0.0039	35.17	9.12	26.05	
160	20.2	0.0037	35.77	9.69	26.08	
170	19.3	0.0036	36.34	10.26	26.08	26.1
180	18.5	0.0034	36.88	10.83	26.05	
190	17.8	0.0033	37.40	11.40	26.00	
200	17.1	0.0032	37.90	11.97	25.93	
210	16.5	0.0030	38.38	12.54	25.84	
220	15.9	0.0029	38.84	13.11	25.73	
230	15.4	0.0028	39.28	13.68	25.60	
240	14.9	0.0028	39.71	14.25	25.46	
250	14.5	0.0027	40.13	14.82	25.31	
260	14.1	0.0026	40.53	15.39	25.14	
270	13.7	0.0025	40.92	15.96	24.96	
280	13.3	0.0025	41.29	16.53	24.76	
290	13.0	0.0024	41.66	17.10	24.56	
300	12.6	0.0023	42.02	17.67	24.35	
310	12.3	0.0023	42.37	18.24	24.13	



MODIFIED RATIONAL METHOD CALCULATIONS
25-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCF Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	25-year	0.07	0.95	1.00	10	10	0.0024

NOTES: 25-year pre-development target is 0.0034 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	148.2	0.0288	17.28	1.44	15.84	
20	100.5	0.0195	23.45	2.16	21.29	
30	77.8	0.0151	27.24	2.88	24.36	
40	64.3	0.0125	30.02	3.60	26.42	
50	55.2	0.0107	32.23	4.32	27.91	
60	48.7	0.0095	34.07	5.04	29.03	
70	43.7	0.0085	35.67	5.76	29.91	
80	39.7	0.0077	37.07	6.48	30.59	
90	36.5	0.0071	38.34	7.20	31.14	
100	33.8	0.0066	39.49	7.92	31.57	
110	31.6	0.0061	40.54	8.64	31.90	
120	29.7	0.0058	41.52	9.36	32.16	
130	28.0	0.0054	42.43	10.08	32.35	
140	26.5	0.0052	43.29	10.80	32.49	
150	25.2	0.0049	44.10	11.52	32.58	
160	24.0	0.0047	44.86	12.24	32.62	
170	23.0	0.0045	45.59	12.96	32.63	32.6
180	22.0	0.0043	46.28	13.68	32.60	
190	21.2	0.0041	46.94	14.40	32.54	
200	20.4	0.0040	47.58	15.12	32.46	
210	19.7	0.0038	48.19	15.84	32.35	
220	19.0	0.0037	48.77	16.56	32.21	
230	18.4	0.0036	49.34	17.28	32.06	
240	17.8	0.0035	49.88	18.00	31.88	
250	17.3	0.0034	50.41	18.72	31.69	
260	16.8	0.0033	50.92	19.44	31.48	
270	16.3	0.0032	51.42	20.16	31.26	
280	15.9	0.0031	51.90	20.88	31.02	
290	15.5	0.0030	52.37	21.60	30.77	
300	15.1	0.0029	52.82	22.32	30.50	
310	14.7	0.0029	53.27	23.04	30.23	



MODIFIED RATIONAL METHOD CALCULATIONS
50-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	50-year	0.07	0.95	1.00	10	10	0.0027

NOTES: 50-year pre-development target is 0.0034 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	164.2	0.0319	19.16	1.62	17.54	
20	111.2	0.0216	25.95	2.43	23.52	
30	86.2	0.0168	30.16	3.24	26.92	
40	71.2	0.0139	33.24	4.05	29.19	
50	61.2	0.0119	35.71	4.86	30.85	
60	54.0	0.0105	37.77	5.67	32.10	
70	48.4	0.0094	39.56	6.48	33.08	
80	44.1	0.0086	41.14	7.29	33.85	
90	40.5	0.0079	42.56	8.10	34.46	
100	37.6	0.0073	43.86	8.91	34.95	
110	35.1	0.0068	45.05	9.72	35.33	
120	33.0	0.0064	46.15	10.53	35.62	
130	31.1	0.0060	47.19	11.34	35.85	
140	29.5	0.0057	48.15	12.15	36.00	
150	28.0	0.0055	49.07	12.96	36.11	
160	26.8	0.0052	49.93	13.77	36.16	
170	25.6	0.0050	50.76	14.58	36.18	36.2
180	24.5	0.0048	51.54	15.39	36.15	
190	23.6	0.0046	52.29	16.20	36.09	
200	22.7	0.0044	53.02	17.01	36.01	
210	21.9	0.0043	53.71	17.82	35.89	
220	21.2	0.0041	54.37	18.63	35.74	
230	20.5	0.0040	55.02	19.44	35.58	
240	19.9	0.0039	55.64	20.25	35.39	
250	19.3	0.0037	56.24	21.06	35.18	
260	18.7	0.0036	56.82	21.87	34.95	
270	18.2	0.0035	57.39	22.68	34.71	
280	17.7	0.0034	57.93	23.49	34.44	
290	17.3	0.0034	58.47	24.30	34.17	
300	16.9	0.0033	58.99	25.11	33.88	
310	16.4	0.0032	59.49	25.92	33.57	



MODIFIED RATIONAL METHOD CALCULATIONS
100-year Design Storm Event

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

<<< Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCC Station + 15% to account for climate change

2-year		5-year		10-year		25-year		50-year		100-year	
A =	678.085	A =	853.608	A =	975.865	A =	1146.275	A =	1236.152	A =	1426.408
B =	4.699	B =	4.699	B =	4.699	B =	4.922	B =	4.699	B =	5.273
C =	0.781	C =	0.766	C =	0.760	C =	0.757	C =	0.751	C =	0.759

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = \frac{A}{(B + t_d)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
t_d = Storm Duration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Runoff Volume

$$V_{\text{Runoff}} = Q_{\text{Runoff}} \times t_d \quad (\text{m}^3)$$

where, Q_{Runoff} = Runoff Peak Flow Rate, (m³/sec)
t_d = Storm Duration, (sec)

Released Volume

$$V_{\text{Released}} = Q_{\text{Released}} \times (t_d + T_c)/2 \quad (\text{m}^3)$$

where, Q_{Released} = Max. Release Rate, (m³/sec)
t_d = Storm Duration, (sec)
T_c = Time of Concentration, (sec)

Max. Storage Required

$$V_{\text{Storage}} = V_{\text{Runoff}} - V_{\text{Released}} \quad (\text{m}^3)$$

V_{Runoff} = Runoff Volume, (m³)
V_{Released} = Released Volume, (m³)

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Catchment I.D.	Storm Event	Area A (ha)	Runoff Coeff. C	Runoff Coeff. C _{MOD}	Time of Conc. T _c (min.)	Storm Time Step (min.)	Release Rate (m ³ /s)
B2a / B2b	100-year	0.07	0.95	1.00	10	10	0.003

NOTES: 100-year pre-development target is 0.0082 m³/s for each section of roof

Storm Duration t _d (min.)	Rainfall Intensity (mm/hr)	Runoff Peak Flow Rate (m ³ /s)	Runoff Volume (m ³)	Released Volume (m ³)	Storage Volume (m ³)	Max. Storage Required (m ³)
10	180.2	0.0350	21.02	1.80	19.22	
20	122.9	0.0239	28.68	2.70	25.98	
30	95.4	0.0186	33.40	3.60	29.80	
40	79.0	0.0154	36.85	4.50	32.35	
50	67.9	0.0132	39.59	5.40	34.19	
60	59.8	0.0116	41.88	6.30	35.58	
70	53.7	0.0104	43.84	7.20	36.64	
80	48.8	0.0095	45.58	8.10	37.48	
90	44.9	0.0087	47.14	9.00	38.14	
100	41.6	0.0081	48.56	9.90	38.66	
110	38.9	0.0076	49.86	10.80	39.06	
120	36.5	0.0071	51.06	11.70	39.36	
130	34.4	0.0067	52.18	12.60	39.58	
140	32.6	0.0063	53.24	13.50	39.74	
150	31.0	0.0060	54.23	14.40	39.83	
160	29.6	0.0057	55.17	15.30	39.87	39.9
170	28.3	0.0055	56.06	16.20	39.86	
180	27.1	0.0053	56.91	17.10	39.81	
190	26.0	0.0051	57.72	18.00	39.72	
200	25.1	0.0049	58.50	18.90	39.60	
210	24.2	0.0047	59.25	19.80	39.45	
220	23.4	0.0045	59.97	20.70	39.27	
230	22.6	0.0044	60.66	21.60	39.06	
240	21.9	0.0043	61.33	22.50	38.83	
250	21.2	0.0041	61.97	23.40	38.57	
260	20.6	0.0040	62.60	24.30	38.30	
270	20.1	0.0039	63.21	25.20	38.01	
280	19.5	0.0038	63.80	26.10	37.70	
290	19.0	0.0037	64.37	27.00	37.37	
300	18.6	0.0036	64.93	27.90	37.03	
310	18.1	0.0035	65.47	28.80	36.67	



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RATIONAL METHOD CALCULATIONS

Date: 10-Mar-23
Project: 642 Dunlop St. West

Project No.: 21-710
Prepared By: JR

Elements Requiring Input Information

Rainfall Intensity-Duration-Frequency Coefficients from: Barrie WPCC Station + 15% to account for climate change

2-year	5-year	10-year	25-year	50-year	100-year
A = 678.085	A = 853.608	A = 975.865	A = 1146.275	A = 1236.152	A = 1426.408
B = 4.699	B = 4.699	B = 4.699	B = 4.922	B = 4.699	B = 5.273
C = 0.781	C = 0.766	C = 0.760	C = 0.757	C = 0.751	C = 0.759

$$Q = \frac{C \times I \times A}{360} \quad (\text{m}^3/\text{s})$$

where, C = Runoff Coefficient
I = Rainfall Intensity, (mm/hr)
A = Drainage Area, (ha)

$$I_{2-100} = \frac{A}{(B + T_c)^C} \quad (\text{mm/hr})$$

where, A = Rainfall IDF Coefficient
B = Rainfall IDF Coefficient
C = Rainfall IDF Coefficient
T_c = Time of Concentration, (min)

Runoff Coefficient Equations
Based on MTO Drainage Manual (1984), page BD-4

2-year C₂ = C
5-year C₅ = C
10-year C₁₀ = C
25-year C₂₅ = 1.10 x C
50-year C₅₀ = 1.20 x C
100-year C₁₀₀ = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as indicated above, up to a maximum value of 1.

Rainfall Intensity Equation (25mm storm event)
Based on the MOE SWMP Manual (2003), Eq'n 4.9

$$I_{25\text{mm}} = (43 \times C) + 5.9 \quad (\text{mm/hr})$$

where, C = Runoff Coefficient

Catchment I.D.	A (ha)	T _c (min.)	C	Q _{25mm} (m ³ /s)	Q ₂ (m ³ /s)	Q ₅ (m ³ /s)	Q ₁₀ (m ³ /s)	Q ₂₅ (m ³ /s)	Q ₅₀ (m ³ /s)	Q ₁₀₀ (m ³ /s)
PRE-W	0.60	10.0	0.26	0.007	0.036	0.047	0.055	0.071	0.085	0.098
B1	0.25	10.0	0.20	0.002	0.012	0.015	0.018	0.023	0.027	0.031
B2a	0.07	10.0	0.95	0.009	0.015	0.020	0.023	0.029	0.032	0.035
B2b	0.07	10.0	0.95	0.009	0.015	0.020	0.023	0.029	0.032	0.035
B2c	0.10	10.0	0.95	0.012	0.022	0.029	0.033	0.041	0.046	0.050
B2 total	0.24	10.0	0.95	0.030	0.053	0.069	0.080	0.099	0.109	0.120
B1+B2	0.32	10.0	0.87	0.033	0.064	0.084	0.098	0.126	0.146	0.160
PRE-E	0.21	10.0	0.52	0.009	0.025	0.033	0.038	0.049	0.060	0.068
A1	0.27	10.0	0.91	0.031	0.057	0.074	0.086	0.111	0.123	0.135
A2	0.05	10.0	0.36	0.001	0.004	0.005	0.006	0.008	0.010	0.011
A1+A2	0.32	10.0	0.82	0.030	0.061	0.080	0.093	0.119	0.144	0.160

Z:\Projects\2021\21-710\Design\Storm\230310_Rational_Method_Calcs(A,B,C).xlsx\Rational Method

Allowable Release Rates:

Parking Lot:

A1 allowable = PRE-E - A2(uncontrolled) 0.021 0.028 0.032 0.041 0.050 0.057

Building:

B2b+B2c allowable = PRE-W - B1(uncontrolled) - B2a(uncontrolled) 0.0025 0.0033 0.0039 0.0068 0.0124 0.0163

B2b / B2c allowable (L/s each) 1.3 1.7 1.9 3.4 6.2 8.1

Project: 642 Dunlop St. W



Chamber Model -
Units -

SC-160	
Metric	Click Here for Imperial

Number of chambers -
Voids in the stone (porosity) -

87	
40	%

Base of Stone Elevation -
Amount of Stone Above Chambers -
Amount of Stone Below Chambers -

234.37	m
152	mm
152	mm

☒ Include Perimeter Stone in Calculations

150.93 sq.meters Min. Area - 119.882 sq.meters

StormTech SC-160 Cumulative Storage Volumes

Height of System (mm)	Incremental Single Chamber (cubic meters)	Incremental Total Chamber (cubic meters)	Incremental Stone (cubic meters)	Incremental Ch & St (cubic meters)	Cumulative Chamber (cubic meters)	Elevation (meters)
610	0.00	0.00	1.53	1.53	46.915	234.98
584	0.00	0.00	1.53	1.53	45.382	234.96
559	0.00	0.00	1.53	1.53	43.850	234.93
533	0.00	0.00	1.53	1.53	42.317	234.91
508	0.00	0.00	1.53	1.53	40.784	234.88
483	0.00	0.00	1.53	1.53	39.252	234.86
457	0.00	0.13	1.48	1.61	37.719	234.83
432	0.00	0.33	1.40	1.73	36.110	234.81
406	0.01	0.72	1.25	1.96	34.379	234.78
381	0.01	1.09	1.10	2.19	32.417	234.76
356	0.02	1.33	1.00	2.33	30.231	234.73
330	0.02	1.52	0.93	2.44	27.901	234.70
305	0.02	1.68	0.86	2.54	25.458	234.68
279	0.02	1.81	0.81	2.62	22.919	234.65
254	0.02	1.93	0.76	2.69	20.298	234.63
229	0.02	2.03	0.72	2.75	17.608	234.60
203	0.02	2.12	0.68	2.80	14.856	234.58
178	0.03	2.20	0.65	2.86	12.051	234.55
152	0.00	0.00	1.53	1.53	9.196	234.53
127	0.00	0.00	1.53	1.53	7.663	234.50
102	0.00	0.00	1.53	1.53	6.131	234.48
76	0.00	0.00	1.53	1.53	4.598	234.45
51	0.00	0.00	1.53	1.53	3.065	234.42
25	0.00	0.00	1.53	1.53	1.533	234.40



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STAGE-STORAGE CALCULATION SUMMARY

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop Street West

Prepared By: JR

Stage Notes:	Elevation	250mm Pipe Volume (68m)	300mm Pipe Volume (37m)	Stormtech Storage	5x [CBMH or MH] Storage Vol.	2x CB Storage Vol.	Surface Ponding Storage Volume	Cumulative Storage Volume
	(m)	(m ³)	(m ³)		(m ³)	(m ³)	(m ³)	(m ³)
Orifice invert 234.55	234.55	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	234.60	0.4	0.3	5.6	0.0	0.0	0.0	6.3
	234.65	0.8	0.6	10.9	0.4	0.0	0.0	12.7
	234.70	1.3	0.9	15.9	0.9	0.0	0.0	18.9
	234.75	1.7	1.2	20.4	1.3	0.0	0.0	24.6
	234.80	2.1	1.5	24.1	1.8	0.1	0.0	29.4
	234.85	2.5	1.7	27.2	2.2	0.1	0.0	33.8
	234.90	2.9	2.0	30.3	2.7	0.1	0.0	38.0
OBV of upstream-most 250mm pipe top of Stormtech storage	234.95	3.3	2.3	33.3	3.1	0.2	0.0	42.3
	235.00	3.3	2.6	34.9	3.5	0.2	0.0	44.6
	235.05	3.3	3.3	34.9	4.0	0.3	0.0	45.8
	235.10	3.3	3.3	34.9	4.4	0.3	0.0	46.2
top of CBMH & MH storage	235.15	3.3	3.3	34.9	4.9	0.3	0.0	46.7
	235.20	3.3	3.3	34.9	4.9	0.4	0.0	46.8
	235.25	3.3	3.3	34.9	4.9	0.4	0.0	46.8
	235.30	3.3	3.3	34.9	4.9	0.4	0.0	46.8
	235.35	3.3	3.3	34.9	4.9	0.5	0.0	46.9
	235.40	3.3	3.3	34.9	4.9	0.5	0.0	46.9
	235.45	3.3	3.3	34.9	4.9	0.5	0.0	46.9
	235.50	3.3	3.3	34.9	4.9	0.6	0.0	47.0
	235.55	3.3	3.3	34.9	4.9	0.6	0.0	47.0
	235.60	3.3	3.3	34.9	4.9	0.6	0.0	47.0
	235.65	3.3	3.3	34.9	4.9	0.7	0.0	47.1
	235.70	3.3	3.3	34.9	4.9	0.7	0.0	47.1
top of CB storage + start of ponding	235.75	3.3	3.3	34.9	4.9	0.8	0.0	47.2
	235.80	3.3	3.3	34.9	4.9	0.8	3.6	50.7
	235.85	3.3	3.3	34.9	4.9	0.8	10.7	57.9
	235.90	3.3	3.3	34.9	4.9	0.8	40.5	87.7
curb weir crest = 235.95	235.95	3.3	3.3	34.9	4.9	0.8	81.2	128.4
driveway weir crest = 236.00	236.00	3.3	3.3	34.9	4.9	0.8	162.7	209.9
	236.05	3.3	3.3	34.9	4.9	0.8	280.9	328.1

Notes:

- Top of effective storage' is the cavity of the structure up to approx. 0.6m below T/G of manholes to account for CB grates, moduloc and flat cap thicknesses, and 0.3m below CB grates to account for CB grates and moduloc thicknesses
- refer to Stormtech Cumulative Stage Storage sheet for Stormtech details. The storage volumes listed discounts the 12.05m3 of cumulative storage below the invert of the outlet pipe

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STAGE-STORAGE-DISCHARGE (S-S-D) CALCULATIONS
PARKING LOT SWM FACILITY

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop Street West

Prepared By: JR

<<<				Elements Requiring Input Information
Unsubmerged Orifice (Weir Flow)		Submerged Orifice (Orifice Flow)		Unsubmerged Weir (Weir Flow)
$Q = C_w L H^{3/2} \quad (m^3/s)$		$Q = C_o A_o (2gH)^{1/2} \quad (m^3/s)$		Rectangular Broad- & Sharp-Crested Weirs $Q = C_w L H^{3/2} \quad (m^3/s)$
where, Q = Flow through unsubmerged orifice (m^3/s)		where, Q = Flow through submerged orifice (m^3/s)		Triangular Broad-Crested Weirs $Q = 1.225 H^{5/2} \tan(\Theta/2) \quad (m^3/s)$
C_w = Weir Coefficient		C_o = Orifice Discharge Coefficient		Triangular Sharp-Crested Weirs $Q = 0.581 (8/15) (2g)^{1/2} \tan(\Theta/2) H^{3/2} \quad (m^3/s)$
H = Head/Depth of water acting on weir measured from above the crest/invert of orifice (m)		A_o = Cross-sectional area of orifice (m^2)		Trapezoidal Broad- & Sharp-Crested Weirs $Q_{TRAPEZOIDAL} = Q_{RECTANGULAR} + Q_{TRIANGULAR} \quad (m^3/s)$
L = Length of weir (m)		g = Gravitational acceleration ($9.81 m^2/s$)		
D = Diameter of Pipe/Orifice (m)		For circular vertical orifice, H = Head/Depth of water acting on orifice measured from centroid of the opening (m)		where, Q = Flow through unsubmerged weir (m^3/s)
For circular vertical weir, L = Wetted Perimeter $L = D \times \cos^{-1}((D/2 - H)/(D/2))$		For circular horizontal orifice, H = Head/Depth of water acting on orifice measured from above the invert (m)		C_w = Weir Coefficient (1.65 for Broad-Crested) (1.80 for Sharp-Crested)
For circular horizontal weir, L = Circumference $L = 3.14 \times D$				H = Head/Depth of water acting on weir measured from above the crest (m)
				L = Length of weir measured perpendicular to flow direction (m)
				$\Theta/2$ = Angle of side slope measured from vertical axis (degrees)
				g = Gravitational acceleration ($9.81 m^2/s$)

NOTES: **Orifice Flow Notes**
- **Vertical Orifice Flow** calculations assume weir flow up to the centroid/center of orifice and then orifice flow above the crown/top of the orifice. Between the centroid and crown of the orifice is a flow transition stage from weir to orifice flow and is calculated based on a linear interpolation between the known weir flow at the centroid of the orifice and the known orifice flow at the crown.
- **Horizontal Orifice Flow** calculations assume weir flow up to one-quarter of the orifices diameter (0.25xD) and then orifice flow above three-quarters of the orifices diameter (0.75xD). Between (0.25xD) and (0.75xD) exists a flow transition stage which is calculated based on a linear interpolation between the known weir flow at (0.25xD) and the known orifice flow at (0.75xD).
Weir Flow Notes
- **Orifice control** is only applicable if the weir opening is submerged and not exposed to atmospheric pressure for all ranges of water elevations.
- For all Weir Types, **orifice control** occurs when the water surface elevation is equal to or greater than the crown/top of the opening.

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Starting Water Elevation, m =	234.55
Incremental Depth, m =	0.050

Orifice 1	Orifice 2	Orifice 3	Weir 1	Weir 2	Weir 3
Vertical			Rectangular Broad-Crested	Rectangular Broad-Crested	
234.55			235.95	236.00	
0.05	0.05	0.05	0.05	0.05	0.05
234.55			2.00	6.50	
0.102			1.65	1.65	
234.601					
0.0082					
0.63					
1.80					

Orifice Type =
Orifice Invert Elev., m =
Incremental Depth, m =
Water Elev. @ Inflow, m =
Orifice Diameter, m =
Centroid of Orifice, m =
Orifice Area, m² =
Orifice Coefficient =
Weir Coefficient =

= Weir Type
= Weir Crest Elev., m
= Incremental Depth, m
= Weir Openings Crown Elev., m (if appl.)
= Weir Length, m
= Weir Coefficient
= Side Slope (H:1)
= Theta/2, Degrees
= Centroid of Orifice, m (if appl.)
= Orifice Area, m² (if appl.)
= Orifice Coefficient (if appl.)

NOTES: emergency overland outlet via a weir cut-out in the curb adjacent to the eastern entrance. 2nd emergency overland outlet via the south-east driveway onto Dunlop Street West

*See Stage-Storage Calculation Summary Sheet										Notes
Description	Elevation (m)	Orifice 1 Flow (m ³ /s)	Orifice 2 Flow (m ³ /s)	Orifice 3 Flow (m ³ /s)	Weir 1 Flow (m ³ /s)	Weir 2 Flow (m ³ /s)	Weir 3 Flow (m ³ /s)	Total Flow (m ³ /s)	Total Storage Volume* (m ³)	
Base	234.55	0.0000						0.0000	0.0	2-year storm (Q=0.0125m3/s, V=38.5m3 at 234.90m)
	234.60	0.0032						0.0032	6.3	
	234.65	0.0052						0.0052	12.7	
	234.70	0.0072						0.0072	18.9	
	234.75	0.0088						0.0088	24.6	
	234.80	0.0102						0.0102	29.4	
	234.85	0.0114						0.0114	33.8	
	234.90	0.0125						0.0125	38.0	
	234.95	0.0135						0.0135	42.3	
	235.00	0.0144						0.0144	44.6	
	235.05	0.0153						0.0153	45.8	
	235.10	0.0161						0.0161	46.2	
	235.15	0.0169						0.0169	46.7	
	235.20	0.0176						0.0176	46.8	
	235.25	0.0184						0.0184	46.8	
	235.30	0.0191						0.0191	46.8	
T/G of CBs	235.35	0.0197						0.0197	46.9	5-year storm (Q=0.0196m3/s, V=46.7m3 at 235.35m)
	235.40	0.0204						0.0204	46.9	
	235.45	0.0210						0.0210	46.9	
	235.50	0.0216						0.0216	47.0	
	235.55	0.0222						0.0222	47.0	
	235.60	0.0228						0.0228	47.0	
	235.65	0.0234						0.0234	47.1	
	235.70	0.0239						0.0239	47.1	
	235.75	0.0244						0.0244	47.2	
	235.80	0.0250						0.0250	50.7	
Top	235.85	0.0255						0.0255	57.9	10-year storm (Q=0.025m3/s, V=51.6m3 at 235.81m)
	235.90	0.0260						0.0260	87.7	25-year storm (Q=0.0258m3/s, V=77.9m3 at 235.87m)
	235.95	0.0265			0.000			0.0265	128.4	50-year storm (Q=0.0260m3/s, V=91.2.0m3 at 235.90m)
	236.00	0.0270			0.037	0.000		0.0639	209.9	100-year storm (Q=0.0263m3/s, V=106.3m3 at 235.93m)
	236.05	0.0274			0.104	0.120		0.2517	328.1	



WMI & Associates Limited
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p (705) 797-2027 f (705) 797-2028

STORMWATER MANAGEMENT CALCULATIONS 2 YR STORM- ROOFTOP FLOW CONTROL

Date: 10-Mar-23
Project: 642 Dunlop St. West

Project No.: 21-710
Prepared By: JR

Rooftop Flow Control Design Calculations:

- NOTES:**
- The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).
 - On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.
 - The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.
 - As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.
 - The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.
 - To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<< Elements Requiring Input Information
Rooftop Area	=	700 m ²	
# of Roof Drains	=	2 (see notes provided above)	
Max. Rooftop Release Rate	=	1.2 L/s	
Max. Storage Volume	=	16.2 m ³	(based on attenuating the 2 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.023 m (see notes provided above)	
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)	
	=	0.29 L/s per notch	
# of Notches Required	=	4.2 notches are required to convey 1.2 L/s from the rooftop based on the maximum head/depth of storage available	
	=	4.0	
# of Notches per Drain	=	2.0 OR	
Draw Down Time	=	20.0 hr	Sufficient Design



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**STORMWATER MANAGEMENT CALCULATIONS
5 YR STORM- ROOFTOP FLOW CONTROL**

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

Rooftop Flow Control Design Calculations:

NOTES: -The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).

- On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.

- The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.

- As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.

- The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.

- To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<<	Elements Requiring Input Information
Rooftop Area	=	700 m ²		
# of Roof Drains	=	2 (see notes provided above)		
Max. Rooftop Release Rate	=	1.6 L/s		
Max. Storage Volume	=	22.2 m ³		(based on attenuating the 5 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.032 m (see notes provided above)		
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)		
	=	0.40 L/s per notch		
# of Notches Required	=	4.0 notches are required to convey 1.6 L/s from the rooftop based on the maximum head/depth of storage available		
	=	4.0		
# of Notches per Drain	=	2.0	OR	
Draw Down Time	=	21.3 hr		Sufficient Design



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STORMWATER MANAGEMENT CALCULATIONS 10 YR STORM- ROOFTOP FLOW CONTROL

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

Rooftop Flow Control Design Calculations:

- NOTES:**
- The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).
 - On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.
 - The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.
 - As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.
 - The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.
 - To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<< Elements Requiring Input Information
Rooftop Area	=	700 m ²	
# of Roof Drains	=	2 (see notes provided above)	
Max. Rooftop Release Rate	=	1.9 L/s	
Max. Storage Volume	=	26.1 m ³	(based on attenuating the 10 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.037 m (see notes provided above)	
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)	
	=	0.46 L/s per notch	
# of Notches Required	=	4.1 notches are required to convey 1.9 L/s from the rooftop based on the maximum head/depth of storage available	
	=	4.0	
# of Notches per Drain	=	2.0 OR	
Draw Down Time	=	21.9 hr	Sufficient Design



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STORMWATER MANAGEMENT CALCULATIONS
25 YR STORM- ROOFTOP FLOW CONTROL

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop St. West

Prepared By: JR

Rooftop Flow Control Design Calculations:

- NOTES:**
- The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).
 - On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.
 - The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.
 - As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.
 - The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.
 - To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<<	Elements Requiring Input Information
Rooftop Area	=	700 m ²		
# of Roof Drains	=	2 (see notes provided above)		
Max. Rooftop Release Rate	=	2.4 L/s		
Max. Storage Volume	=	32.6 m ³		(based on attenuating the 25 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.047 m (see notes provided above)		
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)		
	=	0.58 L/s per notch		
# of Notches Required	=	4.1 notches are required to convey 2.4 L/s from the rooftop based on the maximum head/depth of storage available		
	=	4.0		
# of Notches per Drain	=	2.0	OR	
Draw Down Time	=	22.8 hr		Sufficient Design



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STORMWATER MANAGEMENT CALCULATIONS 50 YR STORM- ROOFTOP FLOW CONTROL

Date: 10-Mar-23 Project No.: 21-710
Project: 642 Dunlop St. West Prepared By: JR

Rooftop Flow Control Design Calculations:

- NOTES:**
- The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).
 - On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.
 - The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.
 - As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.
 - The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.
 - To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<<	Elements Requiring Input Information
Rooftop Area	=	700 m ²		
# of Roof Drains	=	2 (see notes provided above)		
Max. Rooftop Release Rate	=	2.7 L/s		
Max. Storage Volume	=	36.2 m ³		(based on attenuating the 50 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.052 m (see notes provided above)		
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)		
	=	0.64 L/s per notch		
# of Notches Required	=	4.2 notches are required to convey 2.7 L/s from the rooftop based on the maximum head/depth of storage available		
	=	4.0		
# of Notches per Drain	=	2.0 OR		
Draw Down Time	=	23.2 hr		Sufficient Design



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STORMWATER MANAGEMENT CALCULATIONS 100 YR STORM- ROOFTOP FLOW CONTROL

Date: 10-Mar-23
Project: 642 Dunlop St. West

Project No.: 21-710
Prepared By: JR

Rooftop Flow Control Design Calculations:

- NOTES:**
- The following calculations are based on the use of **Zurn Control-Flo Roof Drains (Z-105)** which can be specified to consist of between **1-6 possible weirs/notches per drain**. As noted by Zurn Industries Ltd., each weir/notch is designed to provide a **linear flow-depth relationship** and is rated at a **5 GPM per inch of head per notch** flow rating (**12.4 L/s per meter of head per notch**).
 - On **Dead-Level Roofs**, drains should be located no further than **15m from the edge of the roof** and no further than **30m from adjacent drains**. The number of drains proposed to control the rooftop stormwater runoff is determined based on the size of the roof area being controlled and the maximum allowable spacing noted above and required by the 2006 Ontario Building Code.
 - The Draw Down Time associated with the Max. Storage Volume provided on the rooftop is calculated based on a **linear flow-depth relationship** (i.e. release rate decreases linearly with the depth of water). The **maximum allowable draw down time** for stormwater runoff attenuated on a rooftop is **24 hours** based on the 2006 Ontario Building Code.
 - As required by the 2006 Ontario Building Code, **one (1) or more overflow scupper drains are required** to be installed on a flat roof designed to provide stormwater runoff attenuation so that the **maximum depth of water on the roof cannot exceed 150mm**.
 - The **Storage Depth** and associated **Draw Down Time** is based on vertical parapet walls along the perimeter of the roof (i.e. **volume equals depth multiplied by rooftop area**). As a result, by assuming a **constant rooftop area**, the **flow-storage relationship is linear similar to the flow-depth relationship when using the Zurn Control-Flo Drains (Z-105)**.
 - To utilize the **maximum rooftop storage potential** for a particular building, the Max. Rooftop Release Rate and associated Max. Storage Volume should produce as close to the maximum allowable draw down time of **24 hours as possible**.

			<<<	Elements Requiring Input Information
Rooftop Area	=	700 m ²		
# of Roof Drains	=	2 (see notes provided above)		
Max. Rooftop Release Rate	=	3 L/s		
Max. Storage Volume	=	39.9 m ³		(based on attenuating the 100 -year storm rooftop runoff and releasing it at the Max. Rooftop Release Rate (listed above))
Max. Storage Depth	=	0.057 m (see notes provided above)		
Release Rate per Notch	=	12.4 L/s per meter of head per notch (see notes provided above)		
	=	0.71 L/s per notch		
# of Notches Required	=	4.2 notches are required to convey 3 L/s from the rooftop based on the maximum head/depth of storage available		
	=	4.0		
# of Notches per Drain	=	2.0 OR		
Draw Down Time	=	23.5 hr		Sufficient Design



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MANNING'S PIPE EQUATION
STORM SEWER PIPE DESIGN- INLET TO STORMTECH

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop Street West

Prepared By: JR

Mannings' Equation			Box Culvert	
Pipe Diameter (mm) =	300		Height =	0.00
Area =	0.071	m ²	Width =	0
Slope (%) =	0.60		Q =	0.00
Manning 'n' =	0.013		Velocity	
Nom'l Pipe Capacity, Q =	0.075	m ³ /sec	1.06	m/s
Actual Pipe Capacity, Q =	0.078	m ³ /sec	1.11	m/s
			R=	0.075
			Sf=	0.0060

NOTE: This pipe is to convey the entire catchment A1 (uncontrolled) 5-year flow.



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MANNING'S PIPE EQUATION
STORM SEWER PIPE DESIGN- OUTLET FROM STORMTECH

Date: 10-Mar-23

Project No.: 21-710

Project: 642 Dunlop Street West

Prepared By: JR

Mannings' Equation			Box Culvert	
Pipe Diameter (mm) =	300		Height =	0.00
Area =	0.071	m ²	Width =	0
Slope (%) =	0.70		Q =	0.00
Manning 'n' =	0.013		Velocity	
Nom'l Pipe Capacity, Q =	0.081	m ³ /sec	1.14	m/s
Actual Pipe Capacity, Q =	0.084	m ³ /sec	1.19	m/s
			R=	0.075
			Sf=	0.0070

NOTE: This pipe is to convey the entire catchment A1 (controlled) flow of 0.0265 m3/s.



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MANNING'S PIPE EQUATION
STORM SEWER PIPE DESIGN- ROOF OUTLET TO SWM BASIN

Date: 10-Mar-23

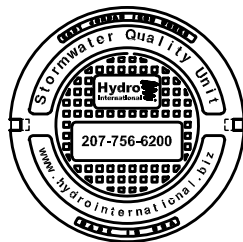
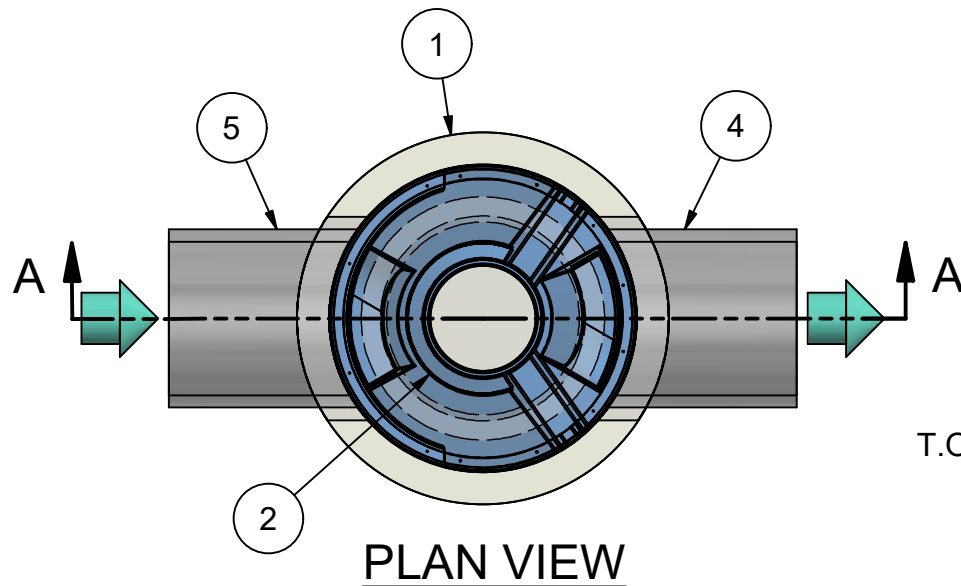
Project No.: 21-710

Project: 642 Dunlop Street West

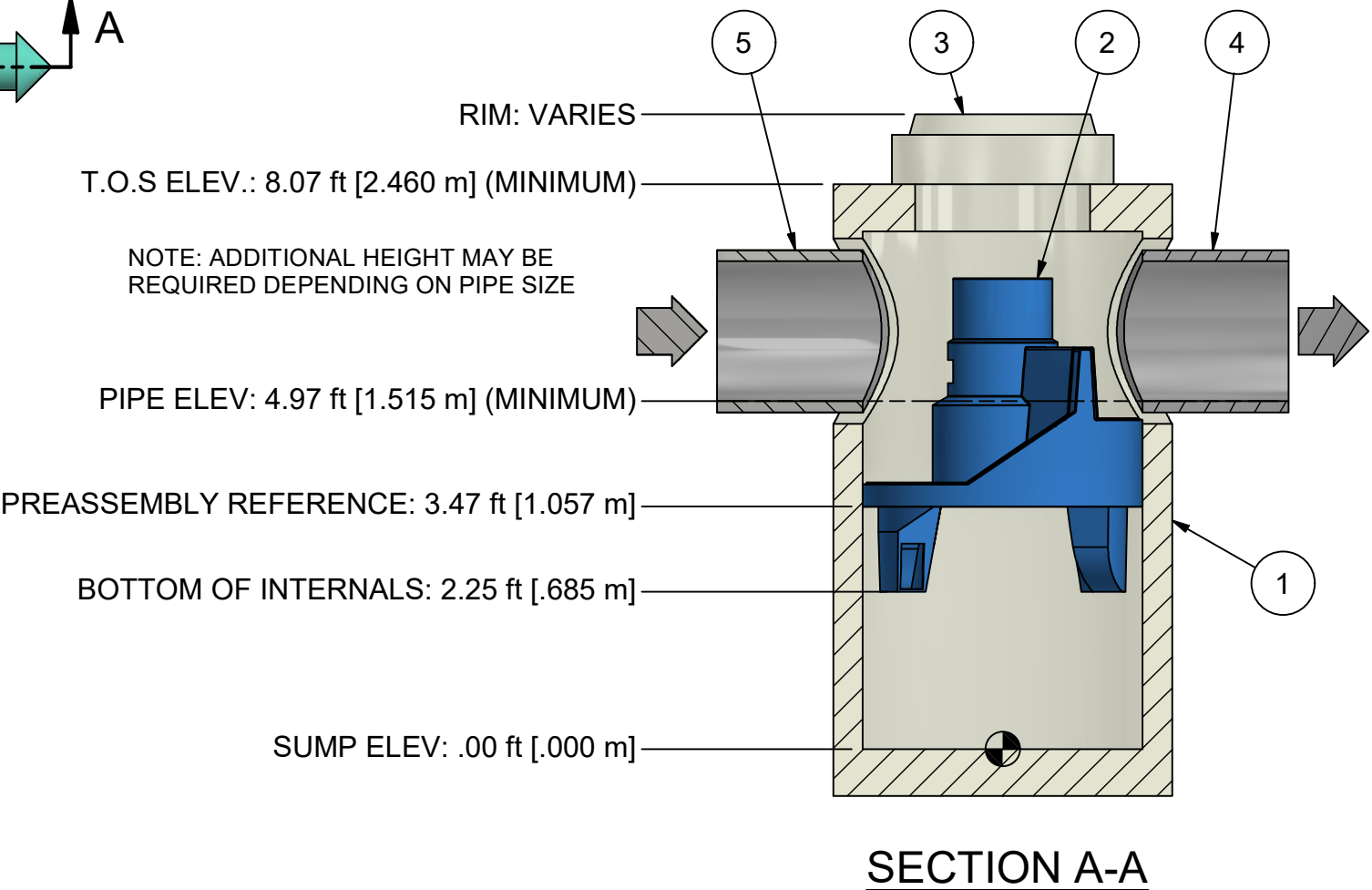
Prepared By: JR

Mannings' Equation			Box Culvert	
Pipe Diameter (mm) =	300		Height =	0.00
Area =	0.071	m ²	Width =	0
Slope (%) =	0.32		Q =	0.00
Manning 'n' =	0.013		Velocity	
Nom'l Pipe Capacity, Q =	0.055	m ³ /sec	0.77	m/s
Actual Pipe Capacity, Q =	0.057	m ³ /sec	0.81	m/s
			R=	0.075
			Sf=	0.0032

NOTE: This pipe is to convey the catchment B2c (uncontrolled) 100-yr flow of 0.050m³/s, and the 100-year controlled flow of 0.003m³/s x 2 from catchments B2a + B2b. Total flow = of 0.056m³/s



HYDRO FRAME AND COVER (INCLUDED)
GRADE RINGS BY OTHERS
AS REQUIRED



1. MANHOLE WALL AND SLAB THICKNESSES ARE NOT TO SCALE.
2. CONTACT HYDRO INTERNATIONAL FOR A BOTTOM OF STRUCTURE ELEVATION PRIOR TO SETTING FIRST DEFENSE MANHOLE.
3. CONTRACTOR TO CONFIRM RIM, PIPE INVERTS, PIPE DIA. AND PIPE ORIENTATION PRIOR TO RELEASE OF UNIT TO FABRICATION.



IF IN DOUBT ASK

DATE: 11/2/2021		SCALE: 1:30
DRAWN BY: ER	CHECKED BY: MRJ	APPROVED BY:

Title
4-ft DIAMETER
FIRST DEFENSE

GENERAL ARRANGEMENT

Hydro International
hydro-int.com
HYDRO INTERNATIONAL

WEIGHT:	MATERIAL:
---------	-----------

STOCK NUMBER:

DRAWING NO.:
FD GA-4

SHEET SIZE: B	SHEET: 1 OF 1	Rev: -
------------------	------------------	-----------

PRODUCT SPECIFICATION:

1. Peak Hydraulic Flow: 18.0 cfs (510 l/s)
2. Min Sediment Storage Capacity: 0.7 cu. yd. (0.5 cu. m.)
3. Maximum Inlet/Outlet Pipe Diameters: 24 in. (600 mm)
4. The Treatment System Shall Use An Induced Vortex To Separate Pollutants From Stormwater Runoff.
5. For More Product Information Including Regulatory Acceptances, Please Visit <https://hydro-int.com/en/products/first-defense>

GENERAL NOTES:

1. General Arrangement drawings only. Contact Hydro International for site specific drawings.
2. The diameter of the inlet and outlet pipes may be no more than 24".
3. Multiple inlet pipes possible (refer to project plan).
4. Inlet/outlet pipe angle can vary to align with drainage network (refer to project plans).
5. Peak flow rate and minimum height limited by available cover and pipe diameter.
6. Larger sediment storage capacity may be provided with a deeper sump depth.

PARTS LIST				
ITEM	QTY	SIZE (in)	SIZE (mm)	DESCRIPTION
1	1	48	1200	I.D. PRECAST MANHOLE
2	1			INTERNAL COMPONENTS (PRE-INSTALLED)
3	1	30	750	FRAME AND COVER (ROUND)
4	1	24 (MAX)	600 (MAX)	OUTLET PIPE (BY OTHERS)
5	1	24 (MAX)	600 (MAX)	INLET PIPE (BY OTHERS)



ADS Treatment Train Sizing

Project Name:	642 Dunlop St. W
Consulting Engineer:	WMI Engineering
Location:	Barrie, ON
Sizing Completed By:	C. Neath
Email:	cody.neath@adspipe.com

Summary of Results	
Isolator Row PLUS TSS Removal:	81.0%
FD-4HC TSS Removal:	57.0%
Combined TSS Removal:	91.7%
Total Volume Treated:	99.9%

Individual OGS Results		
Model	TSS Removal	Volume Treated
FD-4HC	57.0%	>90%
FD-5HC	60.0%	>90%
FD-6HC	62.0%	>90%
FD-8HC	67.0%	>90%
FD-10HC	70.0%	>90%

Overall System Capacities	
Total Sediment Storage Capacity:	2.18 m ³
Oil Storage Capacity:	723 L
Max. OGS Pipe Diameter:	600 mm
Peak OGS Flow Capacity:	510 L/s
Peak Stormtech Inlet Flow Capacity:	311 L/s
Peak IR PLUS Water Quality Flow:	50.2 L/s

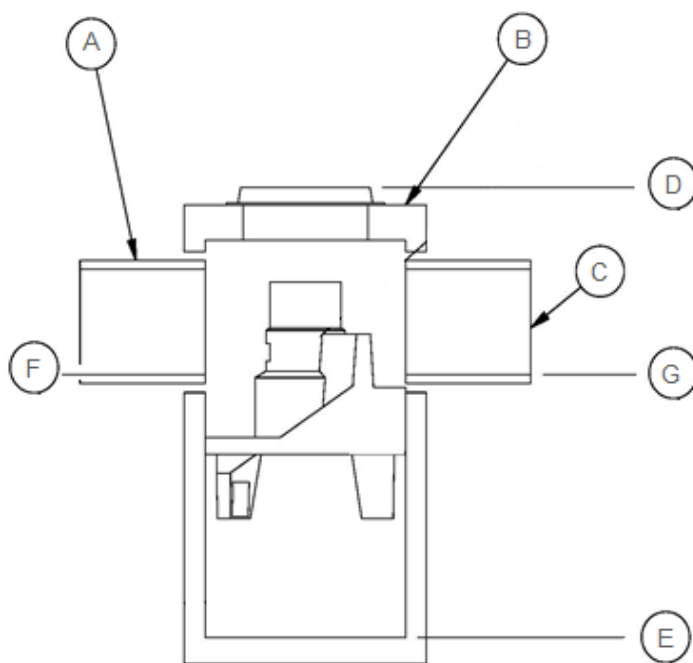
OGS Specifications	
Inlet Pipe Diameter (A):	300 mm
Unit Diameter (B):	1,200 mm
Outlet Pipe Diameter (C):	300 mm
Rim Elevation (D):	235.75 m
Bottom of Sump Elevation (E):	233.12 m
Inlet Pipe Elevation (F):	234.63 m
Outlet Pipe Elevation (G):	234.62 m

Site Details	
Site Area (ha):	0.27
Rational C:	0.91
Particle Size Distribution:	ETV
Rainfall Station:	Barrie, ONT

Notes: OGS results based on ETV PSD and results from ETV testing protocols.

Stormtech Details	
Chamber Model:	SC-160
No. Chambers in Isolator Row PLUS:	17
Volume Treated by Isolator Row PLUS:	99.8%

Notes: Refer to Stormtech drawings for full IR+ configuration.



Notes:

Isolator Row PLUS removal efficiency based on verified ETV test report. For dimensions and configuration of Isolator Row PLUS, please see Stormtech drawing package.



Project Name: 642 Dunlop St. W
 Consulting Engineer: WMI Engineering
 Location: Barrie, ON

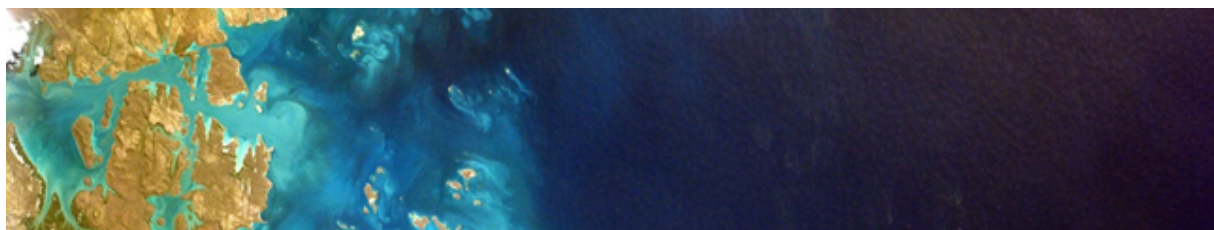
Net Annual Removal Efficiency Summary

Rainfall Intensity	Fraction of Rainfall	Removal Efficiency		Combined Removal Efficiency	Combined Weighted Removal Efficiency
		FD-4HC	IR PLUS ⁽²⁾		
mm/hr	%	%	%	%	%
0.50	0.3%	70.6%	81.2%	94.5%	0.3%
1.00	25.7%	65.6%	81.2%	93.5%	24.1%
1.50	5.3%	62.7%	81.2%	93.0%	5.0%
2.00	13.4%	60.6%	81.2%	92.6%	12.4%
2.50	5.5%	59.0%	81.2%	92.3%	5.1%
3.00	3.7%	57.7%	81.2%	92.0%	3.4%
3.50	7.2%	56.6%	81.2%	91.8%	6.6%
4.00	3.4%	55.6%	81.2%	91.7%	3.1%
4.50	2.4%	54.8%	81.2%	91.5%	2.2%
5.00	4.3%	54.0%	81.2%	91.4%	4.0%
6.00	3.6%	52.7%	81.2%	91.1%	3.3%
7.00	4.3%	51.6%	81.2%	90.9%	3.9%
8.00	3.4%	50.6%	81.2%	90.7%	3.1%
9.00	1.6%	49.8%	81.2%	90.6%	1.5%
10.00	2.1%	49.0%	81.2%	90.4%	1.9%
20.00	8.9%	44.0%	81.2%	89.5%	8.0%
30.00	2.3%	41.1%	81.2%	88.9%	2.0%
40.00	1.0%	39.0%	81.2%	88.5%	0.9%
50.00	0.5%	37.4%	81.2%	88.2%	0.5%
100.00	0.7%	0.0%	59.7%	59.7%	0.4%
150.00	0.1%	0.0%	39.8%	39.8%	0.0%
200.00	0.0%	0.0%	29.8%	29.8%	0.0%
		Total Net Annual Removal Efficiency			91.7%
		Total Runoff Volume Treated			99.9%

Notes:

- (1) Rainfall Data: 1978:2007, HLY03, Barrie, ONT, 6110557.
- (2) IR PLUS removal based on ETV PSD and ETV protocols.
- (3) Rainfall adjusted to 5 min peak intensity based on hourly average.
- (4) Combined removal efficiencies calculated based on NCDENR Stormwater BMP Manual, Section 3.9.4, where
 Total Removal Efficiency = 1st BMP Efficiency + 2nd BMP Efficiency - (1st BMP Efficiency x 2nd BMP Efficiency)

Verification Statement



Hydro International First Defense® HC Oil Grit Separator Registration number: (V-2018-10-01) Date of issue: (2018-October-15)

Technology type	Oil Grit Separator	
Application	Technology to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals	
Company	Hydro International	
Address	94 Hutchins Drive, Portland, Maine USA 04102	Phone +1-207-756 6200
Website	https://www.hydro-int.com	
E-mail	dscott@hydro-int.com	

Verified Performance Claims

The Hydro International First Defense® High Capacity (HC) Oil Grit Separator (OGS) was tested by Good Harbour Laboratories Inc. (GHL), Mississauga, Ontario, Canada in 2018. The performance test results were verified by Toronto and Region Conservation Authority (TRCA), Vaughan, Ontario, Canada following the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The following performance claims were verified:

Capture test¹:

With a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and an influent test sediment concentration of 200 mg/L, the First Defense® HC OGS device removes 67, 60, 55, 50, 45, 45, and 41 percent of influent sediment by mass at surface loading rates of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m², respectively.

Scour test¹:

With 10.2 cm (4 inches) of test sediment pre-loaded onto a false floor reaching 50% of the manufacturer's recommended maximum sediment storage depth, the First Defense® HC OGS device generates adjusted effluent² concentrations of 0, 0, 11, 2, and 0 mg/L at 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m², respectively.

¹ The claims can be applied to other units smaller or larger than the tested unit as long as the untested units meet the scaling rule specified in the Procedure for Laboratory of Testing of Oil Grit Separators (Version 3.0, June 2014)

² The effluent suspended sediment concentration is adjusted based on the background concentration and the smallest 5% of particles captured during the 40 L/min/m² sediment capture test (see Table 2)

Technology Application

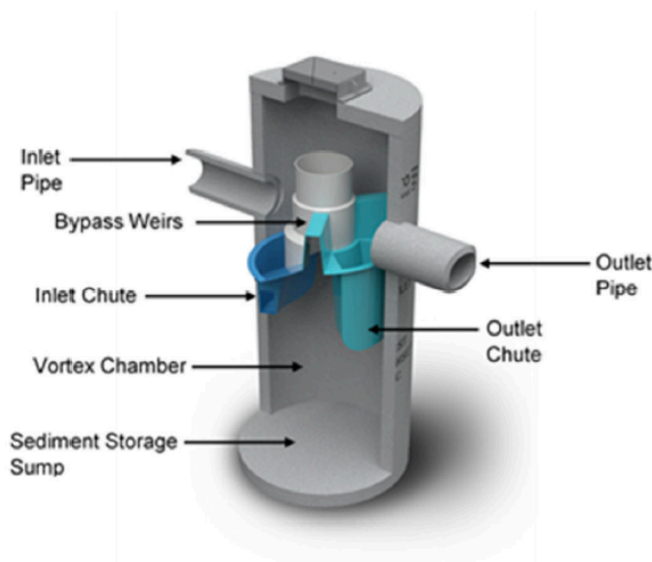
The First Defense® HC (FDHC) Oil Grit Separator can be used as a stand-alone stormwater treatment technology, depending on water quality objectives, or as a pretreatment component in a treatment train when higher TSS removals are required and polishing or volume reduction best management practices (BMPs), such as infiltration or bio-infiltration, are installed downstream. FDHC applications include: stormwater treatment at the point of entry into the drainage line; sites constrained by space, topography or drainage profiles with limited slope and depth of cover; retrofit installations where stormwater treatment is placed on or tied into an existing storm drain line; pretreatment for filters, infiltration, other sedimentation BMPs and storage.

Technology Description

The Hydro International First Defense® HC (FDHC) is an Oil Grit Separator designed to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals. The patented flow modifying internal components are designed to be inserted into standard precast concrete manholes where they collect and treat runoff as part of the drainage system (Figure 1).

Flow entering the manhole via an inlet pipe or inlet grate is diverted into a vortex chamber beneath a separation module that includes both inlet/outlet chutes and bypass weirs. The internal bypass weirs divert flows greater than the maximum design treatment flow rate over the separation module and away from the vortex chamber where oil, sediment, debris and attached pollutants are accumulating. This function prevents high velocities from re-suspending previously captured pollutants during large storm events. The FDHC can be designed and sized to function effectively in either online or offline configurations.

Figure 1: Hydro International First Defense® HC Oil Grit Separator



The test unit was 1.2 m (4 foot) in diameter with a 1.51 m (59 5/8 inches) sump depth measured from the outlet invert to the floor of the unit. The effective treatment area (also known as the effective sedimentation area) is 1.2 m² (12.6 ft²). The maximum sediment storage depth is 0.457 m (18 inches).

Description of Test Procedure

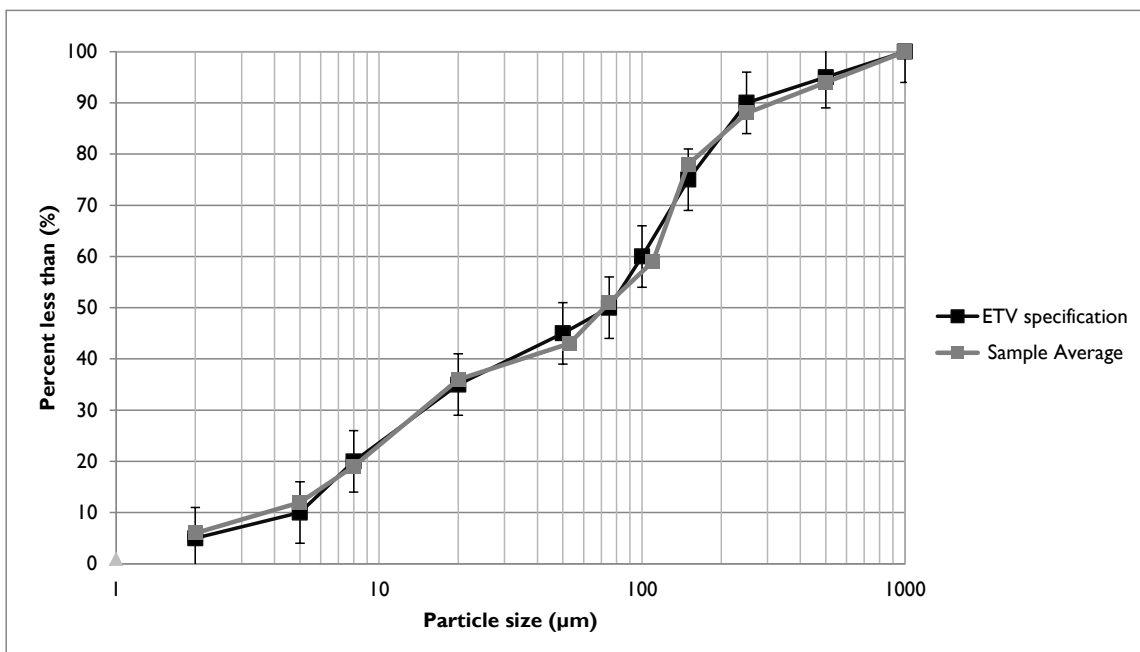
The test data and results for this verification were obtained from independent testing conducted on a 1.2 m (48 inch) diameter Hydro International First Defense® HC OGS device, in accordance with the *Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014)*. The laboratory test procedure was originally prepared by the Toronto and Region Conservation Authority (TRCA) in association with a 31 member advisory committee from various stakeholder groups.

Verification Results

Toronto and Region Conservation Authority verified the performance test data and other information pertaining to the First Defense® HC Oil Grit Separator. A Verification Plan was prepared to guide the verification process based on the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol.

The test sediment consisted of ground silica (1 – 1000 micron) with a specific gravity of 2.65, uniformly mixed to meet the particle size distribution specified in the testing procedure. The *Procedure for Laboratory Testing of Oil Grit Separators* requires that the three sample average of the test sediment particle size distribution (PSD) meet the specified PSD percent less than values within a boundary threshold of 6%, and a median particle size no greater than 75 µm. Comparison of the individual sample and average test sediment PSD to the specified PSD shown in Figure 2 indicates that the test sediment used for the capture and scour tests met this condition. The median particle size was 73 µm. Samples from test sediment batches used for each run met the specified PSD within the required tolerance thresholds.

Figure 2 - The three sample average particle size distribution (PSD) of the test sediment used for the capture and scour test compared to the specified PSD



The capacity of the device to retain sediment was determined at seven surface loading rates using the modified mass balance method. This method involved measuring the mass and particle size distribution of the injected and retained sediment for each test run. Performance was evaluated with a false floor simulating the technology filled to 50% of the manufacturer's recommended maximum sediment storage depth. The test was carried out with clean water that maintained a sediment concentration below 20 mg/L. Based on these conditions, removal efficiencies for individual particle size classes and for the test sediment as a whole were determined for each of the tested surface loading rates (Table 1).

In some instances, the removal efficiencies were above 100% for certain particle size fractions. These discrepancies are not unique to any one test laboratory and are attributed to errors relating to the blending of sediment, collection of representative samples for laboratory submission, and laboratory analysis of PSD. Due to these errors, caution should be exercised in applying the removal efficiencies by particle size fraction for the purposes of sizing the tested device (see Bulletin # CETV 2016-11-0001). The results for “all particle sizes by mass balance” (see Table 1) are based on measurements of the total injected and retained sediment mass, and are therefore not subject to blending, sampling or PSD analysis errors.

Table 1 - Removal efficiencies (%) of the First Defence HC at specified surface loading rates

Particle size fraction (µm)	Surface loading rate (L/min/m ²)						
	40	80	200	400	600	1000	1400
>500	100*	100*	100*	81	72	86	80
250 - 500	100*	97	99	100*	100*	59	88
150 - 250	100*	91	95	93	47	100*	84
105 - 150	96	89	94	89	90	70	75
75 - 105	100*	90	95	77	-20**	100	51
53 - 75	74	100*	97	62	100*	46	37
20 - 53	60	33	10	5	4	0	0
8 - 20	29	16	8	3	3	1	1
5 - 8	8	5	8	4	4	4	3
<5	5	3	0	0	0	3	3
All particle sizes By mass balance	66.5	59.9	55.4	50.2	44.9	45.2	40.5

* Removal efficiencies were calculated to be above 100%. Calculated values ranged between 101 and 184% (average 115%). See text and Bulletin # CETV 2016-11-0001 for more information.

** An outlier in the retained sediment sample sieve data resulted in negative removal for this size fraction. The outlier at the 75 µm particle size is shown in Figure 3.

Figure 3 - Particle size distribution of sediment retained in the First Defence HC in relation to the injected test sediment average

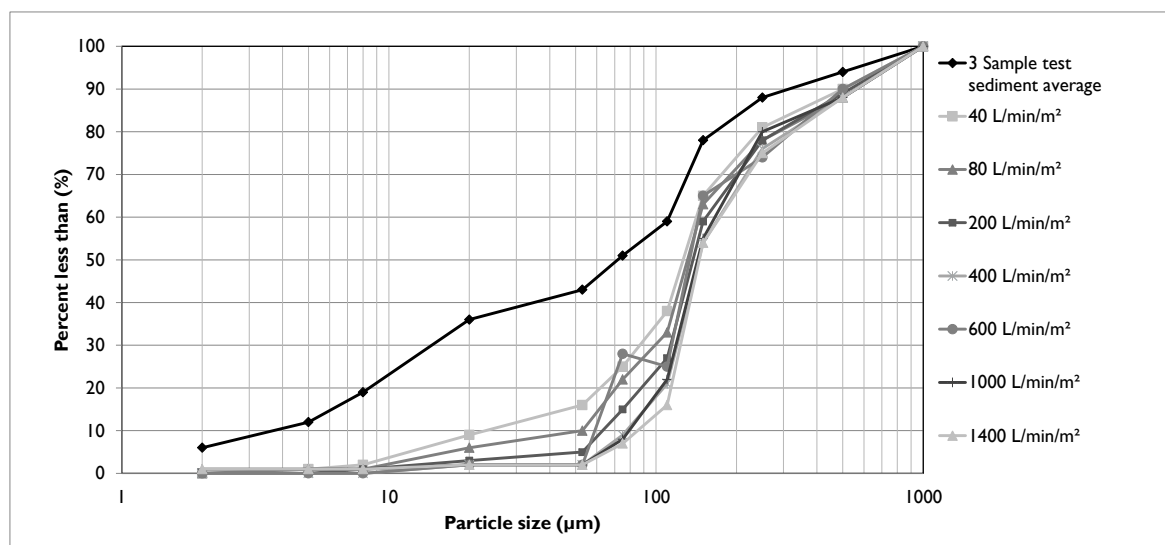


Figure 3 compares the particle size distribution (PSD) of the three sample average of the test sediment to the PSD of the sediment retained by the FDHC device at each of the tested surface loading rates. As expected, the capture efficiency for fine particles was generally found to decrease as surface loading rates increased, particularly in the 40 to 400 L/min/m² range.

Table 2 shows the results of the sediment scour and re-suspension test for the First Defense® HC unit. The scour test involved preloading 10.2 cm (4 inches) of fresh test sediment into the sedimentation sump of the device. The sediment was placed on a false floor to mimic a device filled to 50% of the maximum recommended sediment storage depth. Clean water was run through the device at five surface loading rates over a 30 minute period. Each flow rate was maintained for 5 minutes with a one minute transition time between flow rates. Effluent samples were collected at one minute sampling intervals and analyzed for Suspended Sediment Concentration (SSC) and PSD by recognized methods. The effluent samples were subsequently adjusted based on the background concentration of the influent water. The smallest 5% of particles captured during the 40 L/min/m² sediment capture test (13.5 µm in this case) was used to further adjust the effluent sediment concentrations, as per the method described in Bulletin # CETV 2016-09-0001. Results showed average adjusted effluent sediment concentrations below 11 mg/L at all surface loading rates. Effluent concentrations would be expected to decrease at higher flow rates since bypass over the insert bypass weirs was observed to begin at 1,032 L/min/m².

Table 2 - Scour test adjusted effluent sediment concentration at each surface loading rate

Run	Surface loading rate (L/min/m ²)	Run time (min)	Background sample concentration (mg/L)	Average adjusted effluent suspended sediment concentration (mg/L)*
1	200	1:00 – 6:00	0.8	0
2	800	7:00 – 12:00	0.8	0
3	1400	13:00 – 18:00	1.3	10.6
4	2000	19:00 – 24:00	0.8	2.4
5	2600	25:00 – 30:00	6.6	0

*The effluent suspended sediment concentration is adjusted based on the background concentration and the smallest 5% of particles captured during the 40 L/min/m² sediment capture test, as per the method described in Bulletin # CETV 2016-09-0001.

Variances from the Procedure

Minor variances from the *Procedure for Laboratory Testing of Oil-Grit Separators* used as the basis of testing for this verification were as follows:

1. The *Procedure* states that the tested device “must be a full scale commercially available device with the same configuration and components as would be typical for an actual installation.” The unit tested for this verification had the same internal components as would be typical for a commercial installation, but the internal components were placed inside a structure constructed of composite materials, rather than a manhole made of concrete, the latter of which is typical for most installations. The dimensions of the structure were the same as would have been the case had the manhole been concrete. The use of alternate materials for the structure was not believed to significantly affect system performance.
2. As part of the capture test, evaluation of the 40 and 80 L/min/m² surface loading rate was split into 3 and 2 parts, respectively. The test was conducted in parts because of the long duration (i.e. over 10 hours) needed to feed the required minimum 11.3 kg of test sediment into the unit. At the end of the first and second parts of the test, the flow rates were gradually decreased to prevent capture of particles that would have been washed out under normal circumstances. The requirement to split the test into parts was not anticipated in the *Procedure for Laboratory Testing of Oil-Grit Separators*, but has been a common feature of testing at the 40 L/min/m² surface loading rate. Conducting the test in two parts for the 80 L/min/m² surface loading rate is less common. The testing did not assess the significance of the breaks, however, the test laboratory and verifier do not believe that the breaks significantly affected the test results.

Hydro International First Defense® HC Oil Grit Separator Verification Statement

3. During the sediment scour test, the flow rate coefficient of variation (COV) at the 200 L/min/m² surface loading rate of 0.045 slightly exceeded the target COV of 0.04. The average flow rate during the test remained within ±10% of the target flow rate.

Quality assurance

Performance testing and verification of the First Defense® HC Oil Grit Separator were performed in accordance with the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The verifier, Toronto and Region Conservation Authority, has confirmed that quality assurance requirements were addressed throughout the performance testing process and in the generation of performance test results. This includes reviewing all data sheets and data downloads, as well as overall management of the test system, quality control and data integrity.

Verification Summary

In summary, the First Defense® HC Oil Grit Separator is designed to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals. Verification of performance claims for the Hydro International First Defense® HC Oil Grit Separator was conducted by Toronto and Region Conservation Authority based on independent third-party performance test results provided by Good Harbour Laboratories, as well as additional information provided by Hydro International. Table 3 summarizes the verification results in relation to the technology performance parameters that were identified to determine the efficacy of the First Defense® HC Oil Grit Separator.

Table 3 - Summary of Verification Results Against Performance Parameters

Performance Parameter	Verified Performance
Sediment Removal Rate	The sediment removal rate of the FDHC is dependent upon flow rate, particle density and particle size. Removal efficiency decreased with increasing surface loading rate from 67% at 40 L/min/m ² to 41% at 1400 L/min/m ² . The weighted average removal efficiency achieved by the unit will vary depending on the rainfall distribution of the jurisdiction in which it is installed, and site characteristics.
Sediment Scour	When pre-loaded with sediment with a particle size distribution matching that of the feed sediment used in the sediment capture test, the FDHC generated effluent suspended solids concentrations of less than 11 mg/L at surface loading rates ranging from 200 to 2600 L/min/m ² .
Bypass flow rate	The flow rate at which bypass occurs will vary based on model size. For the 1.2 m (4 foot) diameter test unit, the flow rate at which bypass occurred over the insert bypass weirs was 1238 L/min (327 gpm).
Head loss	The loss of hydraulic head across the FDHC was determined by measuring the water elevation difference between the inlet and outlet sides of the insert. Head loss may vary based on model size. For the tested unit the head loss ranged from 2 mm (0.08 inches) at 93.5 L/min (12.3 gpm) to 100 mm (3.94 inches) at 1238 L/min (327 gpm) when bypass was observed to occur. At 327 gpm, when bypass occurred, the depth of the water was 177 mm upstream and 77 mm downstream for a difference of 100 mm (3.94 inches). The highest water elevation difference was 111mm (4.37 inches) at a flow rate of 1635 L/min (431.8 gpm), after which head loss declined up to the maximum measured flow rate of 3036 L/min (801.9 gpm).

What is ISO 14034?

The purpose of environmental technology verification is to provide a credible and impartial account of the performance of environmental technologies. Environmental technology verification is based on a number of principles to ensure that verifications are performed and reported accurately, clearly, unambiguously and objectively. The International Organization for Standardization (ISO) standard for environmental technology verification (ETV) is ISO 14034, which was published in November 2016.

Benefits of ETV

ETV contributes to protection and conservation of the environment by promoting and facilitating market uptake of innovative environmental technologies, especially those that perform better than relevant alternatives. ETV is particularly applicable to those environmental technologies whose innovative features or performance cannot be fully assessed using existing standards. Through the provision of objective evidence, ETV provides an independent and impartial confirmation of the performance of an environmental technology based on reliable test data. ETV aims to strengthen the credibility of new, innovative technologies by supporting informed decision-making among interested parties.

For more information on the First Defense® HC Oil Grit Separator, contact:	For more information on VerifiGlobal, contact:
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Signed for Hydro International: <i>Original signed by:</i> David Scott David Scott Technical Product Manager, Americas Stormwater	Signed for VerifiGlobal: <i>Original signed by:</i> Thomas Bruun Thomas Bruun, Managing Director <i>Original signed by:</i> John Neate John Neate, Managing Director

NOTICE: Verifications are based on an evaluation of technology performance under specific, predetermined operational conditions and parameters and the appropriate quality assurance procedures. VerifiGlobal and the Verification Expert, Toronto and Region Conservation Authority, make no expressed or implied warranties as to the performance of the technology and do not certify that a technology will always operate as verified. The end user is solely responsible for complying with any and all applicable regulatory requirements. Mention of commercial product names does not imply endorsement.

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Project DEVELOPMENT Summary

DEVELOPMENT: 21-710 642 Dunlop St. West

Subwatershed: Barrie Creeks

Total Pre-Development Area (ha):	0.810	Total Pre-Development Phosphorus Load (kg/yr):	0.77
----------------------------------	--------------	--	-------------

Pre-Development Land Use	Area (ha)	P coeff. (kg/ha)	P Load (kg/yr)
Forest	0.4	0.05	0.02
High Intensity - Comm/Industrial	0.41	1.82	0.75

POST-DEVELOPMENT LOAD

Post-Development Land Use	Area (ha)	P coeff. (kg/ha)	Best Management Practice applied with P Removal Efficiency	P Load (kg/yr)
Forest	0.18	0.05	NONE	0% 0.01

The undeveloped wetland buffer area within catchment B1.

High Intensity - Residential	0.27	1.32	Treatment Train Approach	40%	0.21
------------------------------	------	------	--------------------------	-----	------

*Catchment A1 routed through the ETV OGS (20% removal) and the underground storage tank (25% removal).
Composite removal = 40%*

High Intensity - Residential	0.12	1.32	NONE	0%	0.16
------------------------------	------	------	------	----	------

Catchment A2 + 0.07ha of Catchment B1 that comprises the western driveway and perimeter landscape area

High Intensity - Residential	0.24	1.32	Other	0%	0.32
------------------------------	------	------	-------	----	------

Rooftop area (catchments B2a+b+c) draining to wetland

Post-Development Area Altered:	0.81	P Load (kg/yr)
Total Pre-Development Area:	0.81	

Unaffected Area: **0**

Pre-Development: **0.77**
 Post-Development: **0.84**
 Change (Pre - Post): **-0.07**

10% Net Increase in Load

Post-Development (with BMPs): **0.70**
 Change (Pre - Post): **0.07**

9% Net Reduction in Load

DEVELOPMENT: 21-710 642 Dunlop St. West
Subwatershed: Barrie Creeks

CONSTRUCTION PHASE LOAD

SUMMARY WITH IMPLEMENTATION OF BMPs		P Load (kg/yr)
Pre-Development:		0.77
Construction Phase Amortized Over 8 Years :		to be determined
Post-Development:		0.70
Post-Development + Amortized Construction:		to be determined
Pre-Development Load - Post-Development Load:		0.07
Conclusion:		9% Reduction in Load
Pre-Development Load - (Post-Development + Amortized Construction Load):		to be determined
Conclusion:		to be determined
Based on a comparison of Pre-Development and Post-Development loads, and in consideration of Construction Phase loads, the Ministry would encourage the Municipality to:		

LSRCA Water Balance offsetting costs

642 Dunlop St. West (Project #21-710)

March 10, 2023

Deficit Summary:

- Water Balance Deficit: 893m³/year (see Terraprobe's Hydrogeological Assessment).

Offset Policies & Costs:

Ref: Water Balance Recharge Policy for the Lake Simcoe Protection Plan (July 2021)

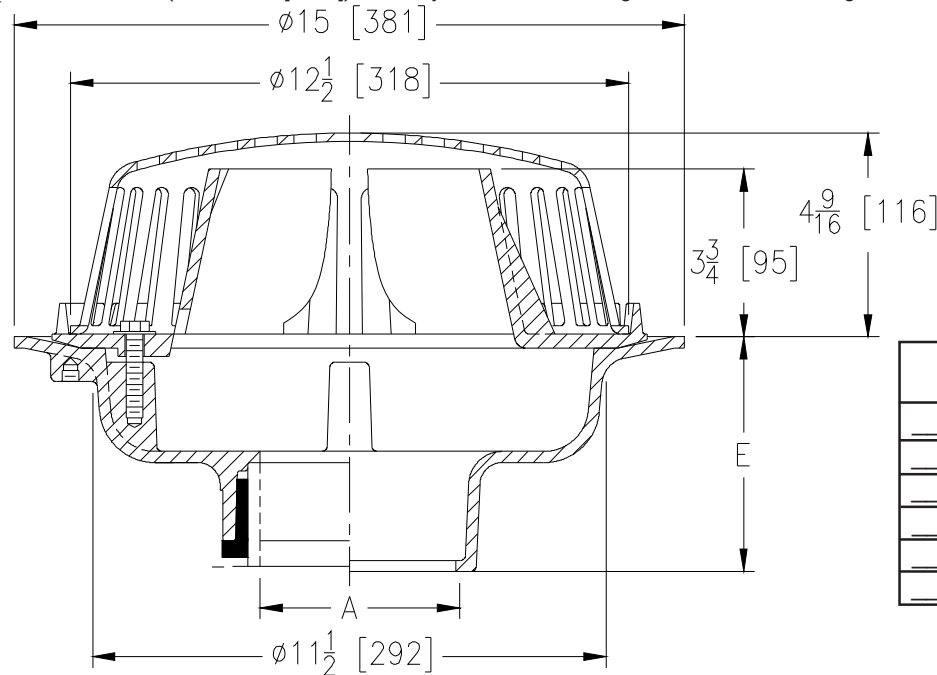
From Table 5: LSRCA Water Balance Offsetting Fee Schedule:

Cost formula = [893m³ x \$44.97] x 1.15

Total WB Offset Cost= \$46,181.94

**Z105****CONTROL-FLO ROOF DRAIN
W/ PARABOLIC WEIR****SPECIFICATION SHEET****TAG** _____

Dimensional Data (inches and [mm]) are Subject to Manufacturing Tolerances and Change Without Notice



Specify Number of Notches in Weir	
___ -N1	One Notch
___ -N2	Two Notches
___ -N3	Three Notches
___ -N4	Four Notches
___ -N5	Five Notches
___ -N6	Six Notches

A- Pipe Size In.[mm]	Approx. Wt. Lbs. [kg]	Dome Open Area Sq. In. [cm ²]
2,3,4 [51,76,102]	34 [15]	103 [665]

ENGINEERING SPECIFICATION: ZURN Z105

15" [381mm] Diameter Control-Flo roof drain for dead-level roof construction, Dura-Coated cast iron body, Control-Flo weir shall be linear functioning with integral membrane flashing clamp/gravel guard and Poly-Dome. All data shall be verified proportional to flow rates. Each notch will allow 10 GPM [LPM] of flow per 1" [25mm] of rain water build up above the drain.

OPTIONS (Check/specify appropriate options)**PIPE SIZE**

3, 4 [76, 102]
2 , 3, 4 [51, 76, 102]
2 , 3, 4 [51, 76, 102]

(Specify size/type) OUTLET

___ IC Inside Caulk
___ NH No-Hub
___ NL Neo-Loc

E BODY HT. DIM.

5-1/4 [133]
5-1/4 [133]
4-9/16 [116]

PREFIXES

___ Z D.C.C.I. Body with Poly-Dome*
___ ZA D.C.C.I. Body with Aluminum Dome
___ ZC D.C.C.I. Body with Cast Iron Dome

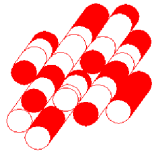
SUFFIXES

___ -C Underdeck Clamp
___ -DP Top-Set® Deck Plate (Replaces both -C & -R)
___ -E Static Extension 1 [25] thru 4 [102] (Specify Ht.)
___ -EA Adjustable Extension Assembly
2-1/8 [54] thru 3-1/2 [89]
___ -G Galvanized Cast Iron
___ -R Roof Sump Receiver
___ -TC Neo-Loc Test Cap Gasket (2,3,4
[51,76,102] NL Bottom Outlet Only)
___ -VP Vandal Proof Secured Top
___ -10 6 [152] High Parabolic Weir for
Sloped Roof (ZC or ZA)

* Regularly furnished unless otherwise specified.

APPENDIX C

**GEOTECHNICAL INVESTIGATION
REPORT, HYDROGEOLOGICAL
ASSESSMENT, AND
SUPPLEMENTAL INFORMATION (BY
TERRAPROBE INC.)**



Terraprobe

*Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing*

**GEOTECHNICAL INVESTIGATION AND
ENGINEERING REPORT
PROPOSED MIXED-USE DEVELOPMENT
642 DUNLOP STREET WEST
BARRIE, ONTARIO**

Prepared For: **25707517 ONTARIO INC.**
c/o Innovative Planning Solutions
647 Welham Road, Unit 9A
Barrie, Ontario
L4N 0B7

Attention: Ms. Karla Tomayo

File No. 3-21-0182-01
April 6, 2022

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Distribution of Report:

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1.0 THE PROJECT

Terraprobe Inc. (Terraprobe) was retained by 25707517 Ontario Inc. to conduct a subsurface investigation and prepare a geotechnical design report for the re-development of a parcel of land with municipal address 642 Dunlop Street West, in the City of Barrie, Ontario. A site location plan is provided as Figure 1.

This report encompasses the geotechnical investigation conducted for the property to assess its geotechnical suitability for the intended re-development. The field investigation consisted of advancing a total of nine (9) exploratory boreholes (eight (8) boreholes were advanced with regular sampling and one (1) borehole was augered without sampling for installation of a groundwater monitoring well) across the site. The objective of the geotechnical investigation was to determine the prevailing subsurface soil and groundwater conditions, in order to provide geotechnical engineering recommendations for the design of the proposed building foundations, slab-on-grade, earthquake and earth pressure design parameters, pavement design, pipe bedding and installation of underground utilities. In addition, comments are also included on the pertinent project construction aspects including excavation, backfill and groundwater control.

Terraprobe was also retained to conduct a hydrogeological study to assess dewatering requirements for the property. The findings of the hydrogeological study are reported under separate cover.

2.0 SITE AND PROJECT DESCRIPTION

The property is located at the northeast quadrant of Dunlop Street West and Miller Road in Barrie, Ontario, and currently houses two low-rise commercial structures. The municipal addresses are 642 Dunlop Street West, Barrie.

Based on the architectural drawing set (Project: Proposed 5-Storey Condo Building, 642 Dunlop Street W., Barrie, ON; Project Number 15-48) dated Dec. 15, 2015, updated Feb. 4, 2022, prepared by n Architecture Inc., it is proposed to demolish the existing structures and construct a new 5-storey mixed use building with slab-on-grade construction (no basement), commercial use and covered parking/driveway at ground level and residential use above. There will also be at-grade exterior parking and driveway. The ground finished floor elevation was not yet established at the time of this investigation, however it is understood that it may be in the range of Elev. 235.80 m to 236.10 m which would be close to or above current site grades.

As such, the geotechnical investigation was conducted to assess the geotechnical suitability of the property for the intended development. The geotechnical investigation consisted of advancing a total of nine (9) boreholes (eight (8) boreholes were advanced with regular sampling and one (1) borehole was augered without sampling for installation of a groundwater monitoring well) across the site to obtain subsurface soil and groundwater information.



3.0 INVESTIGATION PROCEDURE

The field investigation was conducted between January 18 and 20, 2021, and consisted of drilling and sampling a total of eight (8) exploratory boreholes (Boreholes 1 to 8) and one borehole (Borehole 9) was augered without sampling for installation of a groundwater monitoring well. The boreholes extended to the depths ranging from about 1.8 to 12.6 m below existing ground surface.

The boreholes were staked out by Terraprobe based on the existing site features. The approximate borehole locations are shown on enclosed Borehole Location Plan (Figure 2A- Existing Condition and Figure 2B – Proposed Condition).

Various utility locate agencies (including a private locate company) were contacted by Terraprobe to clear the borehole locations prior to the commencement of the field investigation.

The ground surface elevations at the borehole locations were surveyed by Terraprobe. Borehole elevations are provided relative to Geodetic Datum (NAD). The horizontal coordinates are reported relative to the Universal Transverse Mercator geographic coordinate system (UTM Zone 17T). It should be noted that the elevations provided on the Borehole Logs are approximate and provided only for the purpose of relating borehole soil stratigraphy and should not be used or relied on for other purposes.

The borings were drilled by a specialist drilling contractor using a track/truck mounted drill rig power auger drilling techniques and sampled at regular intervals with a conventional 50 mm diameter split barrel samplers when the Standard Penetration Test (SPT) was carried out (ASTM D 1586). Additionally, dynamic cone penetration tests were conducted beside Boreholes 2, 3 and 4 and the Penetration Test Values are shown on those borehole logs. The field work (drilling, sampling, and testing) was observed full time and recorded by a Terraprobe field staff, who logged the boring and examined the samples as they were obtained.

All samples obtained during the investigation were sealed into plastic jars and transported to our geotechnical laboratory for detailed inspection and testing. The borehole samples were examined (tactile) in detail by a geotechnical engineer and classified according to visual and index properties. Geotechnical laboratory testing consisted of water content determination on all samples, while a Sieve and Hydrometer analysis was conducted on four (4) selected soil samples (Borehole 1, Sample 5; Borehole 2, Sample 4; Borehole 3, Sample 7 and Borehole 4, Sample 10). The measured natural water contents of individual samples and the results of the Sieve and Hydrometer analyses tests are plotted on the enclosed borehole logs at respective sampling depths. The results of the Sieve and Hydrometer analyses tests are also summarized in Section 4.5 of this report and appended.

Groundwater levels were observed in the open boreholes upon completion of drilling. Monitoring wells were installed in Boreholes 1 to 5 and 9 to facilitate groundwater monitoring and for hydrogeological purposes. The results of the groundwater monitoring are summarized in Section 4.6 of this report.



4.0 SUBSURFACE CONDITIONS

The specific soil conditions encountered at each borehole location are described in greater detail on the Borehole Logs, with a summary of the general subsurface soil conditions outlined below. This summary is intended to correlate this data to assist in the interpretation of the subsurface conditions at the site.

It should be noted that the subsurface conditions are confirmed at the borehole locations only and may vary between and beyond the borehole locations. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

4.1 Surficial Materials

A pavement structure consisting of 130 mm to 230 mm of asphaltic concrete overlying 130 mm to 370 mm of granular fill was encountered at the surface of Boreholes 1, 3, 5 and 6. A 230 mm thick layer of asphaltic concrete was encountered at the surface at Borehole 8, however there was no granular base material observed below the asphaltic concrete at this borehole. Surficial topsoil with thickness ranging from about 200 to 450 mm was encountered at the ground surface in Borehole 2. Asphaltic concrete with thickness of about 50 mm and underlain by a 150 mm thick granular base/aggregate layer was encountered at the ground surface in Borehole 3.

Surficial topsoil with thickness ranging from about 140 to 150 mm was encountered at the ground surface in Boreholes 2, 4 and 7. The topsoil generally consisted of a silt matrix and was dark brown to black in colour.

The above noted pavement structure and topsoil thicknesses were measured through the collar of the boreholes and are approximate. The pavement structure and topsoil are expected to vary between and beyond the boreholes. We recommend that a shallow test pit investigation be carried out to determine precise pavement structure and topsoil thickness present at the site for quantity estimation and costing purposes (if required).

4.2 Earth Fill

Underlying the surficial topsoil in Boreholes 2, 4 and 7 and the asphaltic concrete in Borehole 8, a layer of earth fill was encountered and extended to a depth of about 0.6 m (Elev. 233.7 m to 234.8 m) below grade. The earth fill material generally consisted of sand or sandy silt with peat and gravel.

Standard Penetration Test results (N-values) obtained from the earth fill zone ranged from 4 to 22 blows per 300 mm of penetration, indicating a loose to compact relative density. The in-situ moisture contents of the earth fill samples ranged from 9 to 27 percent by weight, indicating a generally moist condition. Elevated moisture contents may be attributable to the presence of organic matter in the fill.



4.3 Peat

A layer peat was encountered below the pavement structure in Boreholes 1, 3, 5 and 6, and below the earth fill in Boreholes 2, 4, 7 and 81 and below the organic silt in Borehole 2 and extended to a depth ranging from about 0.8 to 1.5 m (Elev. 232.9 m to 234.5 m) below grade. The peat had inclusions of sand seams.

The Standard Penetration Test results ('N' Values) obtained from the peat deposit ranged from 2 to 14 blows per 300 mm of penetration, indicating a very loose to compact relative density (generally loose). The measured moisture contents of the peat samples ranged from 32 to 132 percent by weight, indicative of the organic nature of the material.

It should be noted that the peat can be a methane generating material. Any such investigation was beyond the scope of this investigation.

4.4 Sand

Sand with trace amount of silt, clay and gravel was encountered below the peat layer in all boreholes and extended to depth of investigation in all boreholes, i.e., depth ranging from 1.8 to 12.6 m below grade. A thin layer of sandy silt was encountered between 4.6 and 5.0 m (Elev. 230.7 m and 230.3 m) in Borehole 3. The sand generally became water bearing below about 1.5 ±m depth below grade.

Standard Penetration Test results (N-values) obtained from the sand deposit ranged from 0 to 46 blows per 300 mm of penetration, indicating a loose to dense relative density (generally very loose to loose or compact to about 9 ±m depth and becoming dense below). However, it should be noted that some of these 'N' values are likely affected by disturbance of the sands from augering.

As noted earlier, dynamic cone penetration tests (DCPT) were conducted beside Boreholes 2, 3 and 4 to get a better appreciation of Penetration values of the sand deposit. At Borehole 2, the DCPT values ranged from 0 to 7 blows per 300 mm of penetration from a depth of 1.2 to 3.5 m below grade; from about 10 to 20 blows per 300 mm penetration from a depth of 3.5 to 7.5 m below grade and generally about greater than 30 blows per 300 mm of penetration from 8 to 12.6 m depth. At Borehole 3, the DCPT values ranged from 0 to 5 blows per 300 mm of penetration from 1.5 to about 5.5 m depth below grade and from about 10 to 22 blows per 300 mm of penetration from about 6.0 to 8.1 m depth. At Borehole 4, the DCPT values ranged from about 4 to 10 blows per 300 mm of penetration between 0.9 and 1.8 m depth below grade, from about 15 to 20 blows per 300 mm of penetration from about 1.8 to 4.0 m depth below grade; 30 to 40 blows per 300 mm of penetration between 4.0 and 8.5 m depth below grade and generally greater than 50 blows per 300 mm penetration or less from 8.5 to 12.6 m depth.



The in-situ moisture contents of the sand samples ranged from 14 to 29 percent by weight, indicating a wet condition.

4.5 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural moisture content determination for all samples from the boreholes, while Sieve and Hydrometer analysis tests were conducted on selected soil samples. The test results are plotted on the enclosed Borehole Logs at respective sampling depths.

The results (graphs) of the Sieve and Hydrometer (grain size) analysis are appended, and a summary of these results is presented as follows:

Borehole No. Sample No.	Sampling Depth below Grade (m)	Percentage (by mass)				Descriptions (MIT System)
		Gravel	Sand	Silt	Clay	
Borehole 1, Sample 5	3.3	6	89	3	2	SAND, s trace gravel, trace silt, trace clay
Borehole 2, Sample 4	2.5	0	97	2	1	SAND, trace silt, trace clay
Borehole 3, Sample 7	6.3	0	95	3	2	SAND, trace silt, trace clay
Borehole 4, Sample 10	10.9	5	91	2	2	SAND, , trace gravel trace silt, trace clay

4.6 Groundwater

Observations pertaining to the depth of groundwater level and caving were made in the open boreholes immediately after completion of drilling in all boreholes and are noted on the enclosed Borehole Logs. Monitoring wells were installed in all boreholes to facilitate groundwater level monitoring and for the purpose of Terraprobe's geotechnical, environmental, and hydrogeological studies. The groundwater readings taken on February 1, 2022, are noted on the enclosed Borehole Logs. A summary of these observations is provided as follows:

Borehole No.	Ground Surface Elevation (m)	Depth of Boring below Grade (m)	Depth to Cave below Grade (m)	Groundwater Level Depth/Elevation (m)	
				At the Time of Drilling	Monitoring Well February 1, 2022
1	235.5	8.1	Not measured	1.5/234.0	Well Inaccessible



Borehole No.	Ground Surface Elevation (m)	Depth of Boring below Grade (m)	Depth to Cave below Grade (m)	Groundwater Level Depth/Elevation (m)	
				At the Time of Drilling	Monitoring Well February 1, 2022
2	234.5	12.6	Not measured	1.5/233.0	1.7/232.8
3	235.3	8.1	Not measured	2.3/233.0	1.8/233.5
4	235.4	12.6	Not measured	2.3/233.1	1.9/233.5
5	235.2	8.1	Not measured	1.5/233.7	1.5/233.7
6	235.7	1.8	Open	1.5/234.2	Monitoring well not installed
7	234.3	1.8	Open	dry	Monitoring well not installed
8	235.1	1.8	Open	dry	Monitoring well not installed
9	234.4	5.0	Open	1.5/232.9	1.7/232.7

The groundwater levels encountered in the monitoring wells on February 1, 2022 monitoring) indicated a shallow water table with an elevation ranging from about 232.7 m to 233.7 m, i.e., a depth ranging from about 1.5 to 1.9 m below existing grade. Groundwater levels may fluctuate with time, and seasonally, depending on the amount of precipitation and surface runoff.

5.0 GEOTECHNICAL DESIGN

The following discussion and recommendations are based on the factual data obtained from this investigation and are intended for use by the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards, and guidelines of practice. If there are any changes to the site development features or any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

It is understood that the development concept consists of the demolition of the existing structures and construction of a new 5-storey mixed use building with slab-on-grade construction (no basement), commercial use and covered parking at ground level and residential use above. The ground finished floor



elevation was not yet established at the time of this investigation, however it is understood that it may be in the range of Elev. 235.80 m to 236.10 m which would be close to or above current site grades.

5.1 Foundation Design Parameters

Based on the borehole findings, below any surficial materials (topsoil or pavement structure) the subsurface conditions generally consist of loose sand fill with peat organics and or loose an organic peat layer extending to a depth of up to 1.5 m below grade. These materials are underlain by the native sand deposit which is generally very loose to loose with some compact zone to about 6 to 8 m depth and becoming generally dense below about 8± m depth.

Given the prevailing very poor upper subsurface conditions on the site, conventional footings on native soils are not expected to be feasible for this structure. As such considerations should be given to ground improvement or the use of helical piers to support the building.

5.1.1 Ground Improvement Techniques

Consideration may be given to ground improvement techniques, which are used to improve the geotechnical properties of relatively weak and poor earth fill and native soils underlying a site to facilitate the construction of the intended structures using conventional construction techniques and design.

Using ground improvement on this site will allow the use of conventional spread footings. It is estimated that the maximum net geotechnical reaction at SLS is about 150 to 200 kPa, and the maximum factored geotechnical resistance at ULS is about 225 to 300 kPa for preliminary design, to be confirmed by GeoSolv or other design-build firms.

There are various patented ground improvement techniques available. For this site, the Geopier Rammed Aggregate Pier (RAP) system may be considered. Improving the soils using the RAP System will allow for conventional spread footing foundations to support the proposed structure. Rigid inclusions or other ground improvement techniques may be required at this site, however, the design-build firm can make that determination.

5.1.2 Helical Earth Screw Anchors

Helical Screw Anchors (see enclosed information in Appendix C) may also be considered to support the proposed structure and could be drilled through the generally loose upper layer soils (fill, peat, and loose sand zone) and into the underlying compact to dense native sand soils to sufficient depth in order to obtain the adequate resistance for the required support for the foundation. The actual pile design is recommended to be provided by a structural engineer from a specialist contractor who would design, install, and certify the foundations. The screw anchors require little to no excavation to install and are not affected by high groundwater levels.



The helical pile section would be drilled into the competent strata which provides sufficient resistance. The section is extended further as needed with extension rods. A pile cap / grade beam is constructed to transfer the building loads onto the underlying competent soils through the helical piers. The actual depth of the helical screw pile is determined on site during installation based on field torque readings and load support requirements. The load carrying capacity of the helical piers is confirmed by the Proprietary Design-Build company based on a full-scale prototype/production pile test.

There are specific companies (such as EBS Geotechnical or other design-build firms) that specialize in the helical screw anchor design and installation, and can provide further information on the methodology, detailed design, installation, and certification. It should be noted that the presence of debris/obstructions within the fill material, if present, could impede the helical pier installation.

The following parameters may be assumed for the underlying dense native soil:

	Sand (Dense)
Soil unit weight (γ_{bulk})	21 kN/m ³
Angle of Internal Friction (ϕ)	34°
Cohesion (c)	0 kPa

For round shaft helical pier design (RS3500), the following preliminary parameters (Ref: EBS Geotechnical Round Shaft Helical Pier Capacity Charts) may be considered below Elevation 226 to 227 m or lower, subject to consultation and confirmation from the design-build contractor:

- Max. Factored Geotechnical Resistance, ULS (kN): 280
- Net Geotechnical Reaction, SLS (kN): 210

The refusal depth of these piers will depend upon the type of helical pier section and loading considerations. It is recommended that the project geotechnical information be provided to a specialized design/build contractor to assess the feasibility of this foundation system and to determine helical pier refusal/installation depths, and any further subsurface investigation, if required.

5.1.3 Placement of Footings

Prior to placing concrete for foundations of the structures, the foundation areas must be cleaned of all deleterious materials such as topsoil, fill, and softened, disturbed, or caved materials, as well as any standing water.



If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. Native soils tend to weather rapidly and deteriorate on exposure to the atmosphere and surface water. Hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

The underside of footing elevations must be designed with a minimum of 1.5 m of soil cover from the proposed site grading elevation or equivalent insulation to the foundation subgrade for frost protection in unheated areas.

Footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal.

5.2 Earthquake Design Parameters

The Ontario Building Code stipulates the methodology for earthquake design analysis. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration, and the site classification.

Under Ontario Regulation 88/19, the ministry amended Ontario's Building Code (O. Reg 332/12) to further harmonize Ontario's Building Code with the 2015 National Codes. These changes will help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the Ontario Building Code (OBC) by uniform hazard spectra (UHS) at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g., shear wave velocity (v_s), Standard Penetration Test (SPT) resistance, and undrained shear strength (s_u) in the top 30 meters of the site stratigraphy below the foundation level, as set out in the Ontario Building Code. There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g., sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients F_a and F_v , respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information, it is recommended that the site designation for seismic analysis be 'Site Class D', as per the Ontario Building Code. Consideration may be given to conducting a site specific Multichannel Analysis of Surface Waves (MASW) at this site to confirm the average shear wave velocity in the top 30 metres of the site stratigraphy.



The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I. using linear interpolation for intermediate values of PGA.

5.3 Earth Pressure Design Parameters

The parameters used in the determination of earth pressures acting on retaining walls are defined below.

Parameter	Definition	Units
ϕ	internal angle of friction	degrees
γ	bulk unit weight of soil	kN / m ³
K_a	active earth pressure coefficient (Rankine)	dimensionless
K_o	at-rest earth pressure coefficient (Rankine)	dimensionless
K_p	passive earth pressure coefficient (Rankine)	dimensionless

The appropriate values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Stratum/Parameter	ϕ	γ	K_a	K_o	K_p
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Existing Earth Fill	29	19.0	0.35	0.52	2.88
Sand	34	21.0	0.28	0.44	3.54

Walls subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

where,

- P = the horizontal pressure at depth, h (m)
- K = the earth pressure coefficient
- h_w = the depth below the groundwater level (m)
- γ = the bulk unit weight of soil, (kN/m³)
- γ' = the submerged unit weight of the exterior soil, ($\gamma - 9.8$ kN/m³)
- q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall that would otherwise act in conjunction with the earth pressure, this equation can be simplified to:



$$P = K[\gamma h + q]$$

This equation assumes that free-draining granular backfill such as Granular 'B' (OPSS.MUNI 1010) is used and effective drainage is provided. Consideration must also be given to the possible effects of frost on structures retaining earth. Pressures induced by freezing in frost-susceptible soils exert pressures and are effectively irresistible.

The factored geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction (**R**) depends on the normal load of the soil contact (**N**) and the frictional resistance of the soil (**tan φ**) expressed as $R_f = N \tan \phi$, which is the unfactored resistance. The factored geotechnical resistance at ULS is $R_f = 0.8 N \tan \phi$.

5.4 Slab-on-Grade Design Parameters

The slab on grade should be set at least 0.5 m above the seasonally high groundwater level.

The proposed ground floor slab (understood range: Elev. 235.80 m to 236.10 m) is expected to be close to or above existing grade. As such some grade raise/placement of fill may be required. All non-structural earth fill, peat and other deleterious or unsuitable materials must be removed prior to constructing the slab on grade or placement of new fill for grade raise. These materials do not constitute an adequate subgrade for support of a slab on grade. After any unsuitable materials are removed, the exposed soil which must not be proof rolled as this will disturb the sand subgrade, must be inspected, and approved by Terraprobe at the time of construction. Any structural fill placed to raise grades, must be placed in maximum 150 mm thick lifts, and compacted to at least 98 percent Standard Proctor Maximum Dry Density (SPMDD).

The modulus of subgrade reaction appropriate for slab on grade design on the compacted earth fill or native sand is 18,000 kPa/m.

It is necessary that building floor slabs be provided with a capillary moisture barrier and drainage layer. This is accomplished by placing the slab on a minimum 200 mm layer of 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of the 200 mm drainage layer may be replaced with 50 mm of Granular A (OPSS.MUNI 1010) to provide a trafficable surface.

Perimeter drainage is not needed where the lowest finished floor is at least 300 mm higher than exterior grade. Drainage of the subgrade and frost-resistant backfill is required where doors are flush to grade or in areas where the finished floor is less than 300 mm higher than grade. This is to prevent impaired door function during winter months.

It should be noted that if ground improvement of the site soils is carried out, these soils would also be suitable for supporting the building slab-on-grade.



5.5 Site Servicing

5.5.1 Bedding

In general, the existing native sand soils and new fill compacted to a minimum of 98 percent SPMDD will provide adequate support for buried services on a conventional well graded granular base material. Where disturbance of the trench base has occurred, such as due to groundwater seepage or construction traffic, the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill.

Granular bedding material should consist of a conventional Class 'B' bedding, such as Granular 'A', conforming to OPSS.MUNI 1010 specification. The use of HL 8 coarse aggregate or 19 mm clear stone (OPSS.MUNI 1004) bedding is also acceptable, however only in conjunction with a suitable geotextile filter (Terrafix 360R or equivalent); otherwise without proper filtering, there may be entry of fines from the surrounding soils into the subfloor drainage layer. This loss of ground could result in a loss of support to the pipes and possible future settlements. The bedding material should be compacted to a minimum of 98 percent SPMDD. Clear stone type bedding material should be vibrated to a dense state. The bedding material should conform to the pertinent City and Region Specifications.

5.5.2 Backfill

In general, excavated soils encountered on site may be re-used as backfill, provided the moisture content of these materials is within 2% of optimum to ensure adequate compaction, and the trenches are wide enough to accommodate a large compaction roller, and the soil is free of any deleterious material (e.g., peat, topsoil, debris). Any native material with higher in-situ moisture content could be put aside to dry or be tilled to reduce the moisture content so that it can be effectively compacted. Alternatively, materials of higher moisture content could be wasted and replaced with imported material which can be readily compacted.

The backfill should consist of clean earth and should be placed in lifts of 150 mm thickness or less and compacted to a minimum of 95 percent SPMDD (in non-settlement sensitive areas) and 98 percent SPMDD (in settlement sensitive areas) at a water content within 2% of optimum. Existing native soils will be difficult to place and compact successfully in narrow trench excavations, where large compaction equipment could not operate. For narrow trench excavations, it is recommended that free draining granular material, such as OPSS.MUNI 1010 Granular 'B' be used in order to allow for adequate compaction using vibratory equipment. The placement and inspection of any earth fill as backfill must be conducted under the full-time observation of the geotechnical engineer.

It should be noted that most of the shallow native soils at the site are not free draining and will be difficult to handle and compact if they become wet as a result of inclement weather or seepage. Hence, it can be expected that earthworks carried out during wet periods (i.e., spring and fall) of the year may result in increased earthwork costs.



6.0 PAVEMENT DESIGN

6.1 Subgrade Preparation

New compacted earth fill, and native soil are considered to be an adequate subgrade for the support of a pavement structure provided the subgrade is approved by Terraprobe at the time of construction, and does not contain excessive amounts of organics, moisture, or deleterious materials. The vertical modulus of subgrade reaction on the compacted earth fill or native sand is 18,000 kPa/m.

The subgrade must be exposed by the removal of any existing deleterious materials or soil with excessive amounts of organics. The pavement subgrade should be proof rolled and inspected by Terraprobe. Any loose, soft, wet, or unstable areas, or areas containing excessive organic materials should be sub-excavated and backfilled with clean earth fill placed in 150 mm thick lifts and compacted to a minimum of 98 percent SPMDD. Some parts of the native soils may require localized sub-excavation and re-compaction to support the pavement structure, as identified by Terraprobe during the subgrade inspection and preparation. These areas must be sub-excavated and backfilled with clean, approved, and compacted earth fill as noted above, and under the supervision of Terraprobe.

The granular materials should be placed in lifts 150 mm thick or less and compacted to a minimum of 100 percent and 98 percent SPMDD for granular base and granular sub-base, respectively. Recommendations are made for flexible asphaltic concrete pavements.

The unheated areas, such as concrete surface sidewalk and entrance slabs (near flush-doors), should be supported on a minimum of 1.5 metres thick non-frost susceptible material (19 mm clear stone separated by a geotextile) provided with a subdrain with positive outlet to help minimize slab heave due to freezing weather conditions.

6.2 Drainage

Control of surface water is an important factor in achieving a good pavement life. The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (preferably at a minimum grade of 2 percent) to provide effective drainage toward subgrade drains. Grading adjacent to pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement.

Continuous pavement subdrains should be provided along any parking lot/driveway pavement areas along the curb-lines/sidewalk, and at all catchbasins within the parking areas and drained into respective catchbasins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 metres below subgrade level. Subdrains should consist of a 150 mm diameter perforated pipe with factory installed filter fabric placed in a trench excavated (300 mm by 300 mm size)



into the subgrade. The trench should be backfilled with 19 mm clear stone wrapped with geotextile. Pavement drainage details are provided in Appendix D.

6.3 Pavement Structure

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions depending on actual traffic volumes. Traffic data was not provided to Terraprobe for review in preparation of this report. The following pavement thickness design is provided on the above noted considerations and subgrade basis.

Performance Asphaltic Concrete Pavement Structure

Pavement Layer	Compaction Requirements	Light-Duty Minimum Component Thickness	Heavy-Duty Minimum Component Thickness
<i>Surface Course Asphaltic Concrete:</i> HL3 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS 1101)	OPSS 310	40 mm	40 mm
<i>Base Course Asphaltic Concrete:</i> HL8 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS 1101)	OPSS 310	60 mm	80 mm
<i>Base Course:</i> Granular A (OPSS.MUNI 1010) or 19mm Crusher Run Limestone	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
<i>Subbase Course:</i> Granular B Type I (OPSS.MUNI 1010) or 50mm Crusher Run Limestone	100% Standard Proctor Maximum Dry Density (ASTM- D698)	250 mm	350 mm

A minimal pavement design is also provided, which will provide an estimated service period of about 8 to 10 years depending on actual traffic volumes. The cost of the minimal pavement design should be compared to the performance design which could be expected to last about twice as long before significant maintenance and rehabilitation.

Minimal Asphaltic Concrete Pavement Structure

Pavement Layer	Compaction Requirements	Light-Duty Minimum Component Thickness	Heavy-Duty Minimum Component Thickness
<i>Surface Course Asphaltic Concrete:</i> HL3 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS 1101)	OPSS 310	65 mm	40 mm
<i>Base Course Asphaltic Concrete:</i> HL8 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS 1101)	OPSS 310	N/A	50 mm



Pavement Layer	Compaction Requirements	Light-Duty Minimum Component Thickness	Heavy-Duty Minimum Component Thickness
<i>Base Course:</i> Granular A (OPSS.MUNI 1010) or 19mm Crusher Run Limestone	100% Standard Proctor Maximum Dry Density (ASTM-D698)	225 mm	325 mm

7.0 DESIGN CONSIDERATION FOR CONSTRUCTABILITY

7.1 Excavations

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III - Excavations, Section 222 through 242. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety.

For practical purposes, the earth fill, peat, and native sand are Type 3 soil above and Type 4 soil below the prevailing groundwater table.

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates safe slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the Act and Regulations and include provisions for timbering, shoring and moveable trench boxes.

The subsurface soils can be removed by conventional excavation equipment. Excavations at this site may encounter construction debris, deleterious materials, and other obstructions in the earth fill. Larger size particles (cobbles and boulders) that are not specifically identified in the boreholes may be present in the native soils. The size and distribution of cobbles/boulders/obstructions cannot be predicted with boreholes, as the sampler size is insufficient to secure representative particles of this size. The risk and responsibility for the removal and disposal of cobbles/boulders/obstructions must be addressed in the contract documents for foundations and excavations contractors.



The measurements of water levels and caving in open boreholes upon completion of drilling was measured and presented in the Borehole Logs. Monitoring wells comprising 50 mm diameter PVC pipes were installed in six selected borehole to facilitate Groundwater monitoring and sampling. The PVC tubing was fitted with a bentonite clay seal as shown on the accompanying Borehole Logs. Water levels in the monitoring wells were measured on February 1, 2022. The results of Groundwater monitoring are presented in Section 4.6 of this report.

Groundwater levels may fluctuate with time, and seasonally, depending on the amount of precipitation and surface runoff.

It is anticipated that excavations for footing foundations and site services may extend below the prevailing groundwater table. The wet native sand materials are considered to be moderate to high permeability soils and free flow of water into the excavations should be expected below the groundwater table. If seepage is encountered, it is expected to be of limited extent and can be controlled using a conventional sump pump system when above the water table. More significant, positive dewatering will be required where excavations are to extend below the encountered groundwater level in sandy soils.

Refer to the hydrogeological report issued under separate cover for comments regarding groundwater volumes for the site.

7.2 Site Work

The soils found at this site will become weakened when subjected to traffic, particularly when wet. If there is site work carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil and use of granular fill material for site restoration or underfloor fill that is not intrinsic to the project requirements.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during paving and other work may be required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil at this site is highly susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.



Prior to pouring concrete for the footings, the footing subgrade must be cleaned of deleterious materials, softened, disturbed, or caved materials, and any standing water. The footing excavations must be inspected and approved by Terraprobe to ensure the bearing capacities stated above are applicable. If soft soils are encountered at the proposed bearing depths during footing excavation, sub-excavation to competent soil is required under the direction of the geotechnical engineer. Furthermore, native soils tend to weather and deteriorate on exposure to the atmosphere or to surface water, therefore foundation bases that will remain open and exposed to the atmosphere for an extended period of time shall be protected by applying a skim coat of lean concrete. If construction is to proceed in freezing conditions, temporary frost protection for the footing bases and concrete must be provided.

7.3 Quality Control

The proposed structure will be founded on spread footing foundations. The foundation installations must be field reviewed by the geotechnical engineer as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by the current Ontario Building Code. If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice contained in this report.

The long-term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the borings. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness and to ensure adequate compaction.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density as determined by ASTM D698. In situ determinations of density during fill and asphaltic placement on site are required to demonstrate that the specified placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.



Terraprobe provides quality control services for Building Envelope, Roofing and Structural Steel, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1-1996.

8.0 LIMITATIONS AND USE OF REPORT

8.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The drilling work was carried out by a drilling contractor and was observed and recorded by Terraprobe on a full-time basis. The boreholes were made by a continuous flight power auger machine using solid and hollow stem augers. A Terraprobe technician logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. Groundwater observations were made in the boreholes as drilling proceeded.

The samples of the strata penetrated were obtained using the technique, Split-Barrel Method, ASTM D1586. The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. A comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment, and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.



The subsurface soil and groundwater conditions encountered in the boreholes are presented on the attached Borehole Logs. The stratigraphic boundaries indicated on the Borehole Logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes and will vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design.

8.2 Changes in Site and Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection.

The design parameters provided, and the engineering advice offered in this report is based on the factual data obtained from this investigation made at the site by Terraprobe and is intended for use by the owner and its retained design consultants in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters, advice, and comments relating to constructability issues and quality control may not be relevant or complete for the project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

8.3 Use of Report

This report is prepared for the express use of 25707517 Ontario Inc. and their retained design consultants. It is not for use by others. This report is copyright of Terraprobe Inc., and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe, 25707517 Ontario Inc. and their retained design consultants are authorized users.

It is recognized that municipal/regional governing bodies, in their capacity as the planning and building authority under Provincial statutes, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.



We trust this report meets with your requirements. Should you have any questions regarding the information presented, please do not hesitate to contact our office.

Sincerely,

Terraprobe Inc.



Mike Tanos, P.Eng.
Review Principal

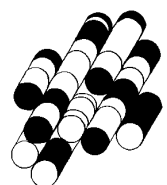


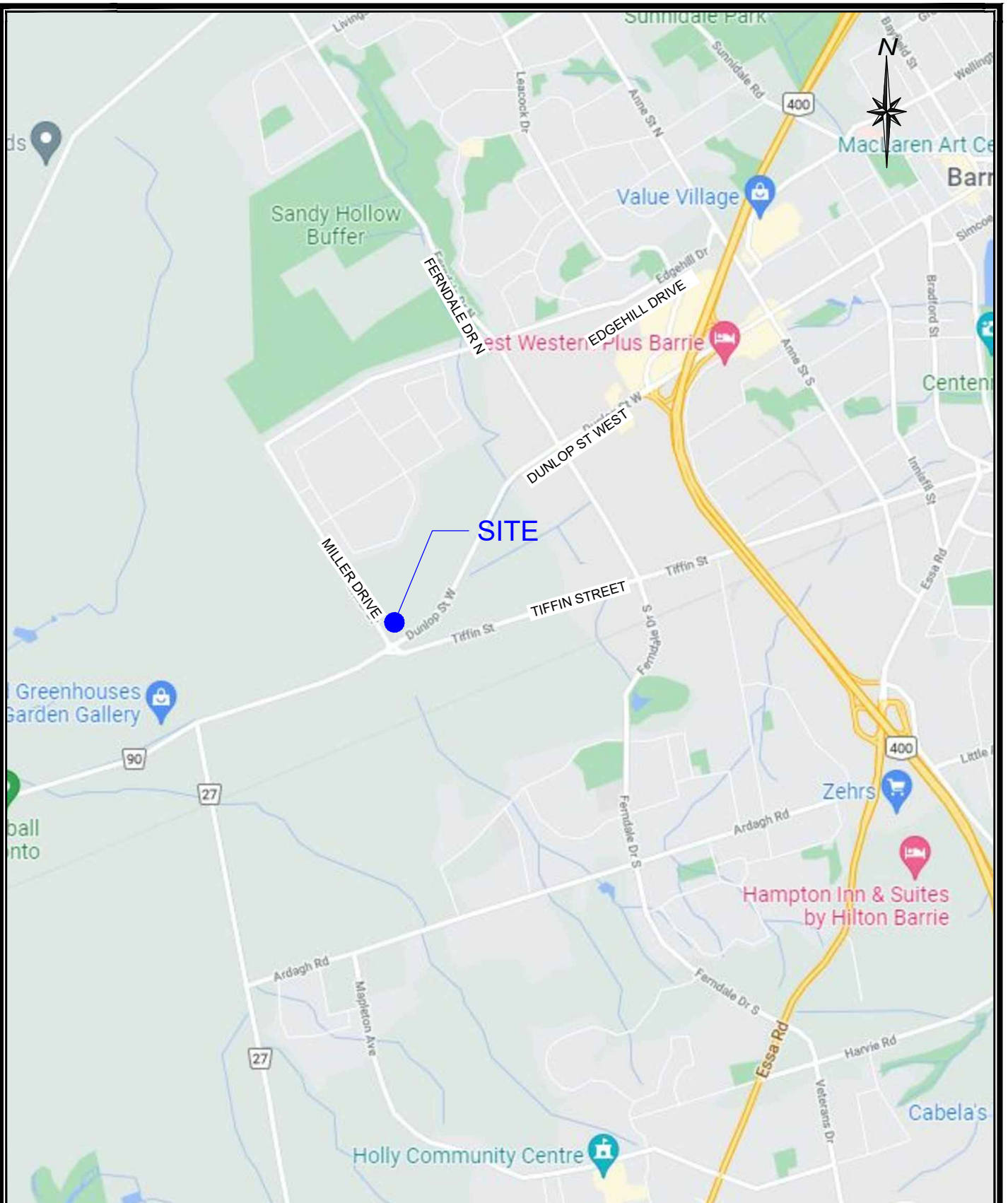
Osbert (Ozzie) Benjamin, P.Eng.
Senior Project Manager, Geotechnical



FIGURES

TERRAPROBE INC.





REFERENCE
Image ©2022 Google Earth

SCALE 200 0 500m



Terraprobe

11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SITE LOCATION PLAN

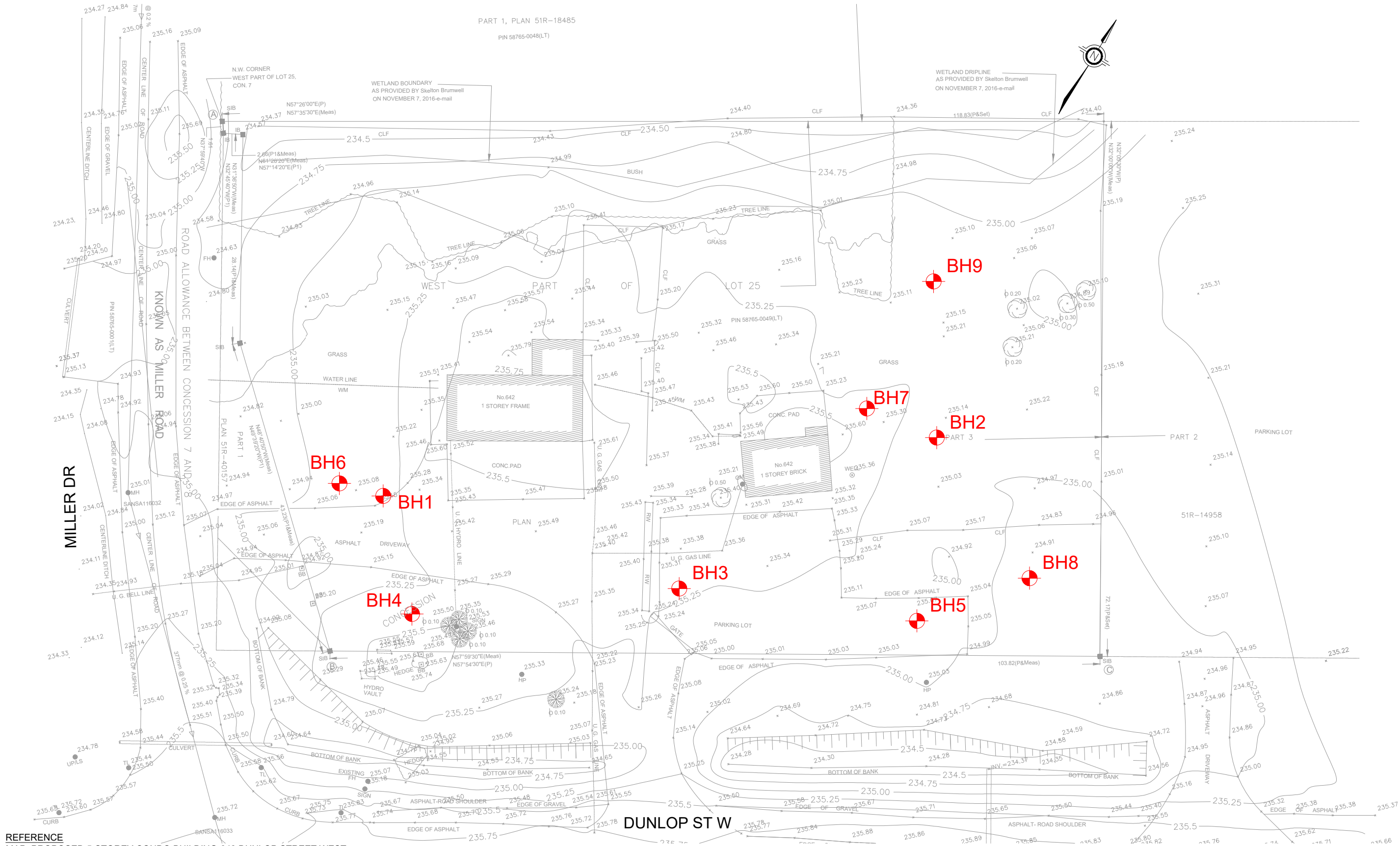
File No.:

3-21-0182-01

FIGURE :

1

Y:\Shared\Terraprobe\Brampton\1-Project Files\2021\Branch Folder\3-21-0182 - 642 Dunlop Street West, Barrie\01 - Geotechnical Investigation\A. Dwg. Logs\AutoCAD\3-21-0182-FIG2.dwg
3/8/2022 9:02:41 AM, DWG TO PDF, pc3



REFERENCE
MAP: PROPOSED 5-STOREY CONDO BUILDING-642 DUNLOP STREET WEST
SITE PLAN
PROJECT NUMBER: 15-48 DRAWING NUMBER: A 1.0
DATE UPDATED: FEB 4, 2022
COMPANY: 'n' ARCHITECTURE INC.
SURVEY INFORMATION: OMARI MWINYI SURVEYING LTD.

LEGEND

⊕ Approximate Borehole Location



Title:

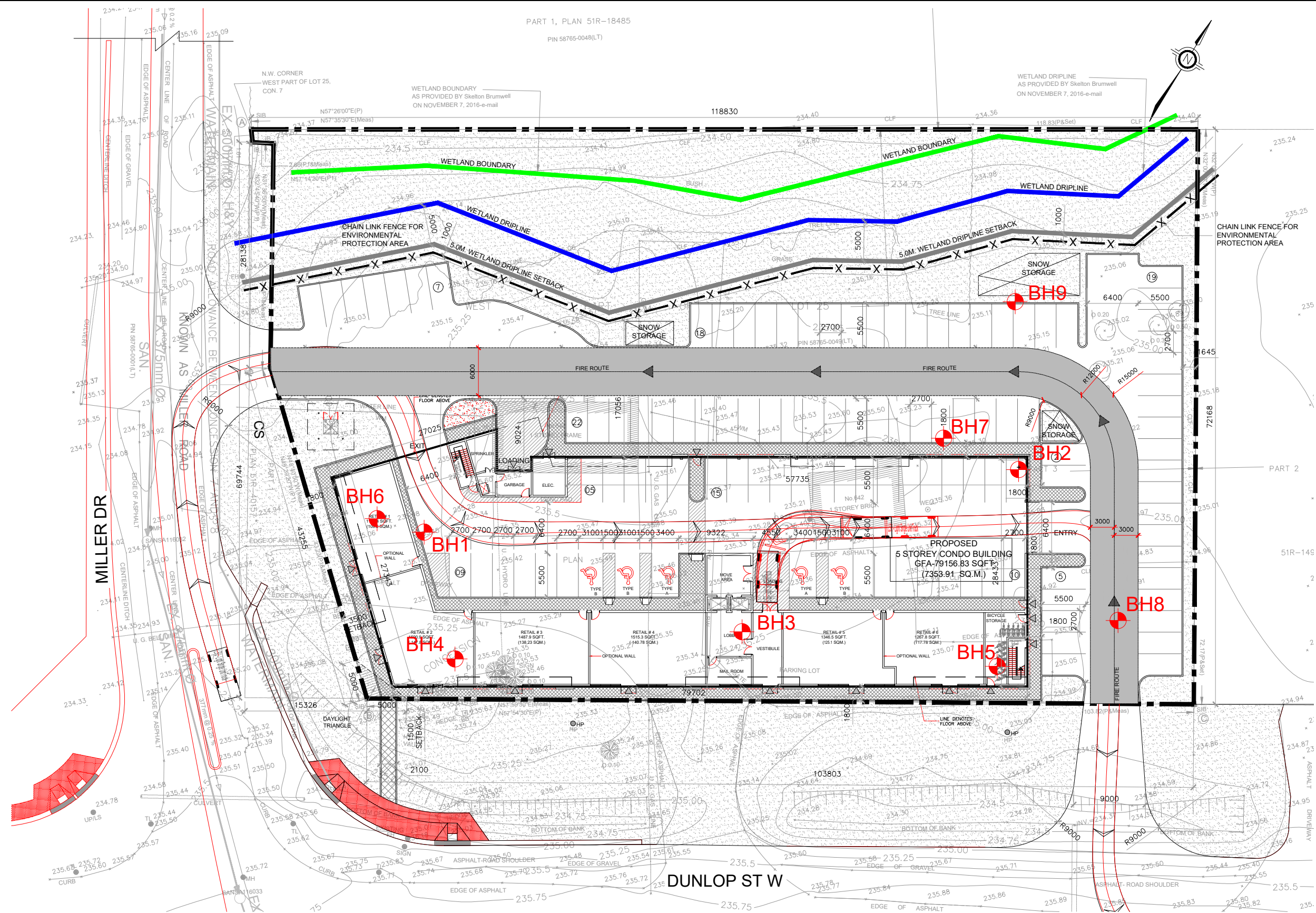
**BOREHOLE LOCATION PLAN
Existing Condition**

File No.

3-21-0182-01

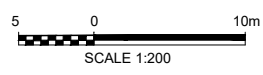
FIGURE :

2A



REFERENCE
MAP: PROPOSED 5-STOREY CONDO BUILDING-642 DUNLOP STREET WEST
SITE PLAN
PROJECT NUMBER: 15-48 DRAWING NUMBER: A 1.0
DATE UPDATED: FEB 4, 2022
COMPANY: 'n' ARCHITECTURE INC.
SURVEY INFORMATION: OMARI MWINYI SURVEYING LTD.

LEGEND
● Approximate Borehole Location



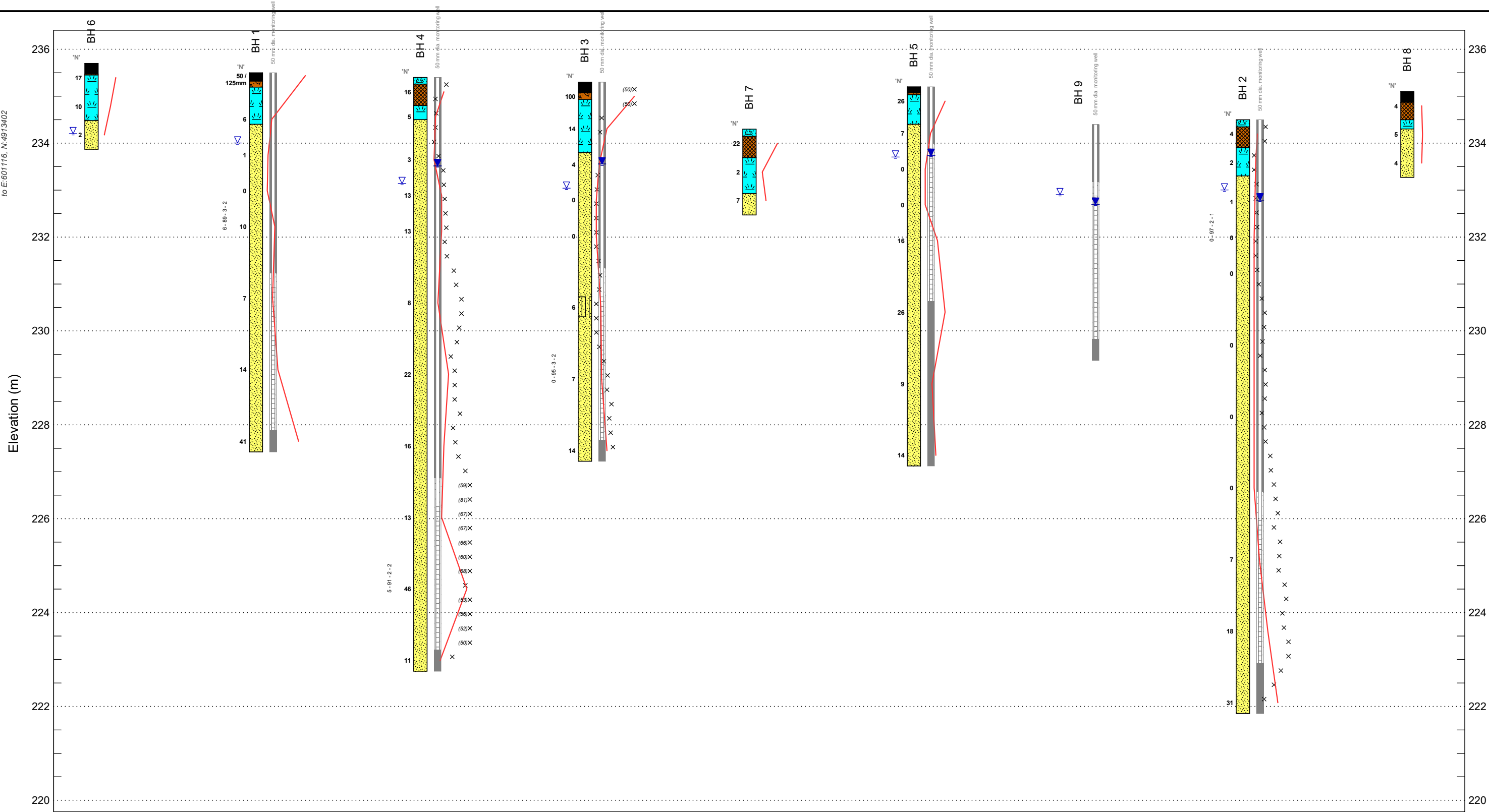
**Terraprobe**
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: BOREHOLE LOCATION PLAN Proposed Condition	
File No.	3-21-0182-01

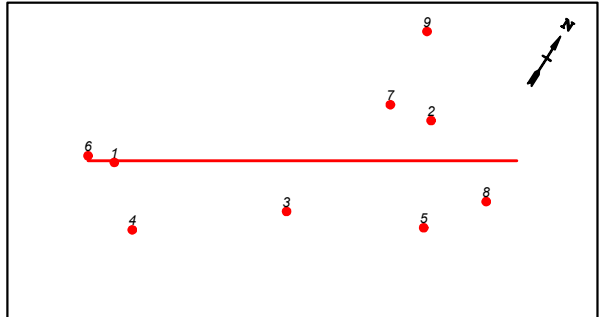
FIGURE :
2B

Y:\Shared\Terraprobe\Brampton\1-Project Files\2021\Branch Folder\3-21-0182-642 Dunlop Street West, Borehole - Geotechnical Investigation\A. Dwg, Logs\AutoCAD\3-21-0182-FIG2.dwg
3/8/2022 10:01:15 AM DWG To PDF.pc3

Alignment From E:601032, N:4913348,
to E:601116, N:4913402



SITE MAP



LITHOLOGY GRAPHIC LEGEND

- | | |
|-----------|------------|
| Asphalt | Topsoil |
| Aggregate | Fill |
| Peat | Sandy Silt |
| Sand | |

Boreholes Equally Spaced

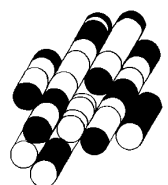
INTERPRETIVE LEGEND

- WL on completion of drilling
Stabilized WL, most recent

- | | |
|-----------------------------------|---|
| FILL | COHESIONLESS TILLS |
| GRAVELS (gravel to gravelly sand) | COHESIVE SOILS (clayey silt to clay, incl. tills) |
| SILT TO SAND (not till) | TOPSOIL/PEAT |

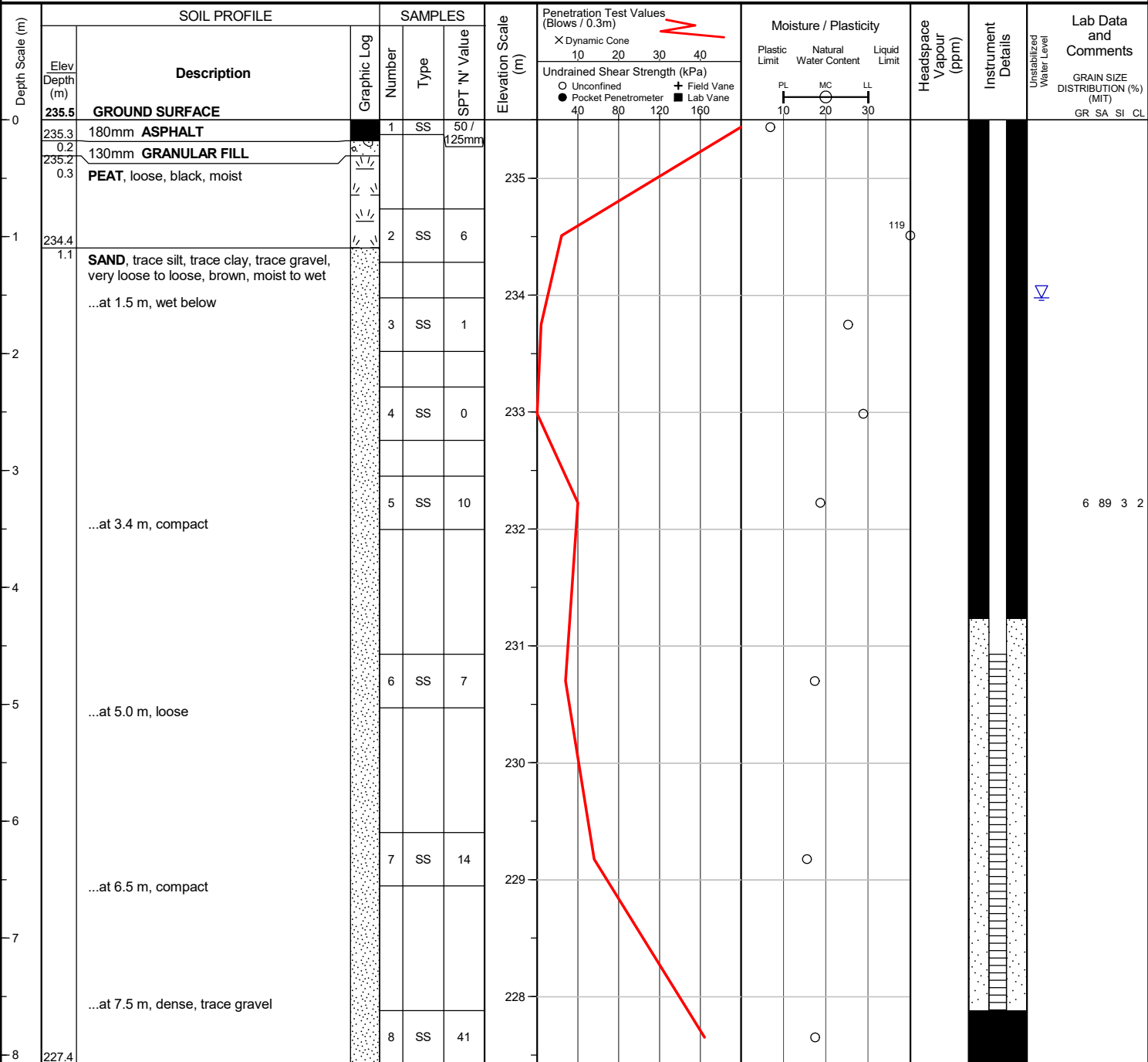
APPENDIX A

TERRAPROBE INC.



Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : SO
 Date started : January 19, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601037, N: 4913351 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers



WATER LEVEL READINGS
 Date: Feb 1, 2022 Water Depth (m): n/a Elevation (m): n/a
 Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.
 50 mm dia. monitoring well installed.



Originated by : CB

Compiled by : GO

Checked by : OB

Drilling Method : Hollow stem augers

file: 3-21-0182-01 bh logs.gpj

(continued next page)

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 20, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 2 of 2	Location : Barrie, Ontario	Checked by : OB

Position : E: 601094, N: 4913399 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone Undrained Shear Strength (kPa) O Unconfined ● Pocket Penetrometer	Moisture / Plasticity Plastic Limit Natural Water Content Liquid Limit PL MC LL	Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value						
		(continued)										
		SAND, trace silt, trace clay, trace gravel, very loose, brown, moist to wet (continued)					224					
11		...at 11.0 m, compact		10	SS	18						
12							223					
				11	SS	31	222					
221.9 12.6												

END OF BOREHOLE

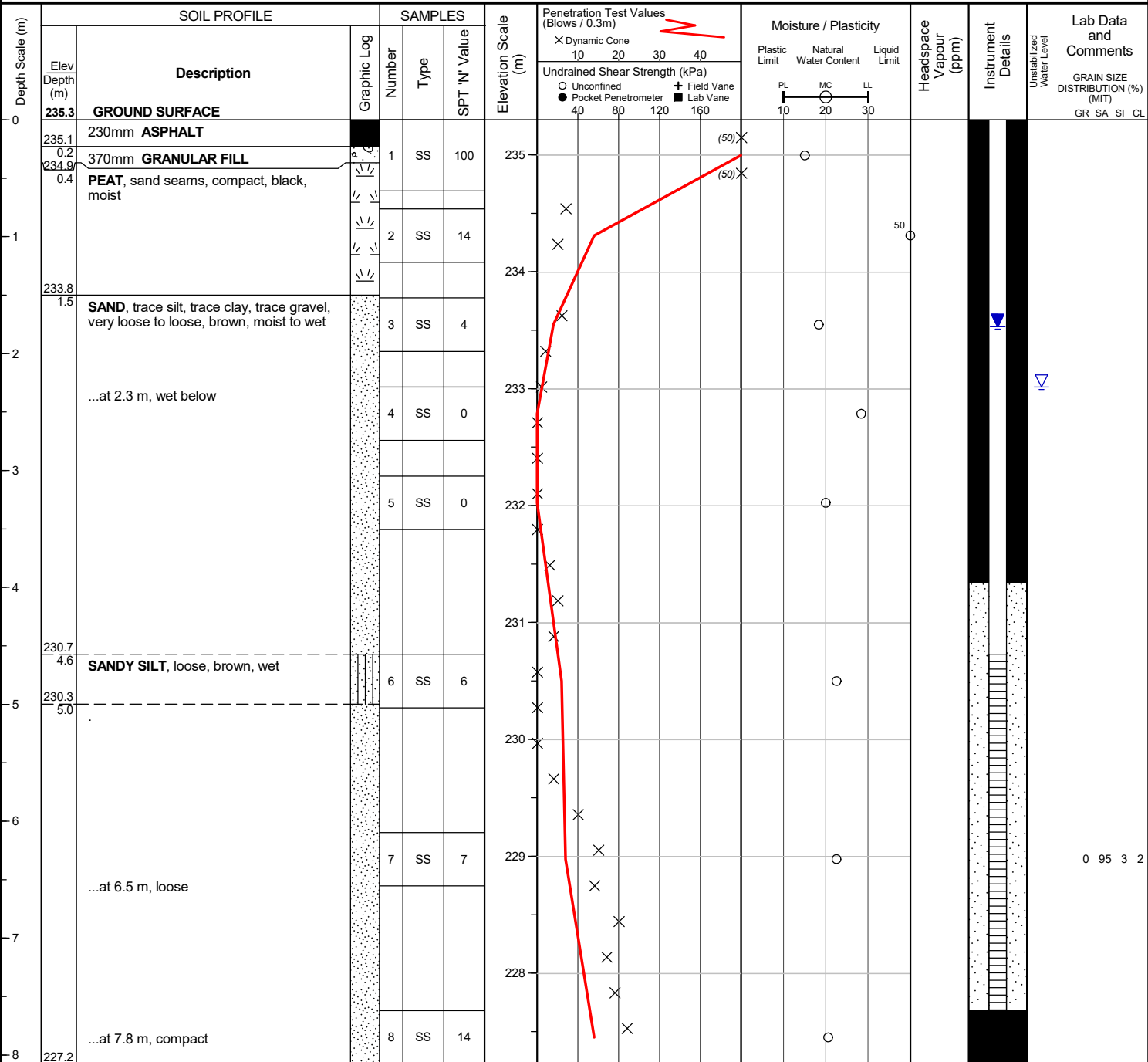
Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS		
Date	Water Depth (m)	Elevation (m)
Feb 1, 2022	1.7	232.8

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601077, N: 4913363 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers



WATER LEVEL READINGS
 Date Feb 1, 2022 Water Depth (m) 1.8 Elevation (m) 233.5

Unstabilized water level measured at 2.3 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.



Originated by : SO

Compiled by : GO

Checked by : OB

Drilling Method : Hollow stem augers

file: 3-21-0182-01 bh logs.gpj

(continued next page)

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : SO
Date started : January 19, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 2 of 2	Location : Barrie, Ontario	Checked by : OB

Position : E: 601049, N: 4913340 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone Undrained Shear Strength (kPa) ○ Unconfined ● Pocket Penetrometer	Moisture / Plasticity Plastic Limit Natural Water Content Liquid Limit PL MC LL	Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value						
		(continued)										
11		SAND, trace silt, trace clay, trace gravel, loose to compact, brown, moist to wet (continued)					225	(60)X				
		...at 10.8 m, dense		10	SS	46		(68)X				
12							224	(53)X				
								(56)X				
								(52)X				
								(50)X				
				11	SS	11	223	X				disturbed sample
222.8 12.6												

END OF BOREHOLE

Unstabilized water level measured at 2.3 m below ground surface upon completion of drilling.

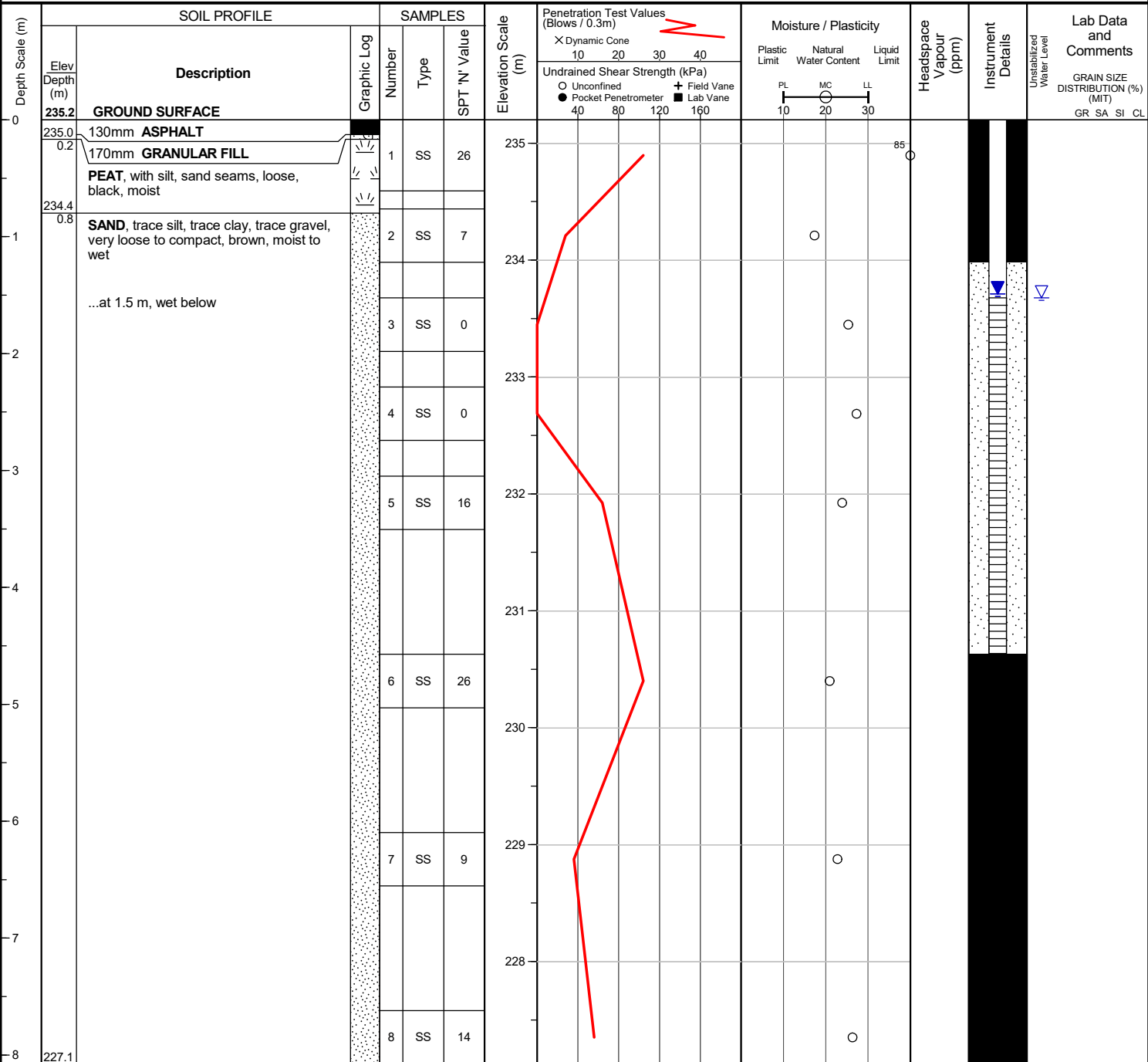
50 mm dia. monitoring well installed.

WATER LEVEL READINGS

<u>Date</u>	<u>Water Depth (m)</u>	<u>Elevation (m)</u>
Feb 1, 2022	1.9	233.5

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 19, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601106, N: 4913377 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers



Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

<u>Date</u>	<u>Water Depth (m)</u>	<u>Elevation (m)</u>
Feb 1, 2022	1.5	233.7

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601031, N: 4913349 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		X Dynamic Cone	Undrained Shear Strength (kPa)	Plastic Limit	Natural Water Content	Liquid Limit			
0	235.7	GROUND SURFACE													
	235.5	230mm ASPHALT		1	SS	17							86		
	235.5	250mm GRANULAR FILL													
	235.5	PEAT, compact, black, moist		2	SS	10							132		
	234.5														
	234.5	SAND, trace silt, trace clay, trace gravel, trace organics, very loose, brown, moist to wet		3	SS	2									
	233.9														
	1.8														

END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface; borehole was open upon completion of drilling.

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601084, N: 4913397 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		Undrained Shear Strength (kPa)		Plastic Limit	Natural Water Content	Liquid Limit			
0	234.3	GROUND SURFACE						X Dynamic Cone							
	234.1	150mm TOPSOIL						O Unconfined + Field Vane							
	233.7	FILL, peat/ sandy silt, some gravel, compact, black, moist		1	SS	22	234	● Pocket Penetrometer ■ Lab Vane							
	233.7	PEAT, sand seams,, very loose, black, wet		2	SS	2		40 80 120 160		PL	MC	LL			
	232.9									10	20	30			
	232.5	SAND, trace silt, trace clay, trace gravel, trace organics, loose, brown, moist		3	SS	7	233								
	1.8														

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601115, N: 4913390 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		X Dynamic Cone	Undrained Shear Strength (kPa)	Plastic Limit	Natural Water Content	Liquid Limit			
0	235.1	GROUND SURFACE					235								
	234.9	230mm ASPHALT		1	SS	4									
	0.2	FILL, organics/peat, some gravel, sand, loose, black/brown, moist													
	234.5	PEAT, sand seams, loose, black, moist													
	0.6														
	234.3	SAND, trace silt, trace clay, trace gravel, some organics, loose, brown, moist		2	SS	5									
	0.8														
1				3	SS	4									
	233.3														
	1.8														

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 18, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601082, N: 4913416 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone 10 20 30 40 Undrained Shear Strength (kPa) ○ Unconfined + Field Vane ● Pocket Penetrometer ■ Lab Vane 40 80 120 160	Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments		
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value			Plastic Limit	Natural Water Content	Liquid Limit					
0	234.4	GROUND SURFACE														
1		Borehole augered to 5m depth without sampling for installation of monitoring well														
2																
3																
4																
5	229.4															

END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface; borehole was open upon completion of drilling.

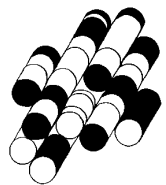
50 mm dia. monitoring well installed.

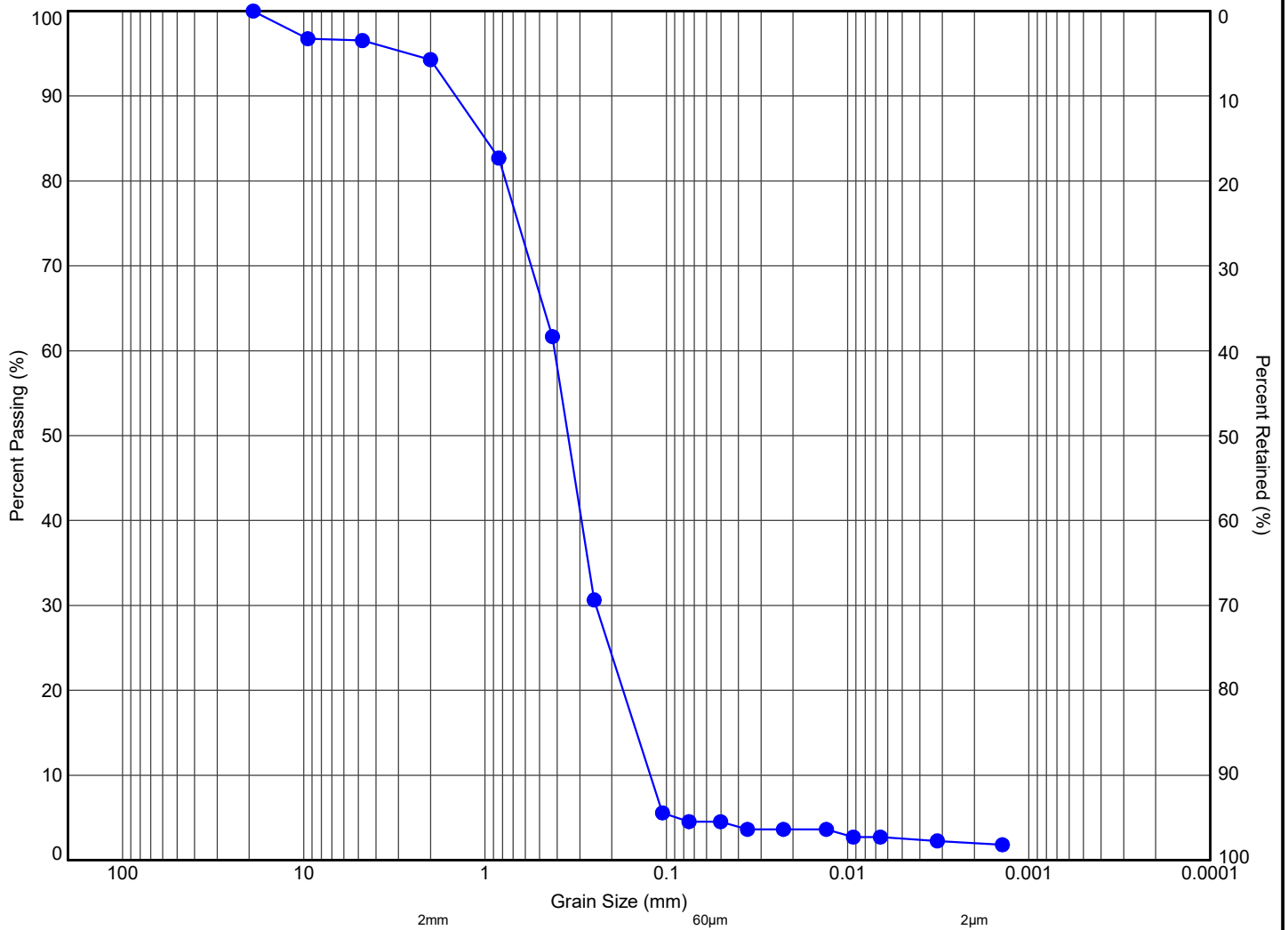
WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Feb 1, 2022	1.7	232.7

APPENDIX B

TERRAPROBE INC.





MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● 1	SS5	3.3	232.2	6	89	3	2		



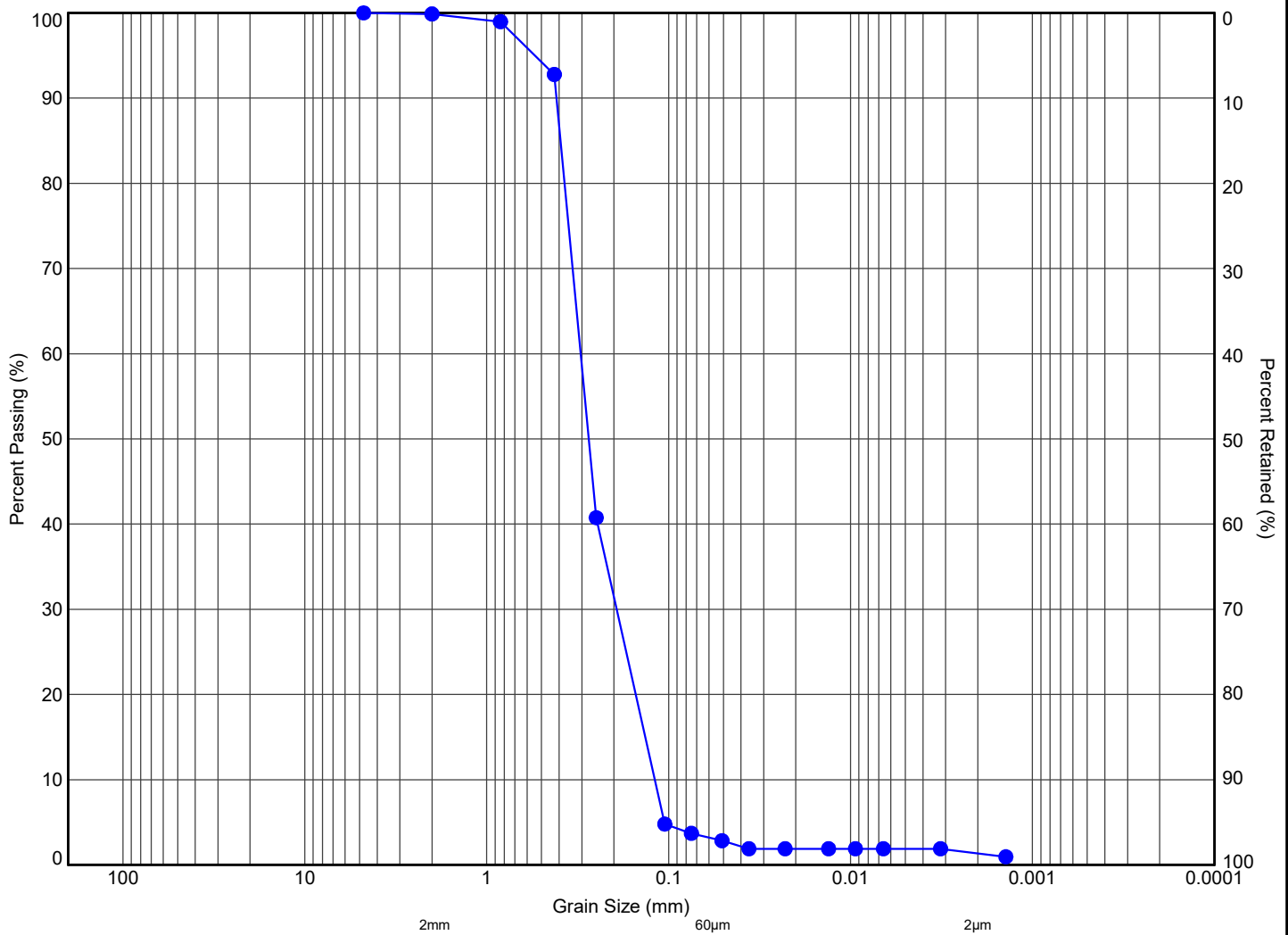
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM

Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)
2	SS4	2.5	232.0	0	97	2	1	



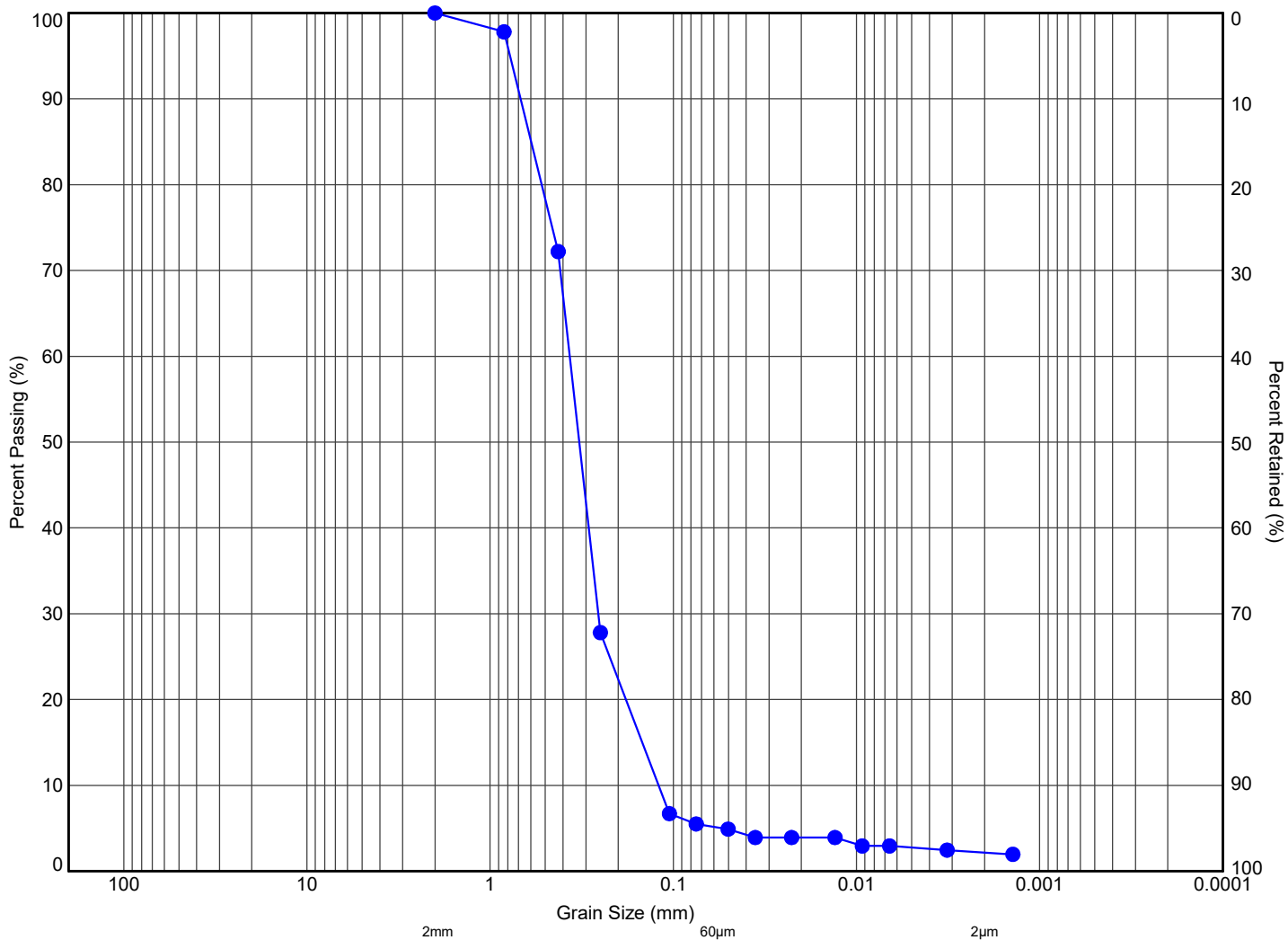
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(905) 796-2650

Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM

Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)
3	SS7	6.3	229.0	0	95	3	2	



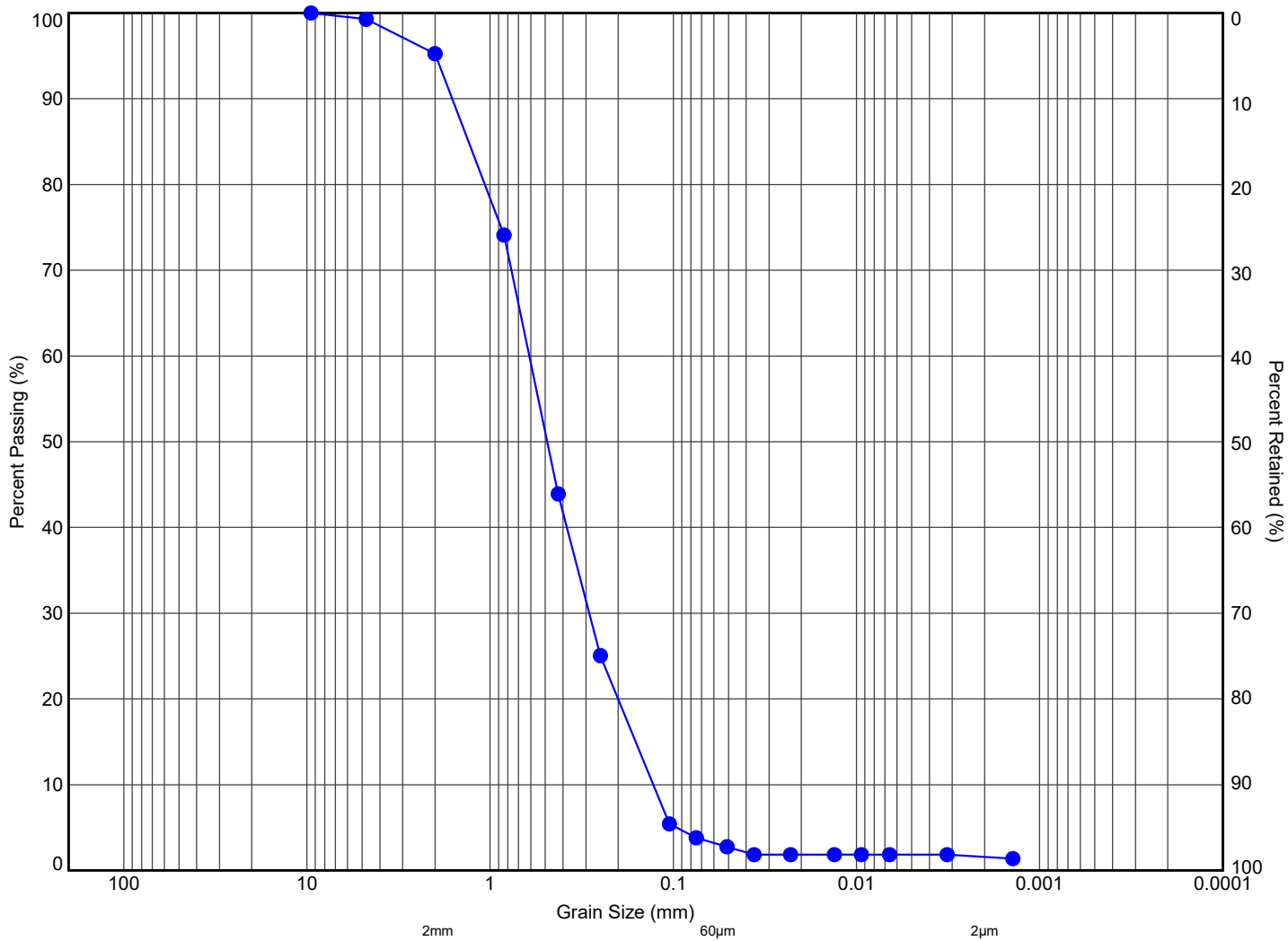
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Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● 4	SS10	10.9	224.5	5	91	2	2		



Terraprobe

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Title:

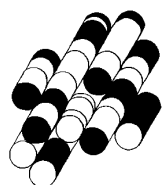
**GRAIN SIZE DISTRIBUTION
SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01

APPENDIX C

TERRAPROBE INC.





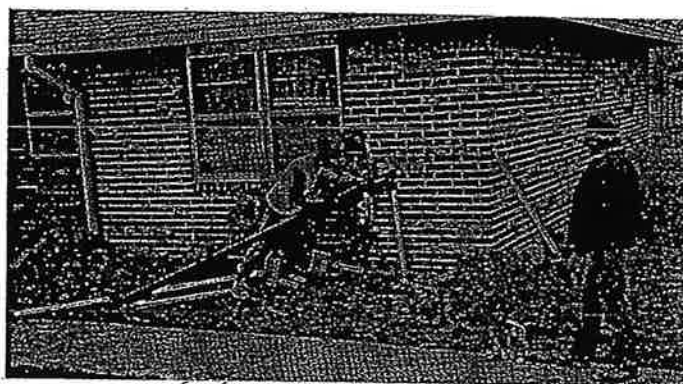
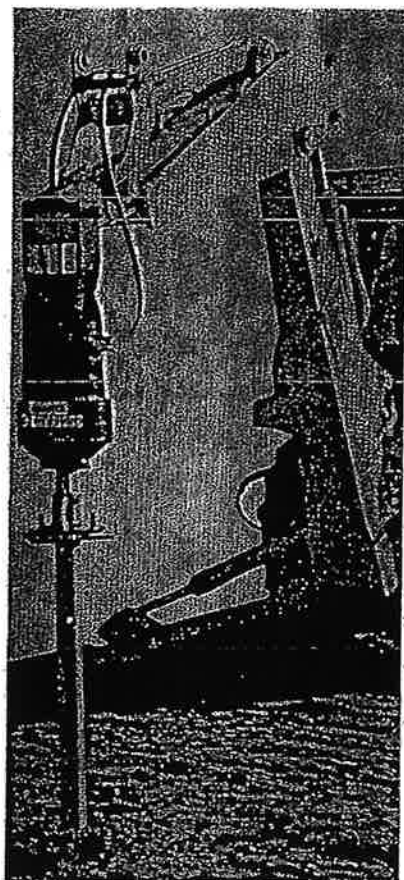
POWER SYSTEMS

CHANCE

A109
02150/CHA
BuyLine 1386

HELICAL-PIER® FOUNDATION SYSTEM

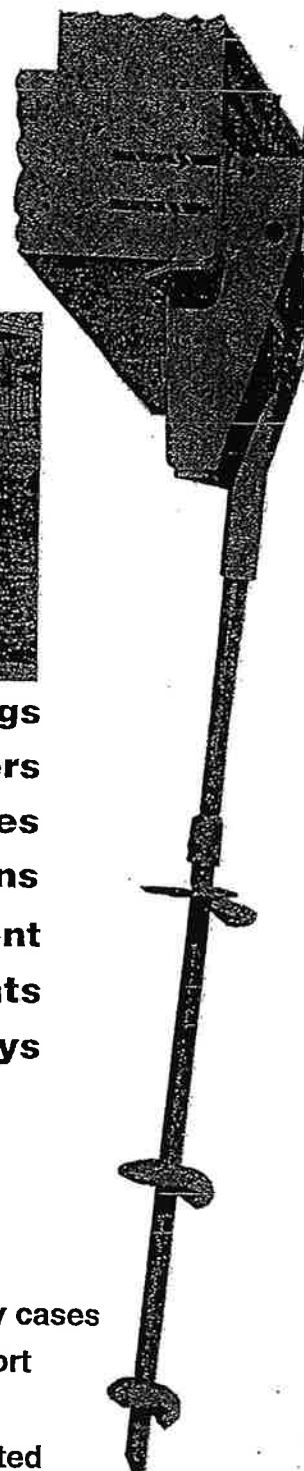
**Saves days and dollars compared to concrete piers,
spread footings or other pile methods**



- Underpinning buildings
- New-construction piers
- Lifting existing structures
- Seismic applications
- Environment enhancement
- Retaining walls • Revetments
- Dams • Levees • Roadways

Immediate benefits:

- Predictable results
- One-trip convenience: No site preparation or concrete required in many cases
- Holds design loads in specific soils
- Installs in any weather and in limited-access situations
- Clean – no excavation spoils to remove
- Less equipment needed: No concrete trucks or grout pumps
- Lower installed costs
- Easy to store and transport
- Reusable
- U.S. Patented



WORLDWIDE EXPERIENCE IN HELICAL-PIER® SYSTEMS SINCE 1907

PRODUCT SPECIFICATION (continued)

Specification References

Details on the Chance Helical Pier® Foundation Support System are available upon request in the three-part section Manu-Spec® format (of the Spec-Data® program copyrighted by The Construction Specifications Institute). Filed under the identical 02150 designation as Sweet's, highlights include:

Part 1 — General

1.01 Section includes: New and remedial building foundation piling and stabilization, retaining walls, tieback systems.

1.02 Related sections: Excavating to working level, load tests, cast-in-place concrete reinforcement.

1.03 References: Fifteen ASTM and one SAE standards specifications.

1.04 Definition of system

1.05 Quality Assurance: Dealer certification, code requirement.

Part 2 — Products

2.01 Manufacturer: Chance Co.

2.02 Manufactured components: Helical plate, pier shaft, bolts, steel bracket.

Part 3 — Execution

3.01 Examination: Verify site conditions under Sect. 01039.

3.02 Preparation: Spare nearby structures; varying elevations.

3.03 Installation: Certified contractor, connect to structure, torque power units, adapters, extensions, torque recording, proper alignment, revolutions per minute, down pressure, coupling bolts, obstructions, minimum depth and torque.

3.04 Field quality control: Inspection, field testing, torque monitoring.

3.05 Project record documents: Submit 8 data records per Sect. 01700.

3.06 Depth and torque tolerances: Maximum torque prior to minimum depth, A/E approval, pier replacement.

Business Practices

Before each job by contractors certified to install the Chance Helical Pier® Foundation Support System, a quotation is prepared. Customarily, the bid for work is based on the amount to be billed per pier, access and final details required.



Light Duty Bracket

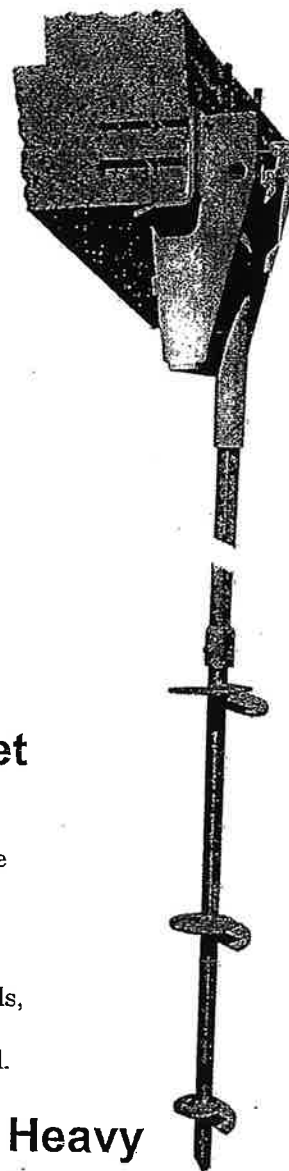
Primarily for correcting sagging lesser loads, affordable "quick fix" outlasts the porches, stairways, decks and patios it repairs.



Standard Duty Bracket

Applied in multiple locations along the foundation to stabilize and correct problems caused by poor soil conditions.

For seismic uplift loads, the Uplift Restraint Bracket may be added.



Heavy Duty Bracket

For such higher loads as commercial buildings and larger residences. Applied in multiples to stop settled areas, resist new movement.

All components are hot-dip galvanized to increase product life in aggressive soils.

PRODUCT SPECIFICATION

Lead and extension section lengths

The standard Helical Pier® foundation lead-section lengths are 5, 7, and 10 ft. The standard extension section lengths are 3½, 5, 7, and 10 ft. long. These combinations of leads and extensions provide for a variety of helical foundation lengths for each specific project.

Helix areas

Standard helices manufactured by the Chance Company are:

6 in. diameter = 26.7 sq. in.

8 in. diameter = 48.4 sq. in.

10 in. diameter = 76.4 sq. in.

12 in. diameter = 111.0 sq. in.

14 in. diameter = 151.0 sq. in.

Helix configuration

Standard helices are ¾ inch thick helically shaped steel plates with outer diameters of 6, 8, 10, 12 and 14 inches. The lead section, or first section installed into the soil, contains the helix plate(s). Multihelix foundations have more than one helix arranged in increasing diameters from the foundation tip to the uppermost helix. If more than one helix is used, the typical spacing between helical plates is three times the diameter of the next lower helix. For example, a helical foundation with an 8-, 10-, and 12- inch helix combination has a 24-inch space between the 8- and 10-inch helix and a 30-inch space between the 10- and 12-inch helix. Helix plates can be added to extension sections if necessary. Extensions with

helices should be installed immediately after the lead section.

Capacities listed in the Ratings Table here are system ratings. One must be aware that the actual installed load capacities are dependent on actual soil conditions at each

Minimum pier type required based on mechanical ratings

Design Load, kips	Minimum Pier Type Required
0 to 25	SS5
25 to 35	SS150
35 to 50	SS175 or HS

Note: This chart uses a factor of safety vs. ultimate capacity = 2.

System Ratings Table		Helical Pier® Family			
		*SS5	*SS150	SS175	HS
		1½" Square Shaft Column 1	1½" Square Shaft Column 2	1¾" Square Shaft Column 3	3½" OD Pipe Shaft Column 4
Allowable Torque Capacity (ft.-lbs.)	Row A	5,500	7,000	10,000	11,000
Allowable Capacity per Helix (Tension/Compression)	Row B	(1) 20 Kips	20 Kips	25 Kips	25 Kips
Ultimate Capacity per Helix (Tension/Compression)	Row C	(2) 40 Kips	(2) 40 Kips	(2) 50 Kips	(2) 50 Kips
Rated Capacity ^(4, 5) Underpinning - Retrofit	Row D	20 Kips	25 Kips	40 Kips	____ (3)
Ultimate Capacity ^(4, 5) Underpinning - Retrofit	Row E	40 Kips	40 Kips	80 Kips	____ (3)
Ultimate Capacity ⁽⁶⁾ for Axially Loaded Foundation (Torque Limited)	Row F	55 Kips	70 Kips	100 Kips	100 Kips

*SS150 shafts have paint stripe at top to distinguish from Type SS5.

(1) For 14"-dia. Helical Pier foundations, reduce allowable capacity by 20% per building code requirements.

(2) For 14"-dia. helices, reduce ultimate capacity by 20%.

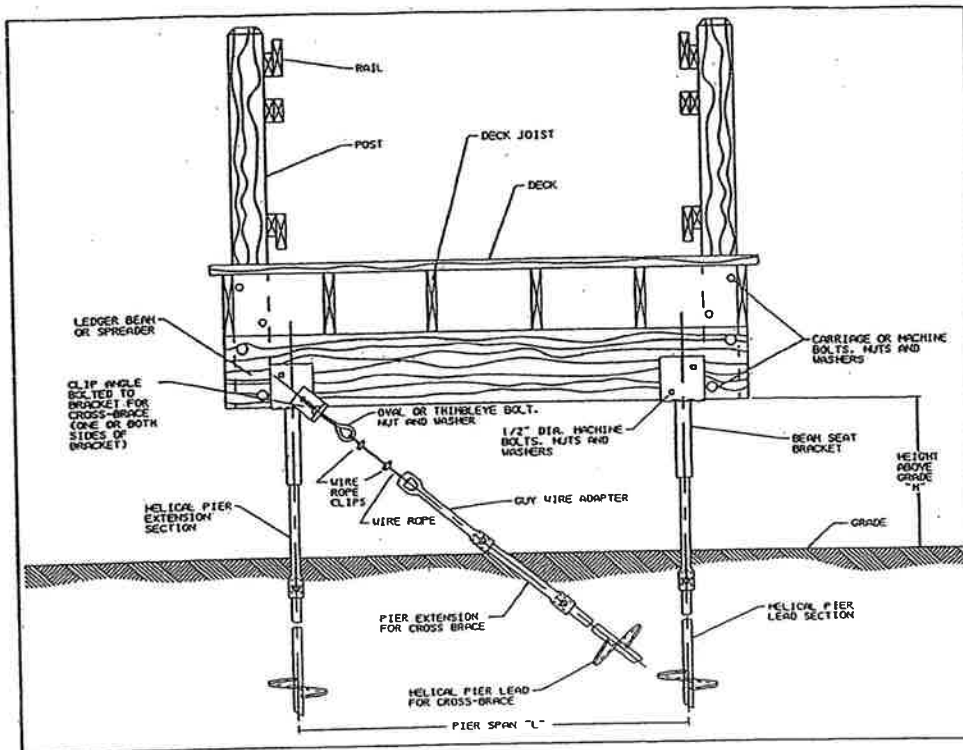
(3) Determined by bracket and haunch design.

(4) Anchor installed into and supported by soil with SPT blow count (N-value) greater than 4.

(5) Rating with Helical Pier Foundation bracket.

(6) Actual installed capacities are dependent on existing soil conditions.

Typical Walkway Hardware Application



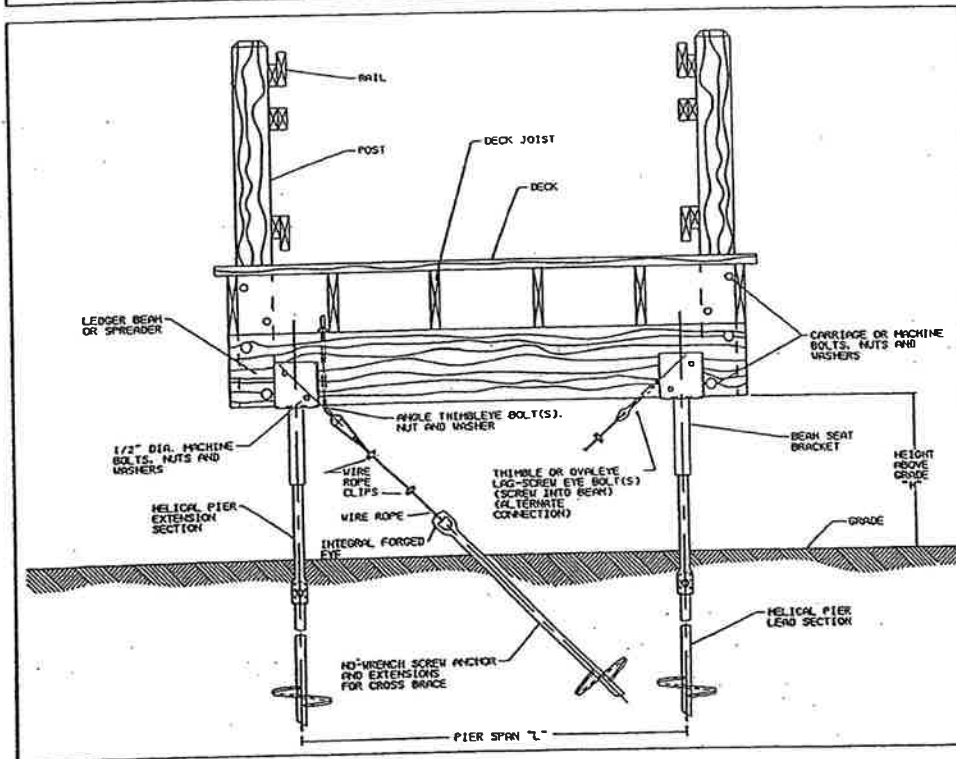
NOTE:

Where applicable, hardware manufactured by A. B. Chance Company is made from steel produced in accordance with ASTM A5 and/or ASTM A576. When galvanized, items are manufactured in accordance with ASTM A153.

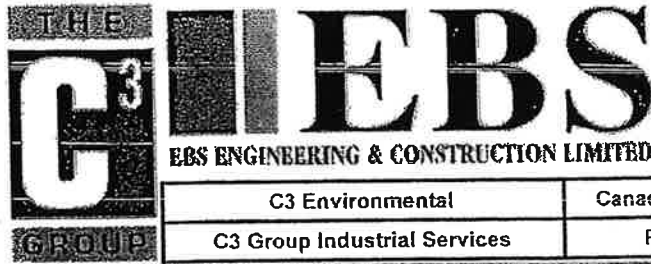
Where appropriate, these hardware items comply to the following ANSI specifications:

ANSI

C135.1
C135.2
C135.22
C135.14
C135.17



ON THE COVER: Aerial view of a 1½-mile boardwalk constructed at a wetlands site for the Fairfax County, Virginia Park Authority. No wheel or track vehicles were allowed into the wetlands. The Helical Pier® foundation anchors (Type SS5 — 1½"-square solid-steel shaft, 10"-dia. helix, hot-dip galvanized) were installed to 7- and 10-ft. depths with average above-grade reveal of 1 to 2 ft. Maximum reveal was 4½ ft.



C3 Environmental	Canadian Construction Controls	EBS Engineering & Construction
C3 Group Industrial Services	Polymeric Engineering	

Services Case Histories Technical Data

Foundation Stabilization

Solves Foundation & Soil Stabilization Problems

Structure Repair/Renovation

Waterproofing

Injection/Grouting/Coatings

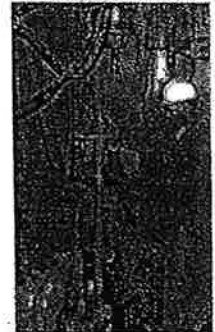
EBS Engineering and Construction Limited is the Foundation Division of the C3 Group. EBS delivers a comprehensive service package to both residential and industrial clients. As specialists in foundation restoration and repair, including foundation stabilization, underpinning, floor lowering and waterproofing, EBS is in the business of problem solving. From new construction to industrial retrofits and environmentally sensitive installations, EBS is your comprehensive foundation resource.

Foundation Stabilization

- CHANCE® HELICAL PIER® Foundation System
- CHANCE® Helical Tieback Anchors
- CHANCE® Soil Screw™ Retention Wall System
- CHANCE® Instant Foundation® System for Walkways
- CHANCE® Helical Pulldown™ Micropiles
- Conventional Foundation Underpinning



Installation of Helical Tieback Anchor in Ground Structures and Retention



Foundation Stabilization of an Eight Story Structure

Structural Repair/Renovation

- Basement Lowering
- Masonry Repair/Stabilization
- Retaining Walls
- Basement Wall Stabilization and Realignment

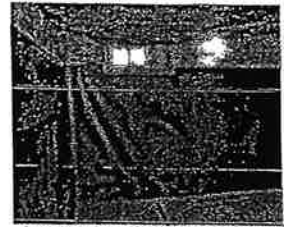


Basement Floor Lowering and Underpinning

Water Proofing

- Installation of Interior/Exterior Membranes

- Epoxy and Urethane Resin Injection for Structural Repair and Water Control
- Underfloor Drainage Systems
- Installation and Replacement of Exterior Drainage Systems



Interior Waterproofing Using an A Membrane

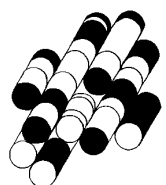
Injection/Grouting/Coatings

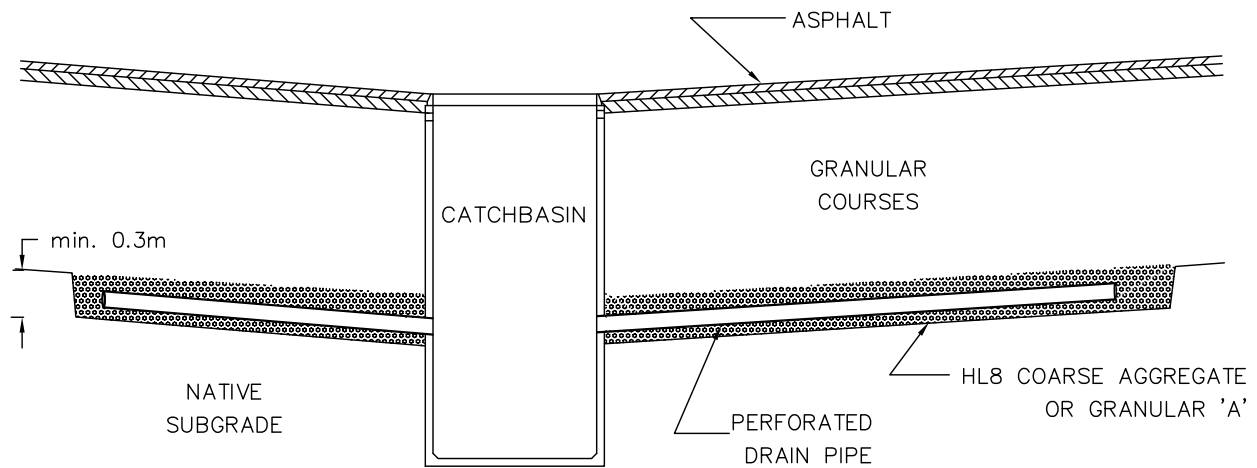
- Cementitious and Polymer Grouting for Structural Repair, Water Control and Stabilization
- Slab Jacking
- Concrete Sealers/Epoxy Coatings

[Return to the top](#)

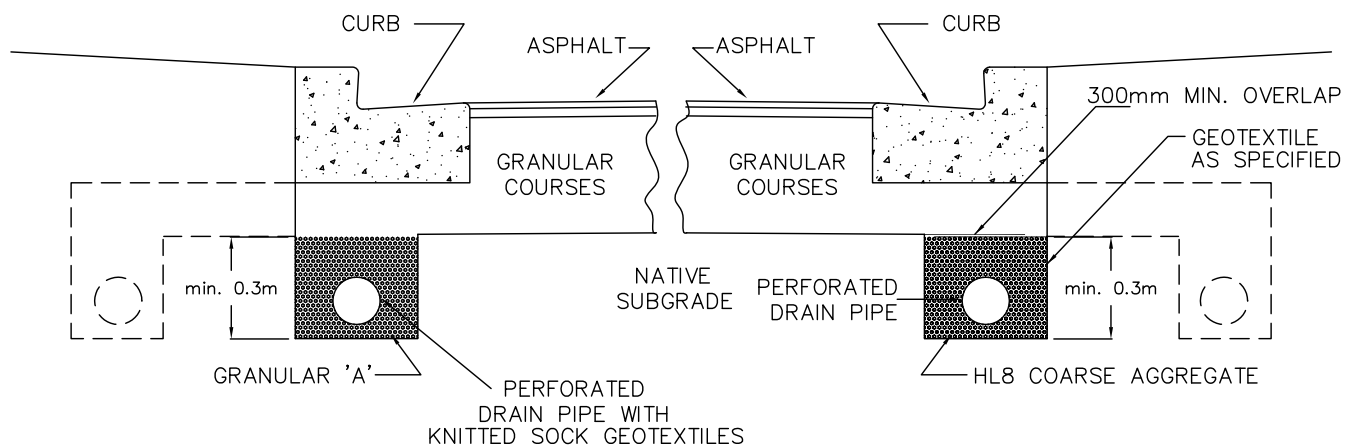
APPENDIX D

TERRAPROBE INC.





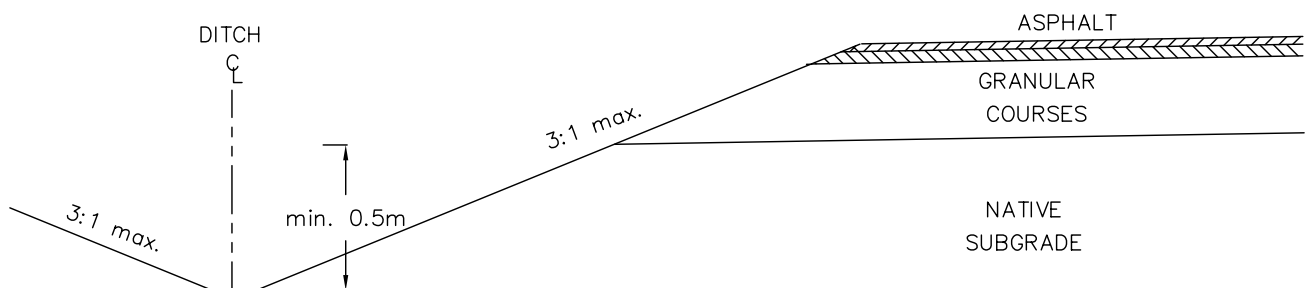
Longitudinal Subdrain Connection to Catchbasin



Unwrapped Trench

Wrapped Trench

Urban Cross Sections



Rural Cross Section

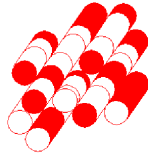


Terraprobe

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Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

PAVEMENT DRAINAGE ALTERNATIVES



Terraprobe

*Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing*

**HYDROGEOLOGICAL ASSESSMENT
PROPOSED MIXED-USE DEVELOPMENT
642 DUNLOP STREET WEST,
BARRIE, ONTARIO**

Prepared For: 25707517 ONTARIO INC.
c/o Innovative Planning Solutions
647 Welham Road Unit 9A
Barrie, ON
L4N 0V7

Attention: Ms. Karla Tomayo

File No. 3-21-0182-46

Issued: April 7, 2022

Revised: March 16, 2023

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Terraprobe Inc.

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Appendix H Groundwater Measurement Results
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1.0 INTRODUCTION

Terraprobe Inc. was retained by 25707517 Ontario Inc c/o Innovative Planning Solutions to conduct a hydrogeological assessment for the property with municipal address 642 Dunlop Street West, Barrie, Ontario, hereinafter referred to as the Property or Site. The Property is located northeast quadrant of the intersection of Dunlop Street West and Miller Road in the City of Barrie. The general location of the Site is shown on **Figure 1**.

The Site is currently developed with two low-rise commercial structures. It is proposed to demolish the existing structures and construct a new 5-storey mixed-use building with slab-on-grade construction (no basement), commercial use and covered parking/driveway at ground level and residential use above. There will also be at-grade exterior parking and a driveway. The proposed development will be municipally serviced for water and sewer.

The purpose of this report is to assess local and regional hydrogeological conditions and the potential impacts of the proposed development on the groundwater system.

2.0 SCOPE OF WORK

The scope of work for the study consisted of the following:

- **Review of available background information:** Available background information for the Site and the project was reviewed. This included the results of geotechnical and environmental investigations of the property and available information regarding the proposed design and construction concepts for the development. In addition, information from public sources, including geologic mapping and MECP well record.
- **Detailed Site Inspection:** An inspection of the property was conducted to review existing Site conditions, including identification of any hydrogeological features such as significant areas of potential groundwater recharge or areas of groundwater discharge. The topographic survey of the Site provided to Terraprobe was reviewed in order to provide a discussion regarding drainage conditions.
- **Private Well Survey:** Due to the current COVID-19 work restrictions that have been imposed by the provincial government and to ensure health and safety, the private well survey has been put on hold.
- **Borehole Drilling:** Prior to the commencement of drilling, the locations of underground utilities, including telephone, natural gas and electrical lines, were marked out by local locating companies, and individual borehole locations were cleared by private utility locating service providers. The field investigation was conducted on January 18 to 20, 2022 and consisted of drilling and sampling a total of nine (9) boreholes ((eight (8) boreholes were advanced with regular sampling and one (1) borehole was augered without sampling for installation of a groundwater monitoring well)). The boreholes extended to depths varying from about 1.8 to 12.6 m below the existing ground surface.
- **Well Installation:** To measure the groundwater level and investigate the quality of groundwater, six (6) boreholes (BH1 to BH5 and BH9) were instrumented with a monitoring well. The

monitoring well consisted of a 50 mm diameter PVC screen with a length of PVC riser pipe and a 10-ft slotted screen. Upon installation, an elevation survey of the monitoring wells, relative to a local datum, was completed so that relative groundwater flow direction can be assessed. The information obtained from the boreholes was used for the hydrogeological assessment.

- **Completion of hydraulic conductivity testing:** Single well response tests (slug tests) were conducted in the selected four (4) monitoring wells (BH2 to BH5) to assess the hydraulic conductivity of the screened strata.
- **Water Balance Analysis:** Pre- and post-development water balance calculations will be conducted to assess infiltration rates and determine the requirements to maintain recharge at the Property.
- **Analysis of potential impacts of groundwater control:** Analysis was conducted to assess the potential impacts of groundwater control on local structures, water wells, and surface water features. This also includes providing recommendations on any requirements for applying for a Permit to Take Water (PTTW) or Environmental Activity and Sector Registry (EASR) posting with the Ministry of the Environment, Conservation and Parks (MECP).
- **Hydrogeology Report:** Following the completion of the above-noted study, a detailed engineering report was prepared regarding the Site hydrogeology. The report provides the following information:
 - Presentation of all the factual information gathered during the study, including the background information and results of site subsurface investigation;
 - Provision of a conceptual site model for local and regional hydrogeologic conditions. The conceptual site model will be used as a basis to assess impacts to local surface and groundwater features;
 - Calculations of water balance for existing (pre-development) and post-development conditions and assessment of infiltration rates to establish the requirements for maintaining groundwater recharge at the property;
 - Develop a Source Water Impact Assessment and Mitigation Plan regarding potential threats to the local water supply aquifer and municipal water supply wells;
 - Impact assessment and mitigation measures to maintain the hydrogeological functions;

3.0 APPLICABLE REGULATIONS AND POLICIES

3.1 Lake Simcoe Region Conservation Authority (LSRCA) Policies and Regulations (O. Reg. 179/06)

Under Section 28 of the Conservation Authorities Act, local conservation authorities are mandated to protect the health and integrity of the regional greenspace system and to maintain or improve the hydrological and ecological functions performed by valley and stream corridors. The LSRCA, through its regulatory mandate, is responsible for issuing permits under Ontario Regulation (**Ont. Reg. 179/06**), *Development, Interference with Wetlands and Alterations to Shorelines and Watercourses* for development proposal or Site alteration work to shorelines and watercourses within the regulated areas.

LSRCA Regulated Area online mapping and pre-consultation comments were reviewed. It is our understanding that the property is partially located within an LSRCA Regulated Area due to the presence of wetland features in the north and the associated minimum vegetation protection zone. As such, it is anticipated that a permit from the LSRCA under Ont. Reg. 179/06 will be required for the proposed development

Refer to **Appendix A** for associated regulatory mapping details.

3.2 Clean Water Act 2006

The MECP mandates the protection of existing and future sources of drinking water under the Clean Water Act, 2006 (CWA). Initiatives under the CWA include the delineation of Wellhead Protection Areas (WHPAs), Significant Groundwater Recharge Areas (SGRAs) and Highly Vulnerable Aquifers (HVAs), as well as the assessment of drinking water quality and quantity threats within Source Protection Regions. Source Protection Plans are developed under the CWA and include the restriction and prohibition of certain types of activities and land uses within WHPAs. This plan dictates that any site within the Lake Simcoe Watershed of South Georgian Bay (SGBLS) region can be rated in terms of score indicating vulnerability to drinking water quality and quantity threats. Based on the review of MECP's Source Protection Information Atlas, County of Simcoe interactive mapping and Lake Simcoe Region Conservation Authority (LSRCA) regulated area mapping, the following information was obtained related to the subject property:

Associated Policy Area	Applicability
Conservation Authority	Lake Simcoe Region and Conservation Authority
Source Protection Area	Lake Simcoe and Couchiching/Black River Source Protection Area
Watershed	Lake Simcoe Watershed
Subwatershed	Barrie Creeks
LSRCA Regulated Area	Yes
Wellhead Protection Area (WHPA)	WHPA-B (2-Year Capture Zone), Intrinsic Vulnerability Score: 6 (Barre Well No: 3A, 4, 5, 7, 17, 18, 19)
Significant Groundwater Recharge Area (SGRA).	No
Highly Vulnerable Aquifer (HVA)	Yes; Intrinsic Vulnerability Score: 6

Wellhead Protection Areas (WHPA - Q) or Recharge Management Area	Wellhead Protection Area Q1: Yes Stress: Low Wellhead Protection Area Q2: Yes Stress: Low
Intake Protection Zone	Intake Protection Zone: 3
Intake Protection Zone Q	No
Issue Contributing Area:	Yes, Contaminant: Chloride
Oak Ridges Moraine (ORM)	The area of proposed development is not located within the Oak Ridges Moraine (ORM)
Niagara Escarpment Plan Area	The area of proposed development is not located within the Niagara Escarpment Plan Area
Greenbelt Protection Act Area	The area of proposed development is not located within the Greenbelt Protection Act Area

Refer to **Appendix A** for associated regulatory mapping details.

3.3 Hydrogeological Assessment Submission Guidelines 2013

The Conservation Authority Guidelines for Development Applications (June 2013) was reviewed as a part of this assessment. This guidance document provides a list of recommended requirements for hydrogeological investigations. Lake Simcoe Region and Conservation Authority (LSRCA) has adopted these guidelines for hydrogeological assessments. The hydrogeology report is completed as per the guidelines provided in the document.

4.0 DESCRIPTION OF SITE CONDITIONS

4.1 Site Location and Description

The Property is located northeast quadrant of the intersection of Dunlop Street West and Miller Road. The general location of the Site is shown on **Figure 1**. The Site is currently developed with two low-rise commercial structures. It is proposed to demolish the existing structures and construct a new 5-storey mixed-use building with slab-on-grade construction (no basement), commercial use and covered parking/driveway at ground level and residential use above. There will also be at-grade exterior parking and a driveway. The proposed development will be municipally serviced for water and sewer.

4.2 Site Topography and Drainage

Based on the boreholes advanced, the Site elevation varies from approximately $234.3 \pm$ masl to $235.7 \pm$ masl (meters above sea level) with an approximate existing grade elevation of $235.0 \pm$ masl. The majority of the Site drains to north/ northwest towards the wetland feature and Miller Drive. **Figure 5** further indicates the topography of the Site.

The nearest surface water feature is the tributary of Dyments Creek, located approximately 50 m south of the proposed development. Dyments Creeks flow northeast, following local topography towards Kempenfelt's Bay/Lake Simcoe. The regional groundwater flow direction is expected to flow east/northeast towards Kempenfelt's Bay/Lake Simcoe, located approximately 3.6 km east.

4.3 Regional Geology and Physiography

The surficial geology of the area is representative of coarse-textured glaciolacustrine deposits in the eastern portion of the Property, consisting of sand, gravel, minor silt and clay of Foreshore and basinal deposits (9c). The northeastern part is recent organic deposits consisting of peat, muck and marl (20). **Figure 3** illustrates the mapped surficial geology for the Site and the surrounding area.

From a regional perspective, the Site is situated within the physiographic feature known as the Sand Plains (11) and within the physiographic landform known as the Simcoe Lowlands (35). The Sand Plains is a narrow stretch of land along the shoreline of Kempenfelt Bay. It also continues from the shoreline through the centre of the Barrie Creeks subwatershed from the lakeshore to the low-lying area to the west of the subwatershed. The physiographic region is typically characterized by having lower elevations, with flat-floored valley features that generally correspond to current river systems (LSRCA, 2012). The lowlands were flooded by glacial Lake Algonquin and, as a result, are floored by sand, silt and clay (Chapman and Putnam, 1984). **Figure 4** shows the location of the Site within the regional physiography map.

4.4 Regional Hydrogeology

The stratigraphy of the surficial deposits within the Barrie Creeks subwatershed is complex as a result of the glacial history. Based on the review of the Barrie Creeks Subwatershed Plan (LSRCA 2012): the subsurface hydrostratigraphy within the Barrie Creeks sub-watershed comprises of the following units.

Geological Unit	Brief Description
Recent Glaciolacustrine Deposits	Recent glaciolacustrine deposits are present at the ground surface, which comprise of sandy silt to silt matrix, moderate to high matrix carbonate content, clast content moderate to high Pleistocene deposits. These surficial deposits generally have low to moderate permeabilities and function as an aquitard.
Halton/Kettleby Till	Underlying the recent glaciolacustrine deposits is the Halton/ Kettleby Till which comprises of sandy silt to clayey silt till. The Halton/ Kettleby Till has low permeabilities and functions as an aquitard.
Oak Ridges Aquifer Complex (ORAC) - Regional Aquifer	Underlying the Halton/Kettleby Till is the ORAC, the shallowest aquifer in the region. It predominantly comprises of granular sediments with interlaid finer materials. The ORAC is generally comprised of soils with medium to high permeabilities.
Upper Newmarket Till	Underlying the ORAC is the Newmarket Till, which acts as an aquitard. The Newmarket till comprises of dense sandy silt to clayey silt and generally has a low permeability.
Sunnybrook Drift	The Sunnybrook Drift underlies the TAC and acts as a regional aquitard in the sub-watershed. The Sunnybrook Drift comprises of silt and clays with low permeabilities and functions as an aquitard.
Scarborough Aquifer Complex (SAC) - Regional Aquifer	The SAC is the third regional aquifer in the area underlying the Sunnybrook Drift. The SAC comprises sands, silt and clays with varying permeabilities.

Bedrock was not contacted over the current subsurface investigation. Bedrock in the area is middle Ordovician Verulam (54a) formation consisting of limestone, dolostone, shale, arkose, sandstone of the Simcoe Group. The Verulam Formation occurs along the shoreline of Kempenfelt Bay and expands west of the City of Barrie. The Verulam Formation ranges from 32 to 65 m thick and is rich in fossiliferous limestone with inter-beds of calcareous shale, which indicates that the depositional environment was a was open marine shelf environment. Based on the review of the Oak Ridges Moraine Groundwater Program (OGRMP) cross-section tool, bedrock lies at an elevation of approximately 120 masl, and depth to bedrock at the Site is approximately 115 m below ground surface.

4.5 Watershed Setting

The Barrie Creeks subwatershed is one of six (6) subwatersheds that drain into Lake Simcoe. The subwatershed's streams empty into Kempenfelt Bay. The Barrie Creeks subwatershed is located almost entirely within the Town of Barrie, with a small portion falling into the Oro-Medonte in the northeast and Springwater to the northwest, accounting for the remaining area at 5% and 2%, respectively. The Barrie Creeks subwatershed is a densely populated subwatersheds and has a total area of approximately 37.5 km² and accounts for 1.4% of Lake Simcoe's total watershed area, with a total watercourse length of 53.8 km (including all branches), or about 1.3% of the total combined watercourse length of the entire Lake Simcoe watershed (LSRCA 2012).

4.6 Local Surface Water and Natural Heritage Features

Mapping from the Ontario Ministry of Natural Resources and Forestry (MNR) was to determine if water bodies, wetland and woodland features were present on the Property and within the Study Area. The Ontario Ministry of Natural Resources National Heritage Information Centre database for listings of Areas of Natural or Scientific Interest (ANSIs) was reviewed. The natural heritage map is presented in **Figure 6**. The information is summarized below.

Water Bodies	Property	No water bodies are present on the Property
	Study Area	The nearest surface water feature is a tributary of Dyments Creek (a tributary of Barrie Creeks), located approximately 50 m south of the proposed development.
Wetlands	Property	<u>Provincially Significant Evaluated Wetland</u> - No Provincially Significant wetlands are present on the Property <u>Non- Provincially Significant Evaluated Wetland</u> - No Non-Provincially Significant wetlands are present on the Property. <u>Unevaluated Wetland</u> - Unevaluated wetland feature is present in the north portion of the Property
	Study Area	<u>Provincially Significant Evaluated Wetland</u> - The closest Provincially Significant Evaluated Wetland is present approximately 180 m southwest from the Property. <u>Non- Provincially Significant Evaluated Wetland</u> - No Non- Provincially Significant wetlands are present in the Study area. <u>Unevaluated Wetland</u> - Unevaluated wetland feature is present approximately 60 m south of the Property.
Woodlands	Property	Woodland areas are identified on the Property in the northern portion of the Property
	Study Area	The closest woodland area was identified in close proximity of the Property along the north and south perimeter.
ANSIs	Property	<u>Provincially Significant Life Science ANSI</u> - No Life Science ANSIs were identified on the Property. <u>Provincially Significant Earth Science ANSI</u> - No Earth Science ANSIs were identified on the Property.
	Study Area	<u>Provincially Significant Life Science ANSI</u> - The closest Life Science ANSIs is located approximately 1.8 km south of the Property <u>Provincially Significant Earth Science ANSI</u> - No Provincially Significant Earth Science ANSIs were identified in the Study Area



4.7 Local Groundwater Resources

MECP Water Well Records (WWRs) were reviewed for the registered wells located at the Site and within a 500 m radius of the Site boundaries (study area). Information contained in these records provides data for determining the nature and use of local groundwater resources. A total of 28 well records were found. The locations of the well records in the Study Area are presented on **Figure 7**, with the details for each well summarized in **Appendix B**. A summary of data obtained from these MECP records is presented in the Table below:

Total Number of Wells	28
Depth Ranges	
50 ft. or less	7 (25%)
51 ft. to 100 ft.	6 (21%)
Greater than 100 ft.	7 (25%)
Unknown	8 (29%)
Water Use	
Domestic Water Supply	11 (39%)
Public/ Municipal Water Supply	1 (4%)
Commercial/ Industrial Water Supply	3 (11%)
Monitoring/Test Holes	9 (32%)
Not Used/ Abandoned	3 (11%)
Unknown	1 (4%)

Based on the review of the well records, it is indicated that fifteen (15) active water supply wells were found within a 500 m radius of the area of the Site constructed in the year 1959 - 2007, with depths ranging from 4.3 mbgs to 96.3 mbgs. The locations of the well records in the Study Area are presented on **Figure 7**. The water supply well record is summarized in the table below:

Map ID	MECP Well ID	Well Depth (m)	Well Use	Construction Date	Approx. Distance from the Site (m)
1	5704843	20.4	Domestic	May 6, 1959	285
2	5704844	75.3	Domestic	October 21, 1964	410
3	5704866	29.0	Domestic	September 18, 1958	220
4	5704867	4.3	Domestic	June 2, 1967	410
5	5709133	34.1	Commercial	April 21, 1972	365
6	5711998	7.0	Domestic	July 5, 1974	260
9	5716164	22.6	Commercial	July 12, 1979	200
10	5716689	79.9	Industrial	June 8, 1979	415
11	5718060	25.3	Domestic	April 23, 1982	390
12	5718624	15.2	Commercial	August 3, 1983	15
13	5718988	19.8	Domestic	January 26, 1984	285
14	5719487	12.2	Domestic	November 7, 1984	205
15	5728600	73.2	Domestic	October 18, 1991	195
19	5739405	27.7	Domestic	November 12, 2004	170
22	7114203	95.7	Municipal	December 7, 2007	440

Based on the review of Risk Management comments dated: January 13, 2021, it is indicated that two (2) potential wells are located on the site (Map ID: 12 and 27). The Well ID 5718624 (Map ID: 12) is registered for commercial use and constructed in 1983. However, the Well ID 7260721 (Map ID: 27) is registered as a test hole for 636 Dunlop St West, Barrie.

Moreover, one (1) active Barrie municipal production well PW-19 (Well ID: 7114203, Map ID: 22) is located within 500 m of the Site under permit number P-300-6139583233 (Issue date: 2022-01-20) with a maximum allowed groundwater taking of 7,862,400 L/day screened in lower A3/A4 aquifer. The details of the Barrie well supply system is provided below.

4.8 Barrie Well Supply

Based on the review of *Lakes Simcoe and Couchiching-Black River SPA Part 1 2014 Updated Assessment Report, Chapter 8, City of Barrie*, it is indicated that the aquifers underlying the City of Barrie are part of a regionally extensive and complex aquifer system, within which four major sand and gravel aquifer units have been identified. Locally, these are referred to as the upper (Aquifer A1), intermediate (Aquifer A2) and lower (Aquifer A3 and A4) aquifers.

Aquifer A1 is largely unconfined under all of the central and most other parts of the City and is, therefore, susceptible to contamination from surface activities. The portions of the upper aquifer identified in the Oro-Medonte and upland areas immediately west of the City correspond to the regional Aquifer A1, which is mapped as ice contact stratified drift. Aquifer A2 is regionally extensive. The aquifer is complex within the central core area of Barrie, where it consists of inter-layered sand and silt/clay materials. The aquifer is overlain by 5 to 20 or more metres of confining materials in most areas, reported to consist of clayey and silty sediments. The lower aquifers, A3 and A4, are extensive sand and gravel aquifers, which form the source of the majority of Barrie's groundwater supply. These aquifers are in direct contact with one another under the central Barrie area, as well as further to the west. Based on the base elevation of the Kempenfelt Bay and the known elevation of the upper surface of Aquifer A3, it is interpreted that the aquifer is in contact with Kempenfelt Bay in areas to the east. Wells 3A, 5, and 12 are all constructed in the deeper Aquifer A4. Wells 7, 9, 11, 13, 14, 15, 17, and 18 are located approximately in the centre of the combined A3/A4 aquifer, whereas the remaining wells (Wells 4 and 16) are located in the upper part of the Aquifer 3.

4.9 Results of Door-to-Door Survey

The private well survey requires to contact residents within the study area to collect information and water samples from private wells to assess groundwater uses in the area. Due to the current COVID-19 restrictions that have been imposed by the provincial government and to ensure health and safety, the private well survey has been placed on hold.

5.0 RESULTS OF SUBSURFACE INVESTIGATION

The field investigation was conducted between January 18 and 20, 2022 and consisted of drilling and sampling a total of nine (9) boreholes ((eight (8) boreholes were advanced with regular sampling and one (1) borehole was augered without sampling for installation of a groundwater monitoring well)) extending to depths varying from about 1.8 to 12.6 m below existing ground surface.

To measure the groundwater level and investigate the quality of groundwater, six (6) boreholes (BH1 to BH5 and BH9) were instrumented with a monitoring well. The locations of boreholes and monitoring wells are shown on the attached **Figure 2**.

The boreholes were drilled by a specialist drilling contractor using truck/track-mounted drilling rigs equipped with power augers. The borings were advanced using continuous flights of solid stem augers and were sampled at 0.75 m intervals with conventional 50 mm diameter split barrel samplers. The drilling was conducted under the full-time supervision of a member of our field staff, who logged the borings and examined the samples as they were obtained. All samples obtained during the investigation were sealed into plastic jars and transported to our geotechnical testing laboratory for detailed inspection and testing.

5.1 Local Site Setting

Based on the review of the geotechnical report **File No. 3-21-0182-01**; the subsurface soil stratigraphy is indicated below. The following stratigraphy is based on the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples. The stratigraphic boundaries indicated on the Borehole Logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes and will vary between and beyond the borehole locations.

5.1.1 Surficial Topsoil/ Earth Fill Material/Organic Deposits

A surficial pavement structure consisting of 50 mm to 230 mm of asphaltic concrete was encountered at the surface of Boreholes 1, 3, 5, 6 and 8. The asphaltic concrete was found underlain by 130 mm to 370 mm of granular fill at Boreholes 1, 3, 5 and 6. **Topsoil** was encountered at Boreholes 2, 4 and 7, and its thickness ranged from 140 to 150 mm. A zone of earth fill materials was encountered in boreholes 2, 4, 7 and 8 and extended to the depths of 0.6 m below grade. The earth fill materials consisted of sand to sandy silt to with organics, peat and some gravel. The fill material indicated a loose to compact relative density with the moist condition. A layer of **Peat**, with some sand seams, was encountered in Boreholes 1, 3, 5 and 6 beneath the pavement structure/ earth fill zone and extended to a depth ranging from 0.8 to 1.5 m (Elev. 232.9 m to 234.5 m) m below grade indicating a moist condition.

5.1.2 Native Soils

Undisturbed native soil deposits underlie the topsoil/earth fill/organic deposits, which extended to the full depth of investigation are as follows:

- **Sand**, with trace amounts of silt, clay and gravel, was encountered beneath the surficial pavement/topsoil layer in all boreholes and extended to the full depth of the investigation (about 12.6 m depth below grade), indicating a wet condition. The sand generally became water-bearing below about 1.5 ±m depth below grade.

The detailed stratigraphic conditions are presented on the accompanying borehole logs provided in **Appendix C**. A subsurface profile of Site is provided in **Figure 8**. Geotechnical characterization of the various soil types, including grain size analysis, was conducted and is presented in **Appendix D**. Additional information pertaining to soil stratigraphy is discussed in the geotechnical report by Terraprobe under a separate cover (**File No. 3-21-0182-01**).

5.2 Monitoring Well Installation

Monitoring wells were installed in six (6) boreholes (BH1 to BH5 and BH9) for groundwater monitoring and to investigate groundwater quality. The monitoring wells were constructed using 50-mm diameter PVC riser pipes and screens, which were installed in each of the selected geotechnical boreholes in accordance with Ontario Regulation (O. Reg.) 903. Filter sand was placed around the well screen to approximately 0.6 m above the top of the screen. The wells were then backfilled with bentonite to approximately 0.3 m below the ground surface. All monitoring wells were surveyed using an R10 Trimble GPS relative to a geodetic datum. The details are provided below:

Well ID	Well Diameter (mm)	Ground Surface Elevation (masl)	Top of Screen		Bottom of Screen		Screened Geological Units
			Depth (mbgs)	Elev. (masl)	Depth (mbgs)	Elev. (masl)	
BH 1	50	235.5	4.6	230.9	7.6	227.9	Sand
BH 2	50	234.5	8.5	226.0	11.6	222.9	Sand
BH 3	50	235.3	4.6	230.7	7.6	227.7	Sand
BH 4	50	235.4	9.1	226.3	12.2	223.2	Sand
BH 5	50	235.2	1.5	233.7	4.6	230.6	Sand
BH 9	50	234.4	1.5	232.9	4.6	229.8	Sand

Note: masl: meters above sea level, mbgs: meters below ground surface

Additional details of the monitoring well installation is presented on the enclosed borehole logs provided in **Appendix C**.

5.3 Groundwater Monitoring Program

It is noted that there is a requirement for long-term monitoring of groundwater levels to assess seasonal variations for the period of one year (12 months). The groundwater monitoring program is ongoing. Monthly manual groundwater level observations will be noted in the installed monitoring wells over the period of one year. The first reading was measured at the event of hydrogeological testing on February 01, 2022 as indicated below:

Monitoring Well	Ground Surface Elevation (masl)	February 01, 2022	
		Groundwater Depth (mbgs)	Groundwater Elevation (masl)
BH 1	235.5	*	*
BH 2	234.5	1.7	232.8
BH 3	235.3	1.8	233.5
BH 4	235.4	1.9	233.5
BH 5	235.2	1.5	233.7
BH 9	234.4	1.7	232.7

Note: masl: meters above sea level, mbgs: meters below ground surface

*non-accessible due to snow cover

Based on the available water level recordings till date from February 2022 to December 2022, it is noted that the groundwater level in the overburden varies from $232.2 \pm \text{masl}$ to $234.4 \pm \text{masl}$. Local groundwater flow direction in the area expected to mimic the surface topography appears to be in the north towards the adjacent wetland area. However, the regional groundwater flow direction is expected to flow east/northeast towards Kempenfelt's Bay/Lake Simcoe, located approximately 3.6 km east. The groundwater flow direction is shown in **Figure 9**. However, this pattern does not take into account the water level observed at monitoring well BH1 and should be revised later based on the groundwater monitoring data.

It should be noted that the groundwater levels noted above may fluctuate seasonally depending on the amount of precipitation and surface runoff. Further, long-term groundwater monitoring will be required to capture the seasonal groundwater flow fluctuations. Upon completion of the ongoing long-term monitoring, an addendum report will be issued incorporating the hydrographs representing the seasonal groundwater fluctuations at the Site.

The monitoring wells installed at the Site need to be maintained in accordance with Ontario Water Resources Act, O. Reg. 903/90. When the wells are no longer required for monitoring or sampling purposes, these wells will need to be appropriately decommissioned by a licensed well contractor as outlined in the Regulation.

Please note: the results of the groundwater measurement at the property between February 2022 to December 2022 is attached.

5.4 Estimation of Hydraulic Conductivity

5.4.1 Estimation from Grain Size

The Allen Hazen equation estimation method was also used to estimate the hydraulic conductivity (K) for saturated subsoils at selected depths beneath the water table below the subject site. The Hazen Equation method was adopted to estimate the hydraulic conductivity (K) for different soil layers which may contain groundwater during the seasonal high-water table (spring) period or if they are not encountered within the screen intervals. The Allen Hazen Equation method relies on the interrelationship between hydraulic conductivity and effective grain size, d_{10} , in the soil media. This empirical relation predicts a power-law relation with K, as follow:

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

where;

- d_{10} : Value of the soil grain size gradation curve as determined by sieve analysis, whereby 10% by weight of the soil particles are finer, and 90% by weight of the soil particles are coarser.
- C: Coefficient; it is equal to 1 when K in cm/sec and d_{10} is in mm
- K: Estimated hydraulic conductivity in cm/s

The Allen Hazen Equation estimation provides an indication of the groundwater yield capacity for saturated soil strata at the depths where soils samples were selected for grain size analysis. The grain size distribution graphs prepared for the geotechnical investigation were used to estimate the hydraulic conductivity. The grain size analysis results are provided in **Appendix D**. The results of the Hazen equation are summarized below:

Borehole No./Sample ID	Sampling Depth (mbgs)	Sampling Elevation (masl)	D ₁₀ Grain Size (mm)	Soil Description (Native)	Estimated Hydraulic Conductivity (m/s)
Borehole 1 Sample 5	3.3	232.2	0.122	SAND, trace gravel, trace silt, trace clay	1.5×10^{-4}
Borehole 2 Sample 4	2.5	232	0.119	SAND Trace silt, trace clay	1.4×10^{-4}
Borehole 3 Sample 7	6.3	229	0.120	SAND Trace silt, trace clay	1.5×10^{-4}
Borehole 4 Sample 10	10.9	224.5	0.129	SAND, trace gravel, trace silt, trace clay	1.7×10^{-4}

Note: masl: meters above sea level, mbgs: meters below ground surface

Based on the Allen Hazen method, the hydraulic conductivity of the native sand deposit is estimated in the order of 10^{-4} m/s, indicating a higher permeability of the native soils.

5.4.2 Estimation from In-situ Hydraulic Testing

The hydraulic conductivity was also determined based on single well response tests (Slug Tests)

performed on the selected four (4) monitoring wells (BH2 to BH5). The monitoring wells were developed in advance. Well development involves the purging and removal of groundwater from the monitoring wells to remove remnants of clay, silt and other debris introduced into the monitoring well during construction and to induce the flow of formation groundwater through the well screens, thereby improving the transmissivity of the subsoil strata formation at the well screen depths.

The static water level was measured prior to the test. The Solinst Datalogger was programmed to record the water levels at one (1) second of the interval throughout each test. The data from the tests were analysed using Bouwer and Rice method (1967) included in the Aquifer Test V.7 software Package. The results of the analysis are presented in **Appendix E**. The hydraulic properties of the strata applicable to the Site are as follows:

Monitoring Well ID	Top of Well Screen Elevation (masl)	Bottom of Well Screen Elevation (masl)	Screened Geological Units	Hydraulic Conductivity (m/s)
BH 2	226.0	222.9	Sand	4.1×10^{-6}
BH 3	230.7	227.7	Sand	5.8×10^{-6}
BH 4	226.3	223.2	Sand	2.2×10^{-6}
BH 5	233.7	230.6	Sand	6.1×10^{-6}

Note: masl: meters above sea level, mbgs: meters below ground surface

Based on the in-situ hydraulic testing, the hydraulic conductivity of the native sand deposit is estimated in the order of 10^{-6} m/s, indicating a higher permeability of the native soils.

5.4.3 Estimation from Literature

According to Freeze and Cherry (1979), the typical hydraulic conductivity of the strata investigated at the site are:

Soil Unit	Estimated Hydraulic Conductivity Range (m/s)
Earth Fill	10^{-6}
Sand (Native)	$10^{-2} - 10^{-6}$

Based on the analyses, the hydraulic conductivity calculated from the single well response testing and grain size analyses are consistent with the published values associated with the geological material tested.

5.5 Assessment of Groundwater Quality

One (1) unfiltered groundwater sample was collected by Terraprobe and analyzed by a Canadian laboratory accredited and licensed by the Standards Council of Canada and or Canadian Association for Laboratory Accreditation. The sample was collected directly from monitoring well BH2 on February 01, 2022. The monitoring well BH2 was developed and purged prior to sample collection.

Upon sampling, all of the bottles were placed in ice and packed in a cooler for shipment to the analytical laboratory. Sample analysis was performed by AGAT Laboratories, a laboratory accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA). The filtration for aluminum dissolved and mercury was performed in the lab. The sample was compared with Water Quality Assessment - Provincial Water Quality Objectives (PWQO) Standards.

In summary, the results indicate the following:

- The groundwater sample meets the permissible limits for Provincial Water Quality Objectives (PWQO) for all the parameters analyzed except the following parameter.

Parameter	BH 2 Groundwater Quality Results (mg/L)	PWQO Guideline Limits (mg/L)	Reported Detection Limit (RDL)
Total Iron	1.48	0.3	0.010

The groundwater quality test results and the certificate of analysis are presented in **Appendix F**.

6.0 IMPACT ASSESSMENT

The impact assessment details that are applicable to the Property are discussed below:

6.1 Impact Assessment of Water Quantity Threats

The Site is located in the Recharge Management Area - Well Head Protection Area WHPA-Q1 and Q2. WHPA Q1- Refers to the area where activities that take water without returning it to the same source may be a threat, and WHPA Q2- refers to the area where activities that reduce recharge may be a threat. The Clean Water Act, 2006 identifies two activities that are prescribed to be drinking water quantity threats. The activities prescribed to be drinking water threats under the Clean Water Act, 2006 are those considered to be manmade. These activities, as listed in the Act, are provided below. Activities 19 and 20 are potential threats to water quantity as indicated below:

Threat #	Drinking Water Threat (Description)	Vulnerable Area	Applicable Circumstance	Risk
19	An activity that takes water from an aquifer or a surface water body without returning the water taken to the same aquifer or surface water body (dewatering requirements)	WHPA-Q1	An existing taking, an increase to an existing taking or a new taking. The water is or would be taken from within a WHPA-Q1.	Low
20	An activity that reduces recharge to an aquifer (Paving of driveways and increased roof area in post-development)	WHPA-Q2	An existing activity, a modified activity or a new activity. The activity is or would be wholly or partly located within a WHPA-Q2	Low

6.1.1 Water Balance Methodology and Assessment

It is common practice and an accepted method to provide estimates of surplus using a Thornthwaite and Mather approach, where surplus is estimated based on precipitation minus evapotranspiration. With the recent completion of technical studies required under The Clean Water Act 2006, numerical models were utilized to estimate interception, evaporation, potential and actual evapotranspiration, snowmelt, runoff, infiltration, interflow, and groundwater recharge. Many of these model estimates are based on soils, surficial geology and land use mapping products but may also consider detailed vegetation attributes as well as hydrological cycle functions. These modelling output data are available, and consultants are encouraged to use them to complete site-specific water balance assessments.

A water balance for the Site was prepared to assess the distribution of rainfall, runoff, and infiltration for pre-development and post-development conditions based on the "Lake Simcoe Climate Data Reference Document to Support the Completion of Water Balance Assessment Version 1.0 (April 2017)". The water balance tables provided in this document are average values obtained from the numerical modelling undertaken by Earthfx (2010) required under the Clean Water Act, 2006. The resulting water balance parameters are categorized by various vegetation covers in different soil types for each

subwatershed within the Lake Simcoe Basin. Infiltration factors can then be applied based on specific site conditions – vegetation, soil and topography, per the above-mentioned MOE methodology. When applied to an appropriate catchment area, they can provide reasonable estimates of infiltration for comparison purposes

Based on the Barrie Creeks Subwatershed climate data, the mean annual **precipitation** is considered as 952 mm/yr., and the **mean annual actual evapotranspiration** is considered as 446 mm/yr. The volume of surplus water, i.e. $952 \text{ mm/yr.} - 446 \text{ mm/yr.} = 506 \text{ mm/yr.}$ that infiltrates into the soil was determined by applying an infiltration factor to the surplus volume based on topography, soil type and land cover as per Table 2 approach in the *Technical Information Requirements for Land Development Applications* (1995). Based on the Table 2 approach, the **infiltration factor** is estimated as 0.5, considering the topography as flat topography (0.2), soil type as clay and loam (0.2) and cover as cultivated (0.1). The infiltration factors for the post-development conditions were considered the same as the pre-development conditions. The rate of infiltration was based on the Table 2 approach in the *Technical Information Requirements for Land Development Applications* (1995) and is considered as 278 mm/yr.

Based on the above information, a conceptual model of groundwater flow and water balance was developed. A water balance was conducted for the post-development conditions for the entire Site and the feature-based water balance for the wetland north of the site, using the proposed land use statistics information and property development plan. The post-development water balance accounts for hard-surfaced areas created by buildings and pavements. The post-development conditions will result in a surplus of water available from run-off. The surplus of water available from roof runoff can be used for infiltration into the shallow groundwater system.

The following assumptions were applied for the pre-and post-development water balance:

- No infiltration will occur beneath the hard surface areas, including asphalt/concrete surfaced parking areas and walkways or driveways.
- It is assumed that there will be 10% of evaporation in hard-surface areas/impervious surfaces, and the remaining will contribute to run-off.
- Run-off from parking areas will be directed towards storm sewers and is not included in the infiltration calculations.
- Infiltration rates in open areas of the property (landscaped areas) will occur at rates similar to those for pre-development conditions.
- There will be no infiltration beneath hard-surface areas, including buildings, pavements, and walkways.

Based on the land use statistic information provided in Stormwater Management Report prepared by WMI & Associate Limited for the site File # 21-710 dated March 2023. To calculate the water balance for the property and the feature-based water balance of the wetland north of the property. The site is divided into two catchment/ drainage areas (**Drainage area A and Drainage Area B**). The proposed development statistics of each of the wetland is summarized below:

6.1.1.1 Drainage A (Catchment Area 0.6 ha):

The drainage area of approximately 0.6 ha, and proposed development of approximately 0.14 ha (building and hard surface area).

Drainage A (Catchment Area 0.6 ha)-Summary of Site Statistics (Pre-development)

Land Use	Area (ha)	Area (m ²)
Building Footprint/Envelope	0.02	200
Hardscape/Impervious	0.12	1,200
Softscape/Pervious (Undeveloped Area)	0.46	4,600
Total	0.60	6,000

Pre- Development Water Balance (Entire Site)

	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Existing	6,000	5,712	1,932	133	1,835	1,811

Please note: the pre-development run-off from drainage A- catchment area support the adjacent north wetland, it is proposed to be marshed in the post-development condition by conveying the captured building roof run-off to mitigate the post-development condition.

6.1.1.2 Drainage B (Catchment Area 0.21ha):

The drainage area of approximately 0.21 ha, and proposed development of approximately 0.16 ha (hard surface area).

Drainage B (Catchment Area 0.21 ha)-Summary of Site Statistics (Pre-development)

Land Use	Area (ha)	Area (m ²)
Building Footprint/Envelope	0.16	1,600
Hardscape/Impervious	0.015	150
Softscape/Pervious (Undeveloped Area)	0.035	350
Total	0.21	2,100

Pre- Development Water Balance (Entire Site)

	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Existing	2,100	1,999	210	152	200	1,437

Please note: the pre-development run-off from drainage B- catchment area drain to sewer systems located to the south of the site and this is proposed to be maintained in the post-development condition.

6.1.1.3 Entire Site Water Balance

The water balance for pre-development conditions for the entire Site is summarized in the Table below:

Entire Site- Summary of Site Statistics (Pre-development)

Land Use	Area (ha)	Area (m ²)
Building Footprint/Envelope	0.02	200
Hardscape/Impervious	0.28	2,800
Softscape/Pervious (Undeveloped Area)	0.51	5,100
Total	0.81	8,100

Entire Site- Pre- Development Water Balance (Entire Site)

	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Existing	8,100	7,711	2,142	286	2,035	3,249

Development of an area affects the natural water balance of the Site. The most significant difference is the addition of impervious surfaces as a type of surface cover. Impervious surfaces prevent the infiltration of water into the soils. Net effect of the construction of impervious surfaces is that most of the precipitation that falls onto the impervious surfaces becomes surplus water and direct runoff. The natural permeability of the ground surface changes by reducing the current undeveloped land/ open space and vegetation at the Site and replacing part of these areas with less permeable/ impervious surfaces such as building roofs, roads, and driveways. The development will result in an increased volume of runoff and a reduction in infiltration. Pre-development conditions result in approximately 2,035 m³ of water available for infiltration to the groundwater system, as mentioned in the above table.

Based on the post-development plan, a summary of proposed land use and water balance calculations for the post-development are provided below:

Entire Site- Summary of Site Statistics (Post-development)

Land Use	Area (ha)	Area (m ²)
Building Footprint/Envelope	0.24	2,400
Hardscape/Impervious (Roads)	0.30	3,000
Softscape/Pervious (Undeveloped Area)	0.27	2,700
Total	0.81	8,100

Post- Development Water Balance without Mitigation (Entire Site)

	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Proposed Development	8,100	7,711	1,134	514	1077	4,986

Comparison of Pre-Development & Post-Development Water Balance Over (Entire Site)

	Precipitation(m ³)	Evapotranspiration Losses (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-off to Wetland (m ³)
Pre-Development	7,711	2,142	286	2,035	3,249
Post-Development	7,711	1,134	514	1,077	4,986
Difference	0	-1,008	228	-958	1,737

In the post-development, the water balance calculations show that development has the potential to reduce the natural infiltration by **958 m³/yr.** and to increase the runoff by 1,737 m³/yr. Conservation Ontario Guidelines (Conservation Ontario, 2013) suggest a target of 80 % of the predevelopment infiltration being maintained in the post-development conditions. Calculations for the Site are indicative of the post-development infiltration being at a level of about 53 % of the pre-development infiltration. As such, opportunities to capture run-off and provide secondary infiltration in greenspace areas will be required to increase post-development infiltration. This indicates that with proper stormwater management and mitigation measures are required to maintain the overall infiltration rates at the property. The low-impact development (LID) techniques that are being considered as part of the post-development design proposed in the stormwater management report will mitigate the infiltration deficit.

Roof runoff Infiltration Deficit (Entire Site)

	Potential Post-Development Infiltration Deficit (m ³) with mitigation
Proposed Development	958

Post-Development- Available Roof Run-off

The rooftop runoff water will be re-infiltrated through stormwater management features, a total of 2,056 m³/yr. of water could be available for infiltration/compensation of the adjacent wetland north of the site under post-development conditions (assuming 90% available of the roof run-off captured).

The volume of roof run-off available post-development, as noted in the Table below:

Roof Run-off Volume of Available (Entire Site)

	Volume of Roof Run-off Available (m ³)
Proposed Development	1,851

The water balance calculations are provided in **Appendix G.**

Post-development Condition-Feature-Based Water Balance (Wetland North of the Site) :

In the post-development, it is expected that available run-off to the wetland should match the pre-development run-off volume of **1,811 m³/yr.**

As noted above, the total volume of rooftop runoff water available to compensate/mitigate the post-development wetland runoff and the site infiltration is **1,851 m³/yr.**

It is proposed in the Storm Water management LID report prepared by WMI & Associates Limited to channel the run-off to the wetland adjacent north of the site.

Post-development Roof runoff Compensation (Wetland North of the Site & Site Infiltration Deficit)

Available Roof Run-Off (m ³)	Runoff Available to Compensate Adjacent Wetland to the North (m ³)	Volume of Roof Run-off for Site Infiltration deficit(m ³)
1,851	1,811	40

In summary, the pre-development run-off (**1,811 m³/yr.**), can be fully mitigated in the post-development condition, while the post-development infiltration deficit for the site can be reduced from (**958 m³/yr.** to (**918 m³/yr.**) based on the available roof runoff. The infiltration deficit (**918 m³/yr.**) for the site will remain unmitigated.

In order to maintain the post-development water balance of the adjacent wetland north of the site, WMI & Associates Limited has proposed LID techniques to be implemented. The stormwater management measures specifically address the maintenance of surface drainage, ground water recharge and the hydrogeological function of the property and the north wetland and site infiltration to reduce the deficit. These LID measures can be used to increase and mitigate post-development groundwater conditions. The proposed LID measures will include the following:

- Drainage from the building rooftop will drain northward via a separate storm sewer towards the adjacent north wetland. The drainage will be through a single storm sewer outlet swale and overland flow path.
- Water balance will be provided to the greatest extent possible via the rooftop outlet to the adjacent north wetland. In addition, runoff volume controls are provided via retention in the adjacent north wetland.

Further details on the design and implementation of the LID techniques are described in the “ Stormwater Management Report, 642 Dunlop Street West, City of Barrie, dated March 2023, File # 21-710, prepared by WMI & Associates Limited. The information in this report describes the best management practice to mitigate and compensate for the post-development hydrogeological condition.

7.0 Groundwater Control and Dewatering Requirements

7.1 Construction Requirements and Methodology

Current Development:

The Site is currently developed with two low-rise commercial structures.

Proposed Development:

It is proposed to demolish the existing structures and construct a new 5-storey mixed-use building with slab-on-grade construction (no basement), commercial use and covered parking/driveway at ground level and residential use above. There will also be at-grade exterior parking and driveway. The proposed development will be municipally serviced for water and sewer.

Based on the design information provided by the client, Terraprobe identified the following details related to the proposed development:

Proposed Development					
Development Phase	Above Grade Levels	Below Grade Levels			
		Level #	Lowest Finished Floor (FFE)		Approximate Foundation Elevation (masl)
			Depth (m)	Elevation (masl)	
- Five-story slab-on-grade building	5	0	Slab-on-Grade Structure	±236.0	± 234.5* (Spread footing resting on Engineered Fill)

* 1.5 m of earth cover or equivalent insulation for frost protection

In general, the stratigraphic units that were encountered at the Property are summarized below:

A surficial pavement structure consisting of 50 mm to 230 mm of asphaltic concrete was encountered at the surface of Boreholes 1, 3, 5, 6 and 8. The asphaltic concrete was found underlain by 130 mm to 370 mm of granular fill at Boreholes 1, 3, 5 and 6. **Topsoil** was encountered at Borehole 2, 4 and 7, and its thickness ranged from 140 to 150 mm. A zone of earth fill materials was encountered in boreholes 2, 4, 7 and 8 and extended to the depths of 0.6 m below grade. The earth fill materials consisted of sand to sandy silt to with organics, peat and some gravel. The fill material indicated a loose to compact relative density with the moist condition. A layer of **Peat**, with some sand seams, was encountered in Borehole 1, 3, 5 and 6 beneath the pavement structure/ earth fill zone and extended to a depth ranging from 0.8 to 1.5 m (Elev. 232.9 m to 234.5 m) m below grade indicating a moist condition.

Undisturbed native soil deposits underlie the topsoil/earth fill/organic deposits, which extended to the full depth of investigation are as follows:

Sand, with trace amounts of silt, clay and gravel, was encountered beneath the surficial pavement/topsoil layer in all boreholes and extended to the full depth of the investigation (about 12.6 m depth below grade), indicating a wet condition. The groundwater conditions at the Property are summarized below:

Monitoring Well	Ground Surface Elevation (masl)	February 01, 2022	
		Groundwater Depth (mbgs)	Groundwater Elevation (masl)
BH 1	235.5	*	*
BH 2	234.5	1.7	232.8
BH 3	235.3	1.8	233.5
BH 4	235.4	1.9	233.5
BH 5	235.2	1.5	233.7
BH 9	234.4	1.7	232.7

Note: masl: meters above sea level, mbgs: meters below ground surface
*non-accessible due to snow cover

Based on the in-situ hydraulic testing, the hydraulic conductivity of the native sand deposit is estimated in the order of 10^{-6} m/s, indicating a higher permeability of the native soils. The results are shown below:

Monitoring Well ID	Top of Well Screen Elevation (masl)	Bottom of Well Screen Elevation (masl)	Screened Geological Units	Hydraulic Conductivity (m/s)
BH 2	226.0	222.9	Sand	4.1×10^{-6}
BH 3	230.7	227.7	Sand	5.8×10^{-6}
BH 4	226.3	223.2	Sand	2.2×10^{-6}
BH 5	233.7	230.6	Sand	6.1×10^{-6}

Note: masl: meters above sea level, mbgs: meters below ground surface

The prevailing ground water conditions at the Property are as follows:

Ground Water Conditions	
Highest Ground Water Elevation (masl)	±234.4 masl

Ground Water Control

The water taking requirements for ground water control at the Property are established only from rainfall events since there is no groundwater seepage. The results are shown below:

* Rainfall event data obtained from Ministry of Transportation IDF Curve

Ground Water Quantity: Short Term (Construction) – S.F. 2.0 Used						
Location	Ground Water Seepage		26.9 mm Design Rainfall Event*		Total Volume	
	L/day	L/min	L/day	L/min	L/day	L/min
<i>Five-story slab-on-grade building</i>	0	0	1,000	0.7	1,000	0.7
Regulatory Requirements						
Environmental Activity and Sector Registry (EASR) Posting			Not Required			
Short Term Permit to Take Water (PTTW)			Not Required			
Long Term Permit to Take Water (PTTW)			Not Required			

Note: For further details on the geotechnical design considerations and constructability, please refer to the geotechnical report prepared by Terraprobe under a separate cover (**File No. 3-21-0182-01**)

7.2 Impact Assessment of Water Quality Threats

A drinking water threat is defined in the Clean Water Act as “an activity or condition that adversely affects or has the potential to adversely affect the quality or quantity of any water that is or may be used as a source of drinking water.” The Clean Water Act, 2006 prescribes a number of land use that are considered to be drinking water threats. Based on the **web-based Threats Analysis Tool** developed by The Upper Thames River Conservation Authority following prescribed threats could be significant:

Threat #	Threat Sub Category	Risk	Vulnerable Area / Score	Circumstance	Parameter of Concern
16	Handling and Storage of a Dense Non-Aqueous Phase Liquid (DNAPL)	Significant	WHPA-B (6)	- The below-grade handling of a DNAPL in relation to its storage (Any Quantity) - The handling of a DNAPL at or above grade, in relation to its storage (Any Quantity) - The storage of a DNAPL at or above grade (Any Quantity) - The storage of a DNAPL completely below grade (Any Quantity) - The storage of a DNAPL partially below grade (Any Quantity)	Chemical

The Ministry of Environment Tables of Drinking Water Threats identifies five (5) types of dense non-aqueous phase liquids that could make their way into surface and groundwater as a result of a spill from

the handling or storage of these chemicals. The following chemicals could threaten the safety of drinking water sources in certain situations:

- 1,4-Dioxane
- Tetrachloroethylene (Perchloroethylene [PCE])
- Trichloroethylene [TCE]
- Vinyl Chloride [VC]
- Polycyclic Aromatic Hydrocarbons [PAHs]

As required under the Clean Water Act, any use or storage of Dense Non-Aqueous Phase Liquids (DNAPLs) will be prohibited within WHPA-A and WHPA-B zones across the Site as these are considered to be significant drinking water threats. Any amount of dense non-aqueous phase liquids is considered a significant threat within a WHPA-A, B, C or C1 (where the vulnerability score is greater than or equal to 2). Based on the nature of the proposed development, the use or storage of DNAPLs is not anticipated at the Site

Handling and storage of household hazardous waste disposal, maintaining fuel oil tanks and septic systems, maintaining or decommissioning a well increases the risk for contaminating the aquifer; however, the proposed development will be serviced by the City of Barrie water supply and sanitary sewer systems. Therefore, none of the above-mentioned concerns applies to the Site.

7.3 Groundwater and Surface Water Quality Impact

The area of the proposed development is located in the Highly Vulnerable Aquifer (HVA). HVAs are those areas where an aquifer may be more prone to contamination. These areas have been identified where there is little or no protection from an overlying aquitard (a protective layer of low permeability materials). The Property is also mapped under Issue Contributing Area for Chloride. Best management practices for the use of salt for winter driveways maintenance should be adopted that will reduce the potential to affect underlying ground water resources. The risk management plan for water quality threat management is provided under Section 7.0.

Based on the review of Risk Management comments dated: January 13, 2021, it is indicated that infiltration of rooftop runoff and vegetated surfaces may be permitted if current site conditions demonstrate no existing contamination. Depending on the land use, runoff from urban developments may contain a variety of dilute contaminants such as suspended solids, chloride from road salt, oil and grease, metals, pesticide residues, bacteria and viruses. For groundwater, generally with the exception of the dissolved constituents such as nitrogen and salt, most contaminants are attenuated by filtration during groundwater flow through the soils. Under the proposed development, the quality of water directed to pervious areas for infiltration is not expected to contain any contaminant of concerns. Considering the nature of the proposed development, it is anticipated that there will not be any contributing contaminants that may affect the background ground water quality in the area.

The nearest surface water features are a tributary of Dyments Creek located approximately 50 m south. Any groundwater that will be taken from the site will be discharged (if required) into the City's sewer systems and not into any natural water body. As such, there will be no impact to surface water features as

a result of the proposed development.

Furthermore, at the time of preparation of this report, Terraprobe is not aware of any Environmental Site Assessments (ESAs) conducted at the Property.

7.4 Anthropogenic Transport Pathways

No significant anthropogenic transport pathways were identified during investigations conducted on the Property. Six (6) monitoring wells were installed during the site investigation conducted by Terraprobe to gather information regarding the groundwater quality and elevation at the Property. The well depths range from 4.6 mbgs to 12.2 mbgs installed in the shallow groundwater and are therefore not considered to be a risk to the groundwater resource. The monitoring wells installed at the Site need to be maintained in accordance with Ontario Water Resources Act, O. Reg. 903/90. When the wells are no longer required for monitoring or sampling, these wells will need to be appropriately decommissioned by a licensed well contractor as outlined in the Regulation. Based on the nature of the proposed development and structure, the excavation works will not present a transport pathway to the municipal groundwater aquifer.

Based on the review of Risk Management comments dated: January 13, 2021, it is indicated that two (2) potential wells are located on the site (Map ID: 12 and 27). The Well ID 5718624 (Map ID: 12) is registered for commercial use constructed in 1983. However, the Well ID 7260721 (Map ID: 27) is registered as a test hole for 636 Dunlop St West, Barrie. Therefore, the applicant is required to maintain or decommission the well located on the Site in accordance with O.Reg. 903 and Ministry of the Environment, Conservation and Parks (MECP) standards to ensure the structural integrity of the wells and to prevent them from becoming a transport pathway for any possible contamination.

7.5 Impact to Local Groundwater Resources

The proposed development will be municipally serviced, and there is no proposed on-site groundwater use for the development. Based on the review of Risk Management comments dated: January 13, 2021, it is indicated that building supports are not permitted to be constructed within the municipal supply aquifer.

As indicated earlier, one (1) active Barrie municipal production well (PW-19, Well ID: 7114203, Map ID: 22) is located within 500 m of the Site screened in lower A3/A4 aquifer. Based on the review of Oak Ridges Moraine Groundwater Program (OGRMP) cross-section tool and source water comments, it is noted that the confined municipal aquifer is found at approximately ± 75 meters below grade. The proposed development scheme will be a slab-on-grade structure. The excavation activities will not extend to the underneath confined municipal aquifer. Moreover, the zone of influence during any dewatering activities (if applicable) will be limited in its lateral extent and will not extend horizontally to the location of the municipal production wellhead.

Based on the well record review and nature of the proposed development, it is unlikely that impacts from the proposed development will affect municipal water-taking resources in the study area.

8.0 RISK MANAGEMENT PLAN AND MITIGATING MEASURES

8.1 Water Quality Threat Management

8.1.1 Groundwater Recharge Threat Management

The existing groundwater recharge rates at the Site are approximated as 278 mm/yr. These recharge rates are based on the property-specific conditions. This recharge occurs in a broad diffuse manner over the entire Site. Based on the borehole findings, below any surficial materials (topsoil or pavement structure), the subsurface conditions generally consist of loose sand fill with peat organics and or loose an organic peat layer extending to a depth of up to 1.5 m below grade. These materials are underlain by the native sand deposit with permeability estimated in the order of 10^{-4} m/s to 10^{-6} m/s, indicating higher permeabilities which indicates a possible contribution to the groundwater recharge to shallow groundwater zone. Therefore, management of groundwater recharge at the Site following development is recommended where it is feasible and should be designed using a Best Management Practice approach.

It is anticipated that groundwater recharge at the Site can be maintained through the use of appropriate low-impact development (LID) techniques. Appropriate stormwater control measures would be required to prevent erosion due to increased runoff, for slow and delayed-release to promote infiltration and prevent flooding. The implementation of these measures at the Site will assist in maintaining the shallow groundwater recharge and replicate the existing surface drainage. Runoff from the proposed development will conform to the stormwater management report for the Site.

As per CVC-TRCA (2010), the recommended vertical separation between the base of the given infiltration augmentation option and the high groundwater table is at least one meter; however, distances of less than one meter of separation in soils having higher infiltration potential may still be effective. Based on the available water level recordings till date, it is noted that groundwater level in the overburden varies from $232.7 \pm$ masl to $233.7 \pm$ masl (1.5 mbgs to 1.9 mbgs). The suitability of using the proper infiltration options within the Site will be provided in the Stormwater Management Report/Functional Servicing Report. Final design of the LID measures should be reviewed in conjunction with the stormwater management plan for the Site.

Furthermore, as indicated earlier, the water balance calculations show that development has the potential to reduce the natural infiltration by $668 \text{ m}^3/\text{yr}$. Conservation Ontario Guidelines (Conservation Ontario, 2013) suggest a target of 80% of the predevelopment infiltration being maintained in the post-development conditions. Final design of the infiltration measures provided in the FSR report should address these infiltration targets estimated in water balance calculations.

Furthermore, the soils present on the property are of higher permeabilities which would provide potential pathways for groundwater discharge. The future development will also be connected to municipal servicing through underground utility lines. The future development may disrupt the shallow groundwater flow, and its continuity should be maintained where practical. However, it is recommended that the excavation of the building foundation and any underground services or utilities across the site must be backfilled using material of similar permeabilities.

8.1.2 Groundwater Taking Threat Management

The construction dewatering rate and duration is typically governed by the PTTW or EASR. If a PTTW or EASR is required, a monitoring plan will be prepared as part of an Environmental Management Plan (EMP) and implemented during dewatering to track the zone of influence and manage dewatering discharge quality and quantity. Monitoring programs generally involve a combination of: Installation of dedicated monitoring wells; Pre, during, and post-construction water level monitoring and groundwater quality sampling and testing; and dewatering discharge quality sampling and testing

8.2 Water Quality Threat Management

8.2.1 Prohibited Uses

Since the Property is located in a WHPA-B, the handling and storage of a dense non-aqueous phase liquid (DNAPL) (for example, paints, paint-thinners, degreasers, solvents, etc.) as listed in the *Tables of Drinking Water Threats, Clean Water Act, 2006, Ontario Ministry of the Environment (as amended Nov 16, 2009)* is prohibited at the Property.

Future tenants will need to be aware of these requirements, the Property owner will be required to have a sale and purchase agreement clause and/or a lease clause that informs future purchasers or leases that the property is within a wellhead protection area and subject to source protection policies under the Clean Water Act, 2006.

8.2.2 Application of Road Salt

8.2.2.1 Preventive/Mitigation/Management Measures – Road Salt

All salt stored at the Site for later application shall be stored in water-impermeable containers and in roofed areas of the Property that either are asphalt-paved or have a poured concrete floor to minimize entry into the subsurface. Stormwater runoff at the impermeable surfaces, roadways, walkways, and driveways on the Site will drain to catch basins at the public roadways. This will preclude downward migration into the aquifer.

The Transportation Association of Canada (TAC) has produced a document titled *Syntheses of Best Practices – Road Salt Management (2013)*. The Best Management Practices should be generally followed at the Site unless prohibited. In addition, best management practices for contractors, residents, and the community are provided by the not-for-profit organization Smart About Salt Council, and their recommendations may be of benefit in reducing salt loads. Some items recommended include maintaining health and safety as a top priority, training and education for road salt management, salt application rates, and monitoring of releases from the salt application. It is recommended that a Contractor who is certified by Smart About Salt be retained.

Practices of road salt management to be considered so as to reduce the amount of salt needed and minimize associated groundwater and surface water impacts include (TAC, 2013):

- The 4 R's of snow and ice control:
 1. Right Material – the right material will depend on the conditions being treated. In situations where the pavement temperature is extremely cold, chemicals with lower working temperatures and sand/salt mixtures may be warranted.
 2. Right Amount – The right amount of material is dependent upon the type of slippery condition being treated, the amount of residual chemical on the pavement surface, the expected pavement temperature and the amount of precipitation that is expected.
 3. Right Place – Precise placement of materials is important to keep it in the right place to do the job rather than waste the environment. Proper material placement requires the right equipment and skilled operators.
 4. Right Timing – The timing of salt placement is important to minimizing waste and maximizing chemical effectiveness. There are times when the pavement temperature is, and is expected to remain, above freezing and therefore may not warrant salt application. Proactive anti-icing is key to achieving safer conditions quickly with less salt.

Application of a Road Salt Alternative - Calcium Magnesium Acetate: Calcium Magnesium Acetate has a low environmental impact but can contribute to biochemical oxygen demand (BOD). It also has a high purchase cost relative to NaCl. In addition, Potassium acetate (KA) is often used as a base for commercial chloride-free liquid de-icer formulations as a road salt alternative, having low corrosion, relatively high performance, and a low environmental impact. The above risk management and risk mitigation measures should be considered for the proposed development.

- The mechanical removal of snow from the treatment area prior to the application of a freeze point depressant (e.g., salt) to minimize the amount of material needed and the potential for dilution and re-freeze.
- The application of freeze point depressants after plowing only when pavement temperatures are below freezing, and the remaining snow/ice that could not be removed by mechanical means presents a hazard.
- The use of liquid rather than solid salt in the right conditions to speed up the melting process.
- The application of only enough material to do the job.
- The location of snow storage such that meltwater that may contain salt is not directed towards salt vulnerable areas.
- The understanding is that salt should not be used to promote the rapid melting of stockpiled snow.

8.2.2.2 Monitoring, Emergency Response, Financial Assurance and Communication and Implementation Plan – Road Salt

The threats to drinking water quality after the development is completed primarily involve the use of road salt. Given that road salt is typically applied in a solid form, immediate clean-up should be conducted by Site maintenance staff in the event of a spill. Based on the negligible threat posed to water quality in the nearby

municipal wells by application of road salt at the Property, monitoring, emergency response, financial assurance and a communication and implementation plan are not considered to be necessary for this prescribed threat.

8.2.3 Storage of Fuels and Chemicals - Temporary

During construction of the proposed building, it may be necessary to temporarily store fuels and/or chemicals at the Site. This represents a potential threat to groundwater quality, as a spill of significant size may potentially impact the local water supply. The fuel will be stored in a small quantity after construction for the on-site standby generator.

8.2.3.1 Preventive/Mitigation/Management Measures – Temporary Storage of Fuels and Chemicals (During and Post Construction)

The fuels will only be stored (maximum size of 2500L is assumed) as part of the construction activities and will be removed once the construction is completed. During construction, it is expected that fuels such as gasoline and diesel, and other chemicals may be temporarily stored on Site for construction. There is no significant downward pathway into the underlying deep aquifer. This is a result of the hydraulic isolation of the shallow groundwater from the deep aquifer in which municipal wells are installed.

To further prevent and mitigate any spills at the Site, it is recommended to place temporary fuel and chemical storage containers of significant size into secondary containment such that a leak can be contained. The temporary fuel and chemical storage containers should also be protected from vehicle strikes. Appropriate spill kits should be maintained at various locations throughout the Site, and an emergency spill response plan should be developed to outline actions to be taken in case of a spill or leak. Best Management Practices should be used during and post-construction when handling these items.

The fuel will also be stored in a small quantity after construction for the on-site standby generator. Given the small quantity of fuel storage in the post-construction period, it does not pose any threat to groundwater quality. Please refer to the Sections below for details related to mitigation, monitoring and emergency responses.

8.2.3.2 Monitoring, Emergency Response, Financial Assurance and Communication and Implementation Plan – Temporary Storage of Fuels and Chemicals (During and Post Construction)

The monitoring and emergency response measures recommended to be implemented at the Property include the following:

Monitoring

- Prior to beginning construction, spill kits should be available on the Site with absorbent material for spills containment. Contractors should be properly trained in the use of the spill kit materials. Equipment and materials to handle spills should be properly maintained at all times.
- It is recommended that temporary fuel and chemical storage locations be inspected on a regular basis to ensure the integrity of storage containers.

Emergency Response

For the fuel spills, the following steps can be taken:

- If a spill occurs, it should be reported to the contractor's onsite supervisor. The contractor should assess if the spill could be of environmental significance (based on material, quantity, spill-receiving body, and potential sensitive receptors).
- If there is an injury on-site, a 911 call should be made in addition to notifying any Site-emergency personnel.
- All steps should be taken to contain the liquid at the source. If it has already entered (or has the potential to enter) any existing ditch, the ditch should be blocked further downstream to contain the spill and ensure it does not migrate offsite.
- The MECP Spills Action Centre SAC (1-800-268-6060) should be contacted immediately.
- Contact a remediation company to clean up the spill. The remediation contractor will also be responsible for the removal and disposal of hazardous materials from the Site.

For further information related to environmental spills, please visit:

<https://www.barrie.ca/Living/Environment/Pages/EnvironmentalSpills.aspx>

9.0 CLOSURE

We trust this report meets with your requirements. Should you have any questions regarding the information presented, please do not hesitate to contact our office.

Yours truly,

Terraprobe Inc.



Syed Ali, EIT
Project Manager



Muna Mirghani, P.Eng.,
Project Manager-Hydrogeology



Samuel Oyedokun, P. Eng., PMP., QP_{ESA}
Associate, Senior Project Manager



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LIMITATIONS

This report was prepared by Terraprobe Inc. for the use of **25707517 Ontario Inc. c/o Innovative Planning Solutions**, and is intended to provide an assessment of the hydrogeological condition on the property located at **642 Dunlop Street West, Barrie, ON**. 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. Terraprobe accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report, including consequential financial effects on transactions or property values, or requirements for follow-up actions and costs.

The assessment should not be considered a comprehensive audit that eliminates all risks. The information presented in this report is based on information collected during the completion of the subsurface investigation conducted by Terraprobe Inc. It is based on conditions at the property at the time of the site inspection. The subsurface conditions were assessed based on information collected at specific borehole and monitoring well locations. The actual subsurface conditions between the sampling points may vary.

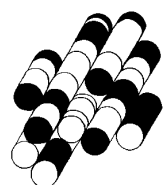
There is no warranty expressed or implied by this report regarding the condition of the property. Professional judgment was exercised in gathering and analyzing information collected by our staff, as well as that submitted by others. The conclusions presented are the product of professional care and competence, and cannot be construed as an absolute guarantee.

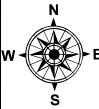
In the event that during future work new information regarding the condition of the property is encountered, or the proposed development is changed from that which was provided to Terraprobe with respect to the property, Terraprobe should be notified in order that we may re-evaluate the findings of this assessment and provide amendments, as required.

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FIGURES

TERRAPROBE INC.

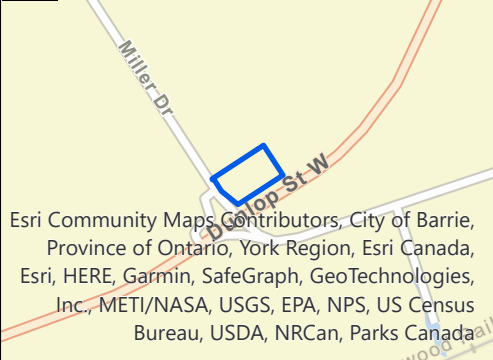




References:

ESRI, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus Ds, USDA, USGS, AeroGRID, IGN, and the GIS User Community, Basemaps

Key Map



Notes:

Legend:

 Approximate Site Boundary

Project Title:

Hydrogeological Assessment

Site Location:

642 Dunlop Street West,
Barrie, Ontario

Figure Title:

Site Location Plan

Designed By:

MM

File No.:

3-21-0182-46

Drawn By:

SSK

Scale:

As Shown

Reviewed By:

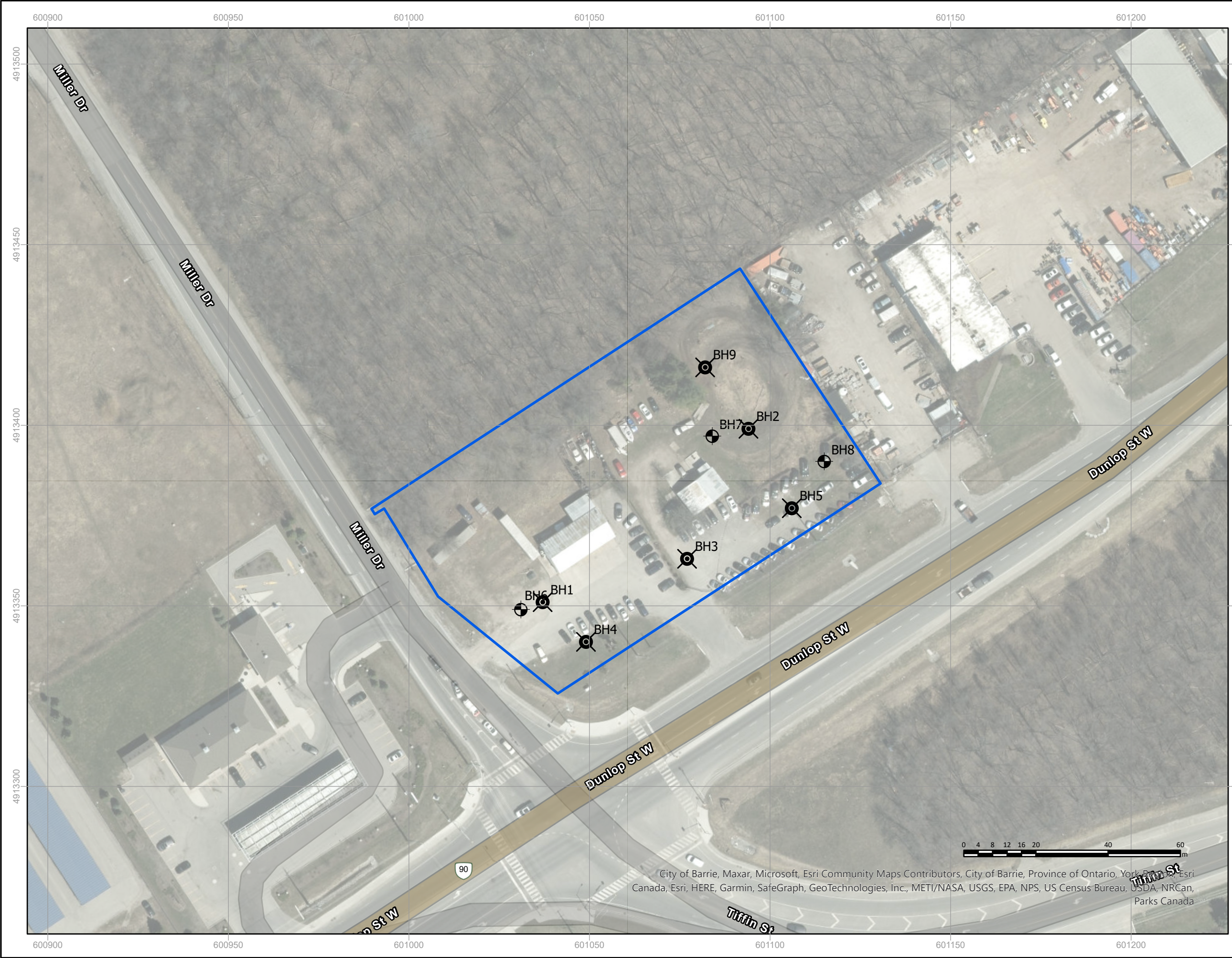
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Date:

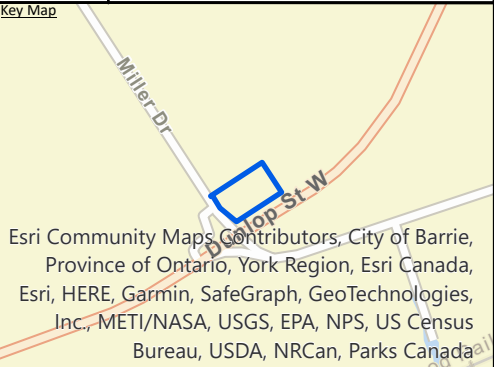
February 2022



References:

ESRI, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus Ds, USDA, USGS, AeroGRID, IGN, and the GIS User Community, Basemaps

Key Map



Notes:

Legend:

- Approximate Site Boundary
- Approximate Monitoring Well Location
- Approximate Borehole Location

Project Title:

Hydrogeological Assessment

Site Location:

642 Dunlop Street West,
Barrie, Ontario

Figure Title:

Borehole and Monitoring Well Plan

Designed By:

MM

File No.:

3-21-0182-46

Drawn By:

SSK

Scale:

As Shown

Reviewed By:

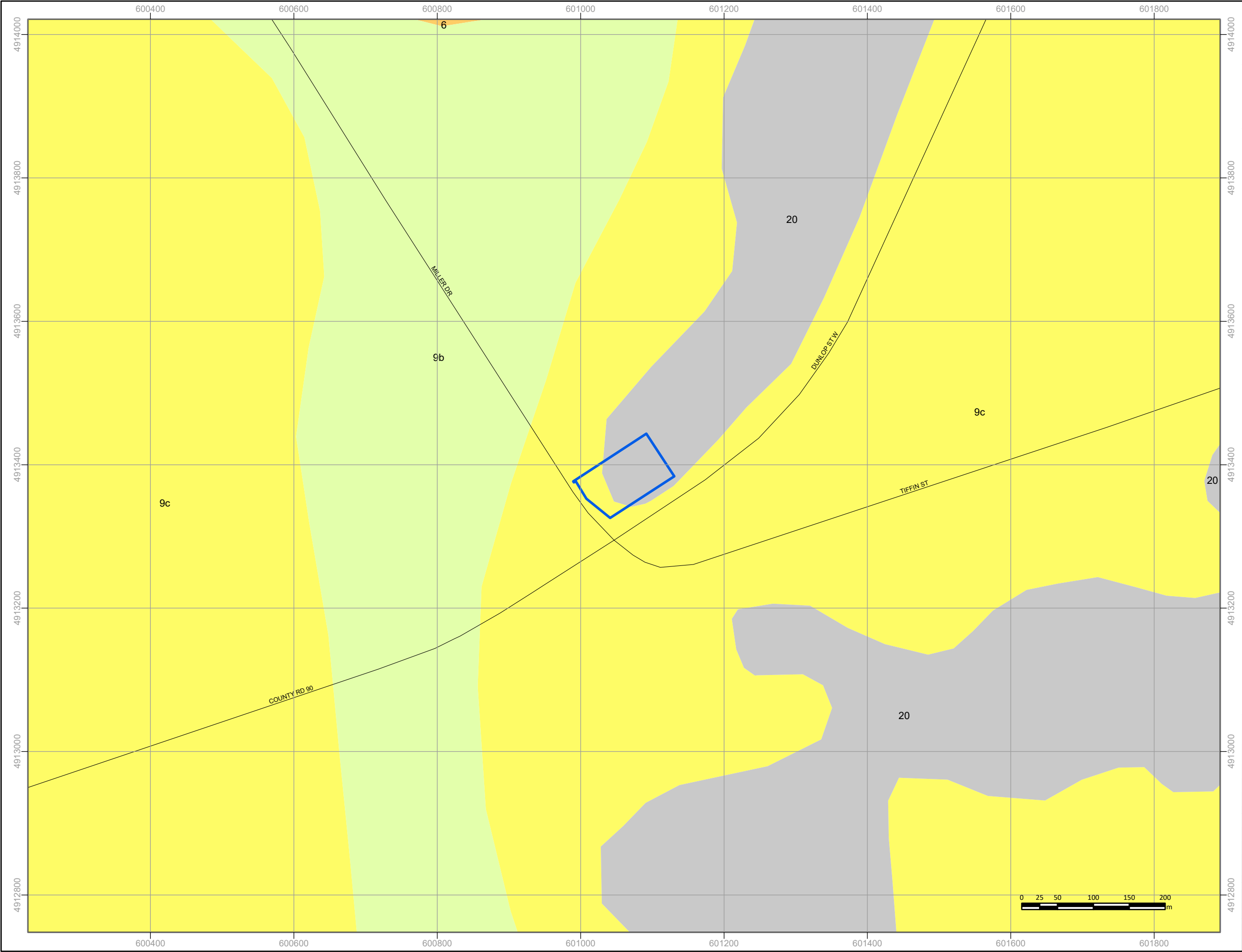
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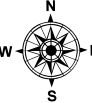
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February 2022





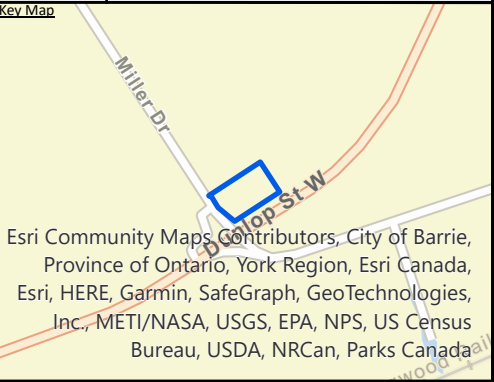
Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing



References:

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Key Map



Esri Community Maps, Contributors, City of Barrie, Province of Ontario, York Region, Esri Canada, Esri, HERE, Garmin, SafeGraph, GeoTechnologies, Inc., METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, NRCan, Parks Canada

Notes:

Legend:

-  Approximate Site Boundary
-  6: Ice-contact stratified deposits
-  9b: Littoral-foreshore deposits
-  9c: Foreshore-basinal deposits
-  20: Organic deposits
-  Collector

Project Title:

Hydrogeological Assessment

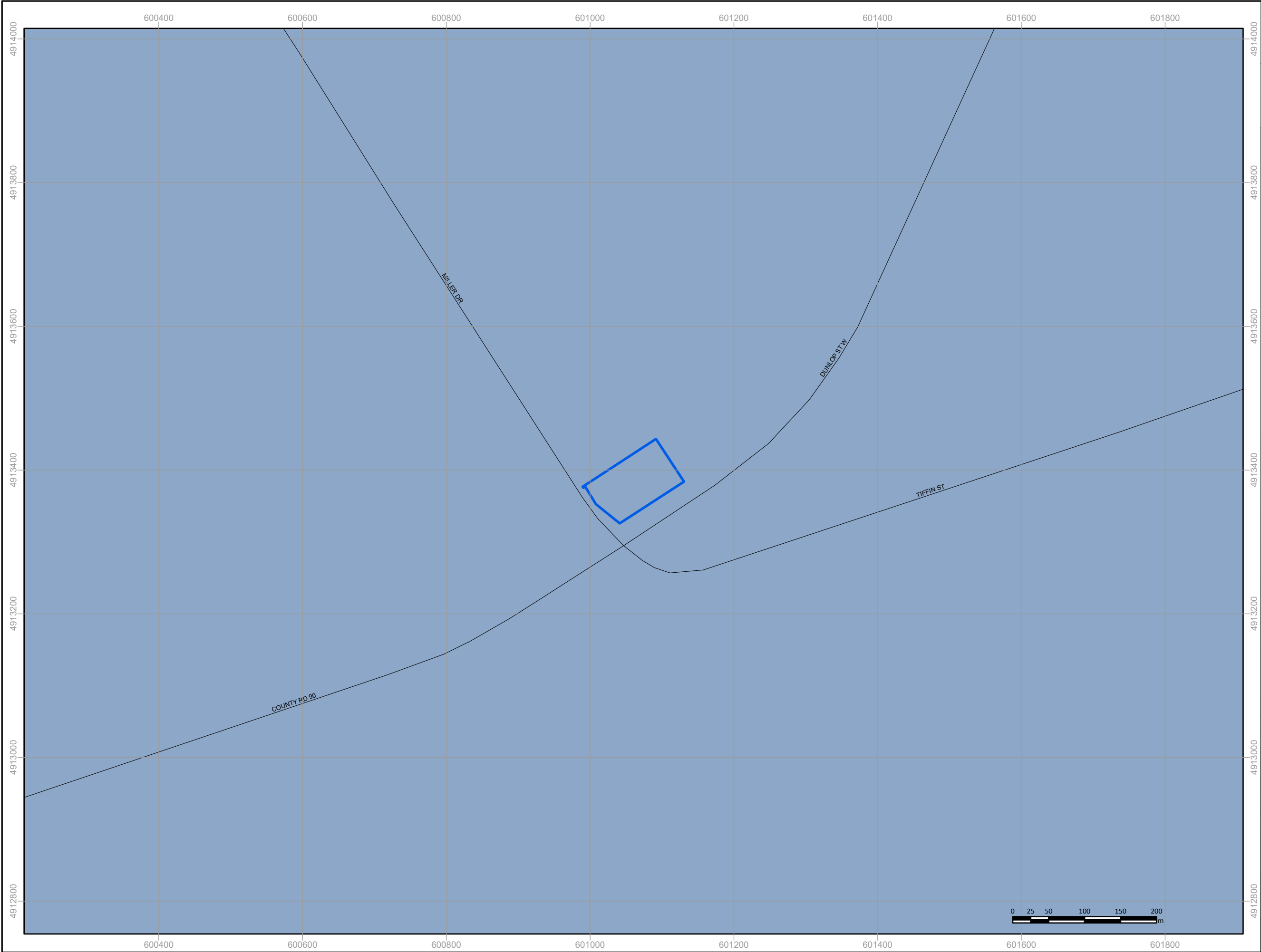
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Figure Title:

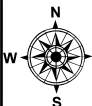
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Drawn By: SSK	Scale: As Shown
Reviewed By: SO	Figure No.: 3
Date: February 2022	





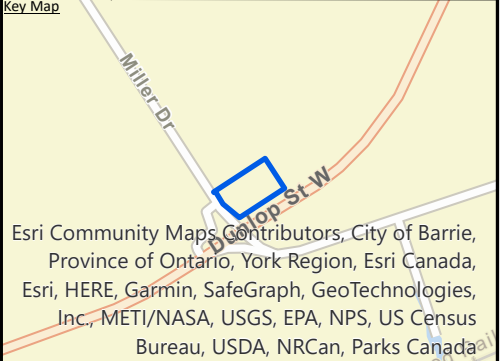
Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing



References:

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Notes:

Legend:

-  Approximate Site Boundary
-  Collector
-  35, Simcoe Lowlands

Project Title:

Hydrogeological Assessment

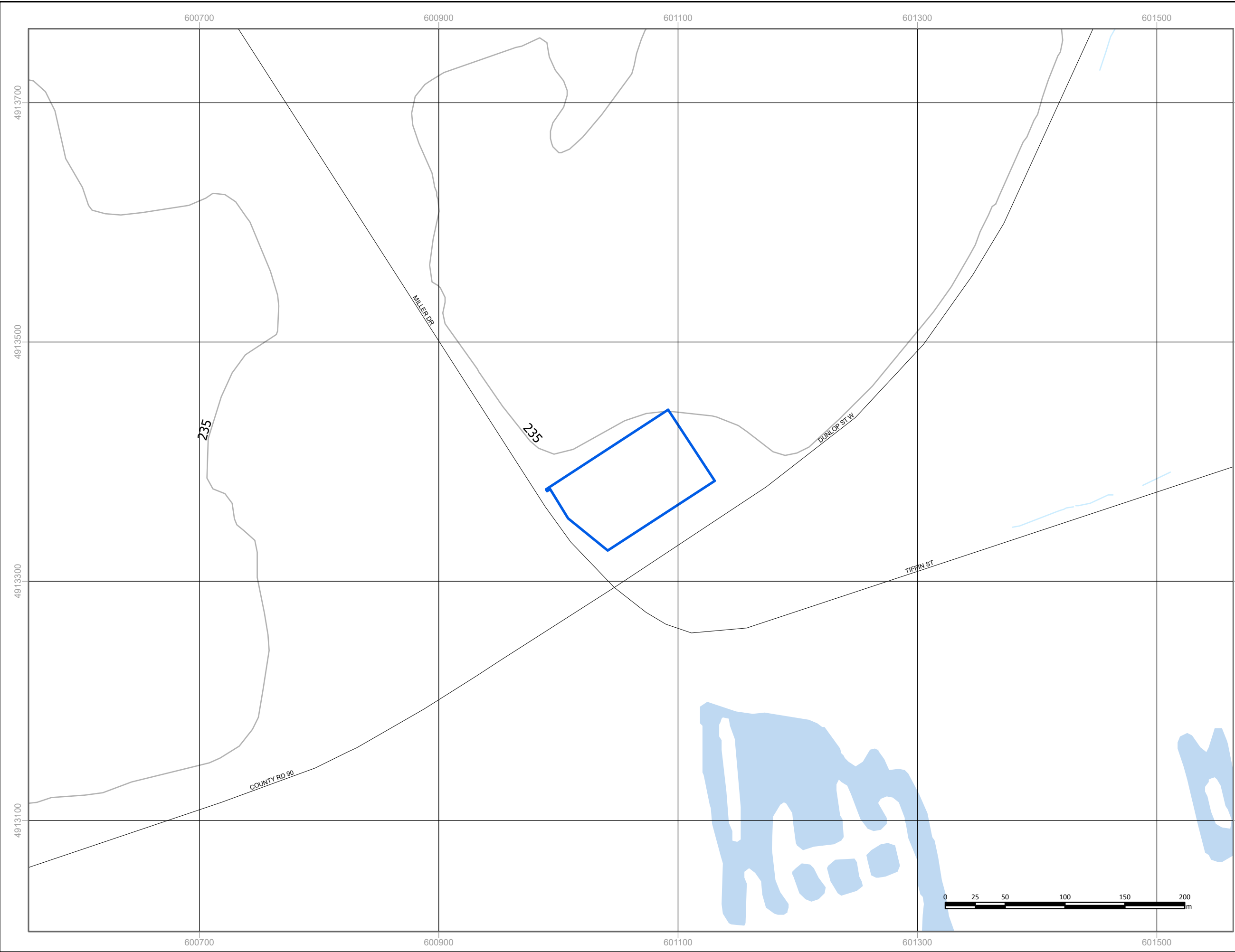
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Barrie, Ontario

Figure Title:

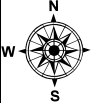
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Drawn By: SSK	Scale: As Shown
Reviewed By: SO	Figure No.: 4
Date: February 2022	

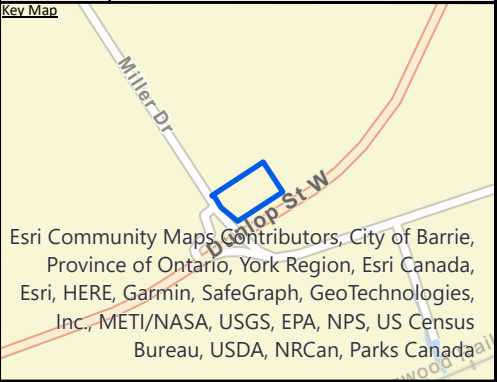




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Notes:

Legend:

- Approximate Site Boundary
- City of Barrie; Topographic Contours
- Water Body
- Water Course
- Collector

Project Title:

Hydrogeological Assessment

Site Location:

642 Dunlop Street West,
Barrie, Ontario

Figure Title:

Topography Map

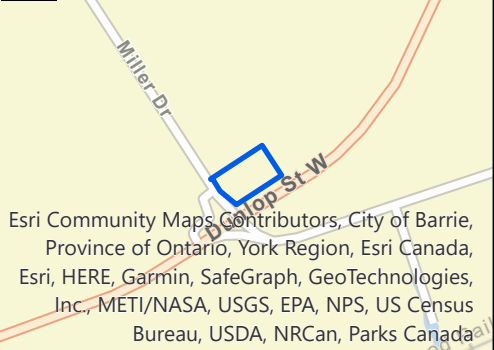
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Drawn By: SSK	Scale: As Shown
Reviewed By: BW	Figure No.: 5
Date: February 2022	



References:

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Key Map



Notes:

Legend:

- Approximate Site Boundary
- Collector
- Water Course
- Wetland Features-Evaluated-Provincial
- Wetland Features-Not evaluated
- Wooded Area

Project Title:

Hydrogeological Assessment

Site Location:

642 Dunlop Street West,
Barrie, Ontario

Figure Title:

Natural Heritage Feature Map

Designed By:

MM

File No.:

3-21-0182-46

Drawn By:

SSK

Scale:

As Shown

Reviewed By:

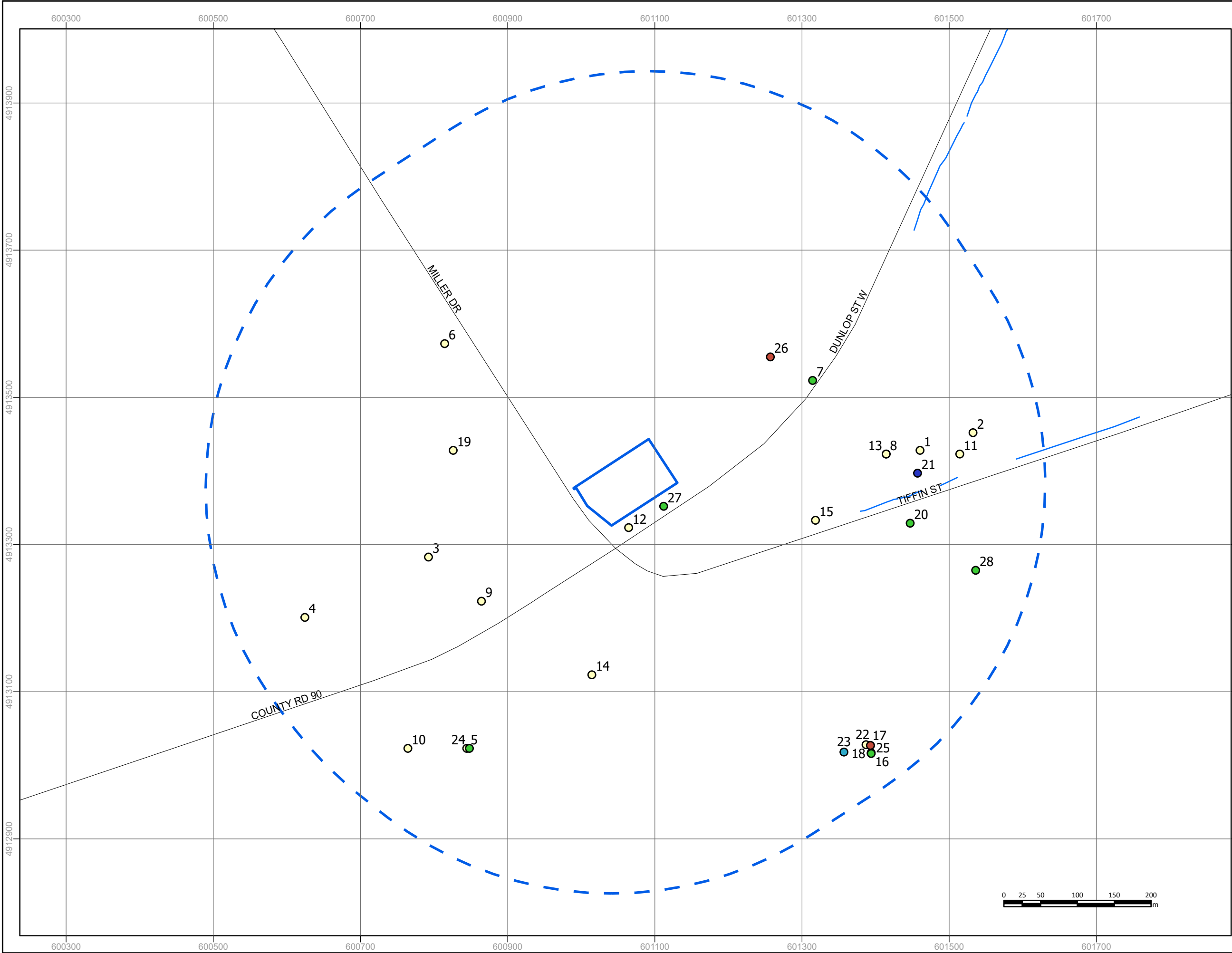
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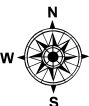
Date:

February 2022



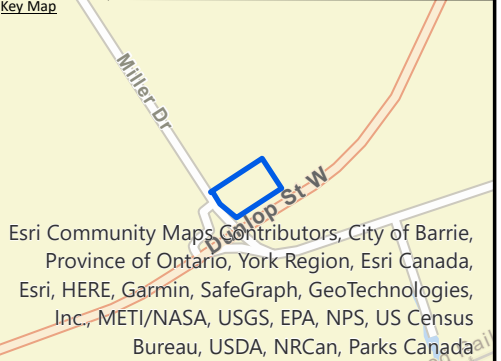


Consulting Geotechnical & Environmental Engineering
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References:
ESRI, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus Ds, USDA, USGS, AeroGRID, IGN, and the GIS User Community produced by Terraprobe Inc. Copyright (c) Queen's Printer 2020. Water Well Information System Ministry of the Environment, Conservation and Parks, 2020

Key Map



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Notes:

Legend:

- Approximate Site Boundary
- Approximate Study Area; 500m
- Water Course
- Collector
- Unknown
- Abandoned-Other
- Observation Wells
- Test Hole
- Water Supply

Project Title:

Hydrogeological Assessment

Site Location:

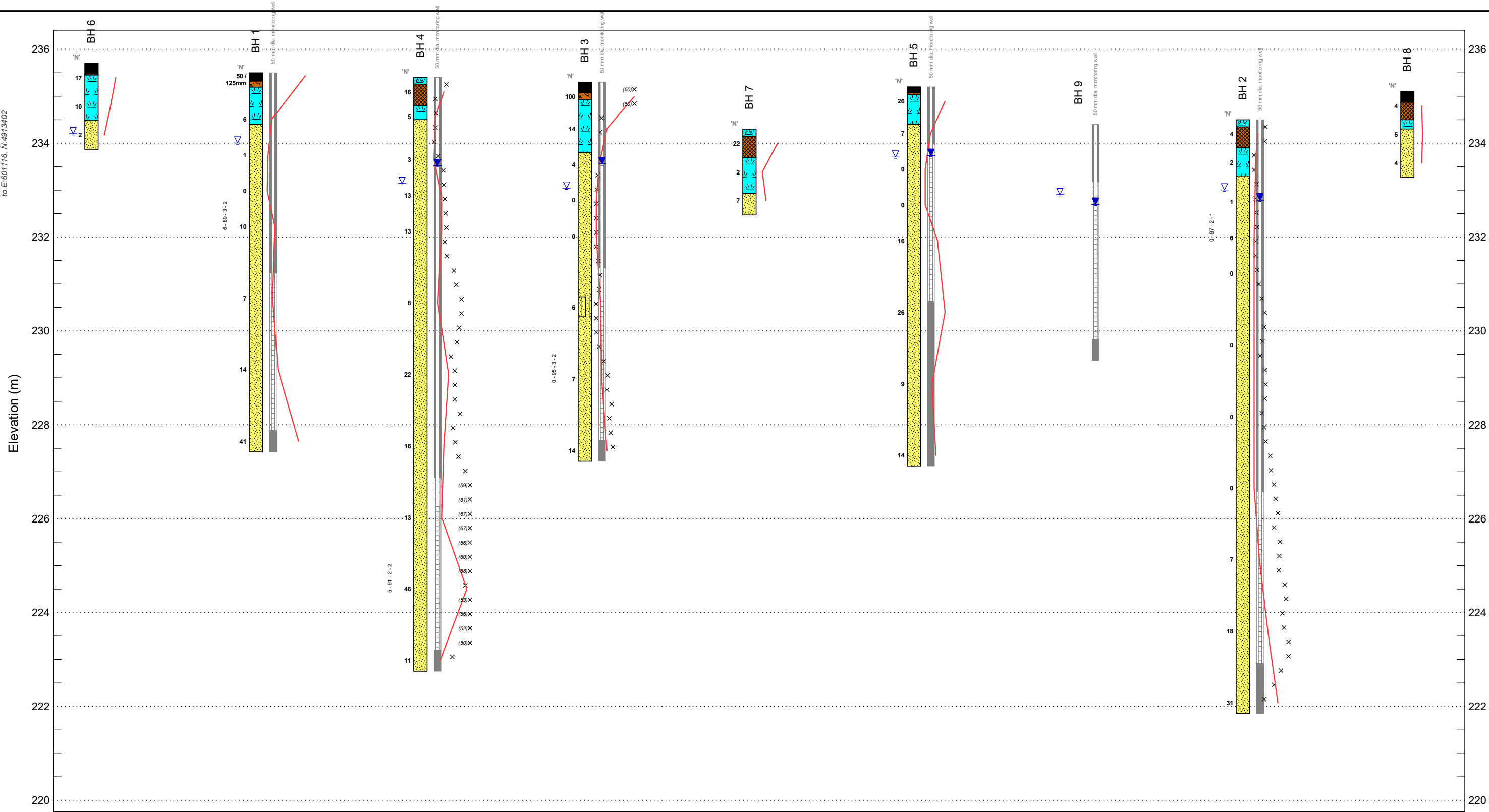
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Figure Title:

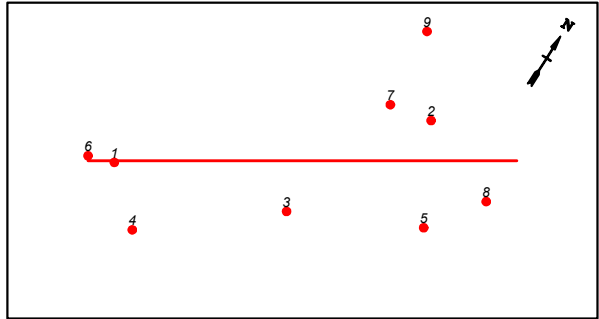
MECP Well Records Map

Designed By: MM	File No.: 3-21-0182-46
Drawn By: SSK	Scale: As Shown
Reviewed By: SO	Figure No.: 7
Date: February 2022	

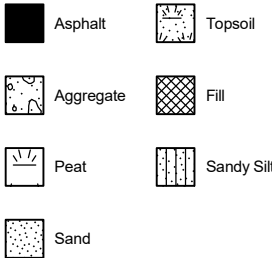
Alignment From E:601032, N:4913348,
to E:601116, N:4913402



SITE MAP

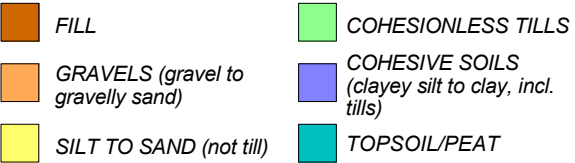
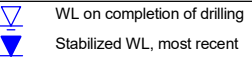


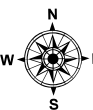
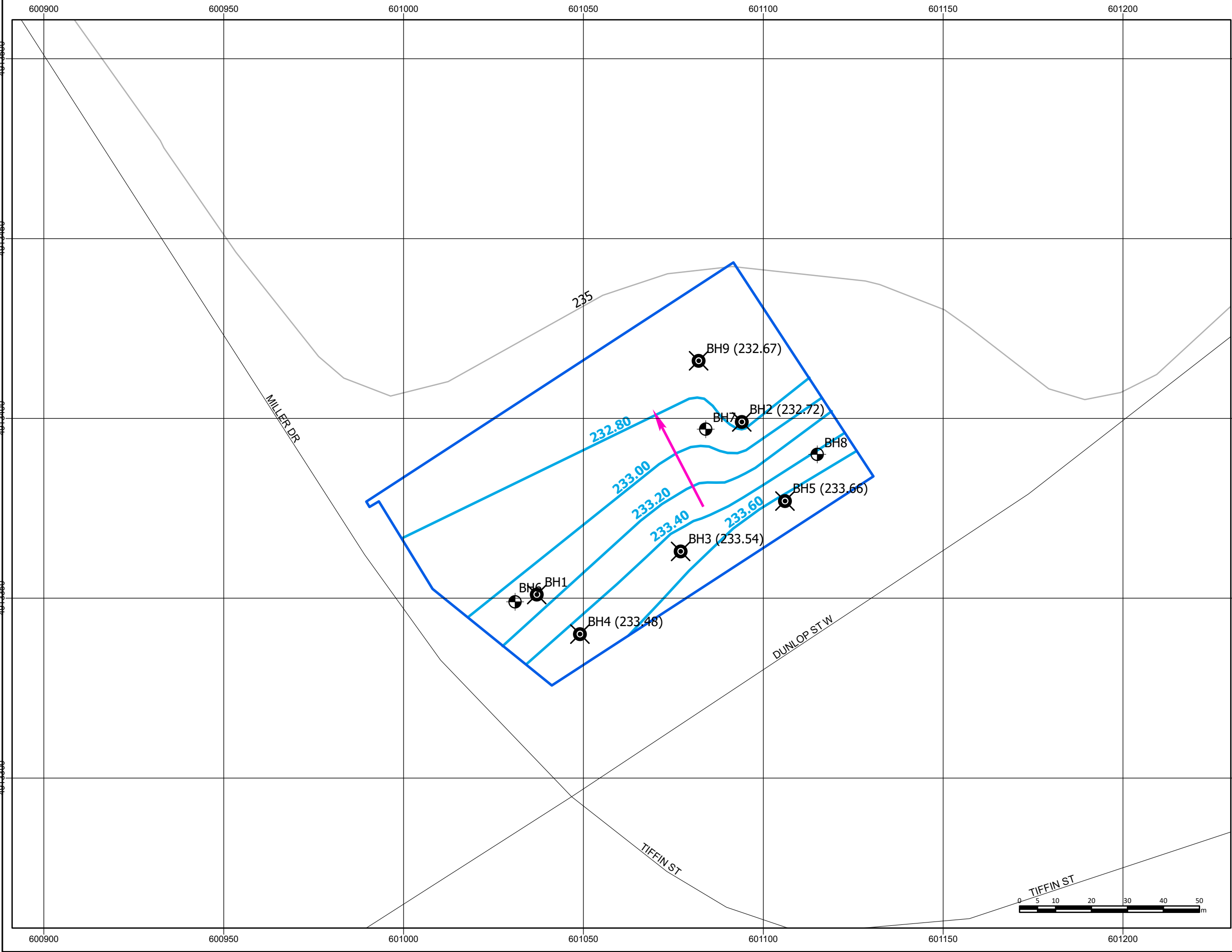
LITHOLOGY GRAPHIC LEGEND



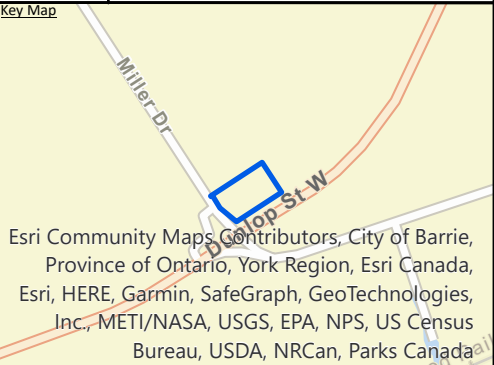
Boreholes Equally Spaced

INTERPRETIVE LEGEND





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Notes:

Legend:

- Approximate Site Boundary
- Approximate Monitoring Well Location
- Approximate Borehole Location
- Collector
- Interpreted Groundwater Contours
- Interpreted Groundwater Flow Direction
- City of Barrie; Topographic Contours

Project Title:
Hydrogeological Assessment

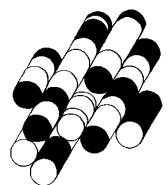
Site Location:
642 Dunlop Street West,
Barrie, Ontario

Figure Title:
Groundwater Flow Direction

Designed By: MM	File No.: 3-21-0182-46
Drawn By: SSK	Scale: As Shown
Reviewed By: SO	Figure No.: 8
Date: February 2022	

APPENDIX A

TERRAPROBE INC.



The Barrie Creeks, Lovers Creek and Hewitt's Creek subwatersheds

Figure 2-4

Legend

- Road
- Municipal Boundary
- Watercourse
-  Barrie Creeks
-  Hewitts Creeks
-  Lovers Creeks



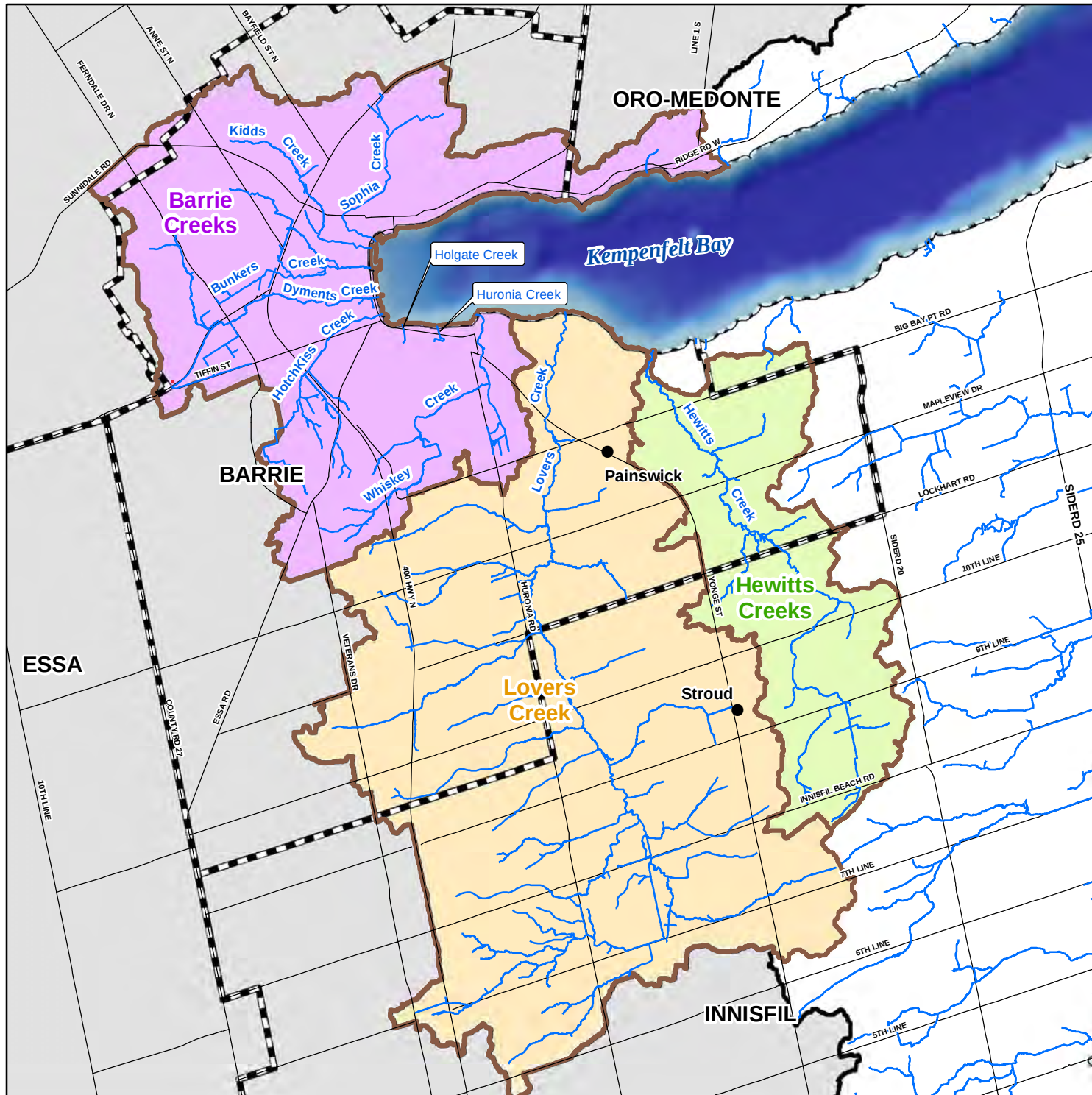
Lake Simcoe Region
conservation authority



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This map has been produced for illustrative purposes only.
LSRCA GIS Services DRAFT created July 2010.

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Features

-
- This aerial map displays the industrial area around Dunlop and Miller streets in St. Louis, Missouri. The map includes the following features:
- Streets:** Miller Dr, Dunlop St W, Tiffin St, and County Rd 90 are labeled. Specific addresses along these streets include 40 Miller Dr, 30 Miller Dr, 652 Dunlop St W, 642 Dunlop St W, 636 Dunlop St W, 630 Dunlop St W, 614 Dunlop St W, 551 Tiffin St, 557 Tiffin St, 653 Dunlop St W, and 551 Tiffin St.
 - Property Boundaries:** A red line outlines the area of interest, which includes the industrial site and surrounding properties.
 - Highlighted Site:** A cyan line outlines a specific industrial site located between Dunlop St W and Miller Dr, near the intersection of Dunlop St W and Tiffin St.
 - Landmarks:** A large, open, grassy area is visible in the upper left, and a large industrial building with a white roof is located near the intersection of Dunlop St W and Tiffin St.



Mapped By:

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Meters

106

0

53

106



Notes:
Natural Heritage Mapping



0.7 0 0.33 0.7 Kilometres Absence of a feature in the map does not mean they do not exist in this area.

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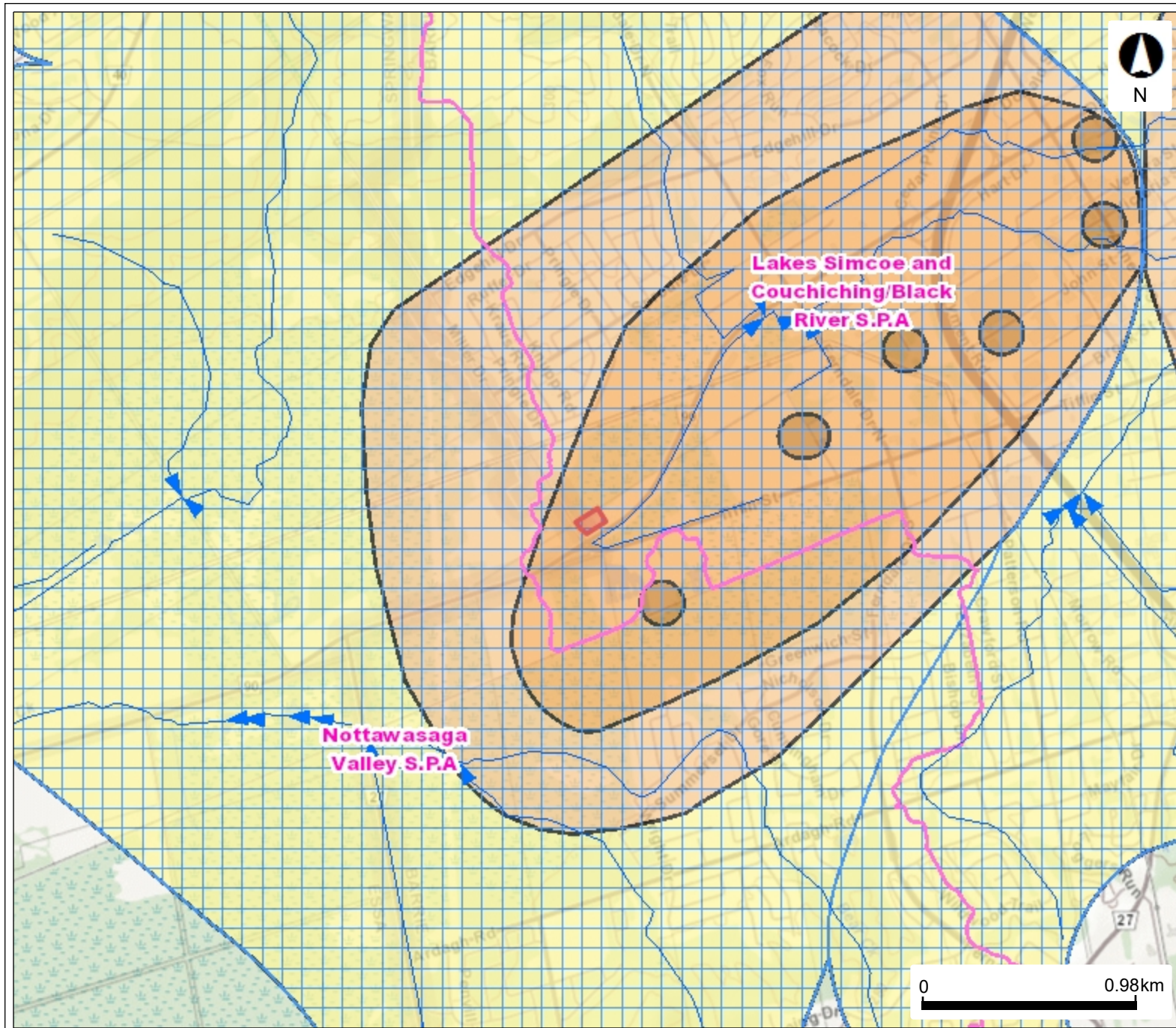
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GTA 2005 / SWOOP 2006 / Simcoe-Muskoka-Dufferin © FirstBase Solutions, 2005 / 2006 / 2008
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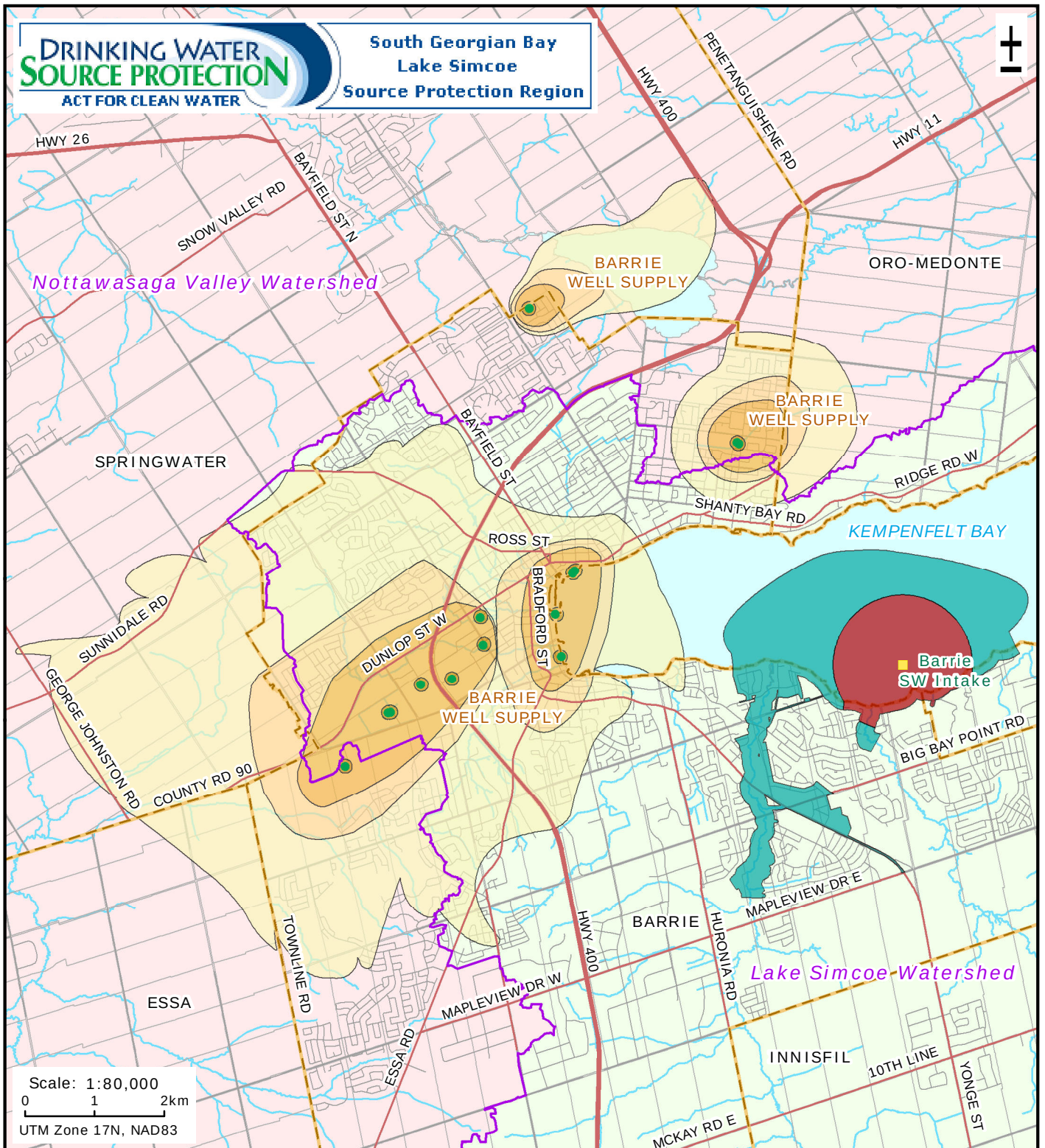
Source Water Protection Mapping



Legend

- Watercourse Direction
- Source Protection Areas
- Issue Contributing Areas
- Wellhead Protection Area**
 - A
 - B
 - C
 - C1
 - D
 - F
- Assessment Parcel with Address

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- Municipal Surface Water Intakes
- IPZ-1 (1000m on water or 120m inland)
- IPZ-2 (2 hr time of travel)
- Municipal Supply Well in City of Barrie
- WHPA-A (100m)
- WHPA-B (2-years time of travel)
- WHPA-C (5-years time of travel)
- WHPA-D (25-years time of travel)

**Drinking Water System
Vulnerable Areas in City of Barrie**

Golder Associates
Barrie, Ontario

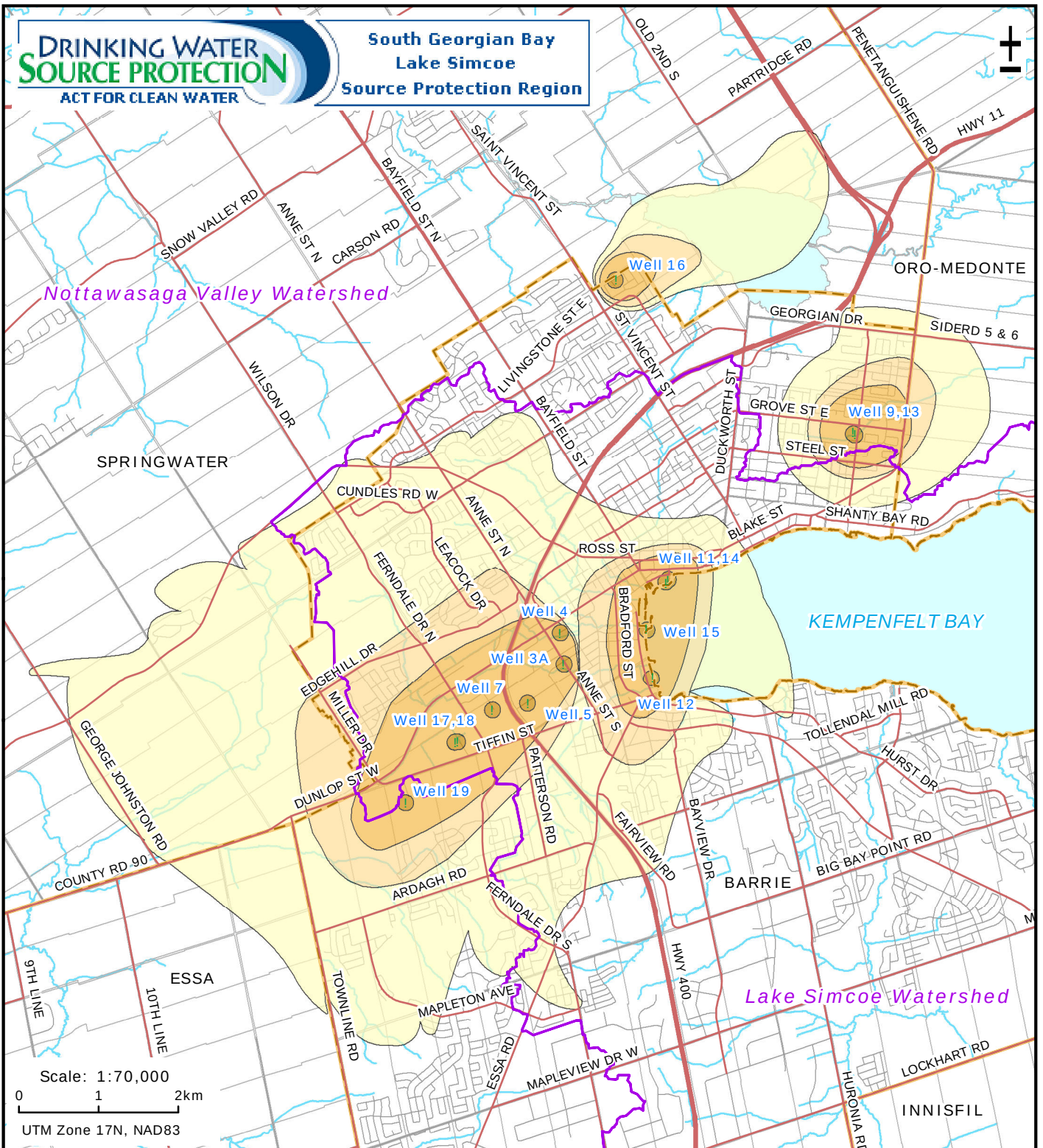
Created by: LSRCA
Date: 2014-04-10



This map was produced by the Lake Simcoe Region Conservation Authority, lead agency of the South Georgian Bay Lake Simcoe Region Source Protection Region. Base data have been compiled from various sources, under data sharing agreements. While every effort has been made to accurately depict the base data, errors may exist.



Figure 9-1



- Municipal Supply Well
- WHPA-A (100m)
- WHPA-B (2-years time of travel)
- WHPA-C (5-years time of travel)
- WHPA-D (25-years time of travel)
- SWP Watershed Area
- Municipality Boundary

**City of Barrie
Wellhead Protection Areas**

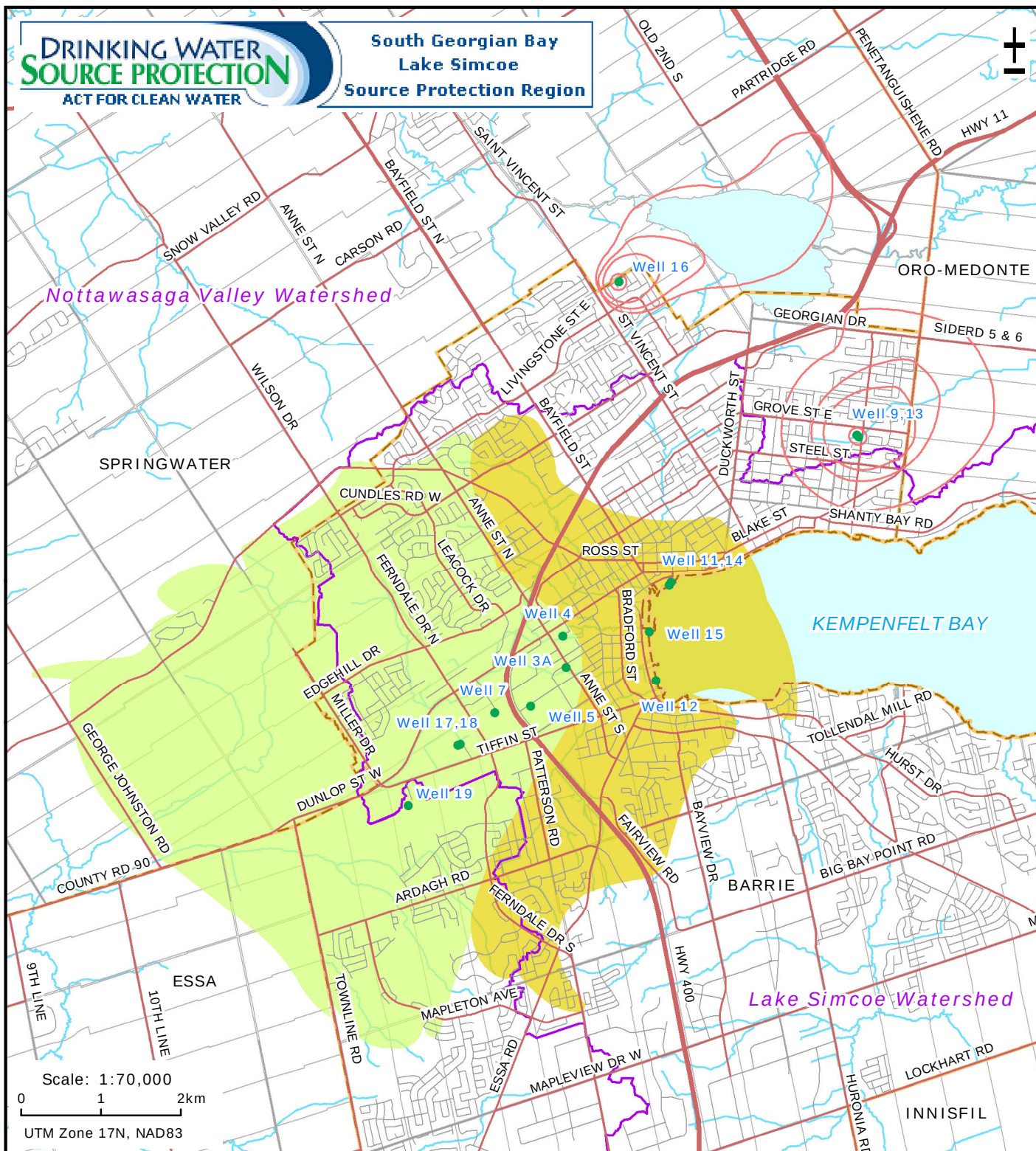
Created by: LSRCA
Date: 2014-04-10



This map was produced by the Lake Simcoe Region Conservation Authority, lead agency of the South Georgian Bay Lake Simcoe Region Source Protection Region. Base data have been compiled from various sources, under data sharing agreements. While every effort has been made to accurately depict the base data, errors may exist.



Figure 9a-1



- ! Municipal Supply Well
- Issue Contributing Area
 - Chloride and Sodium
 - Chloride
- SWP Watershed Area
- Municipality Boundary
- Adjacent Well Field WHPA

**City of Barrie
Drinking Water Issues
Contributing Areas**

Created by: LSRCA
Date: 2014-04-10

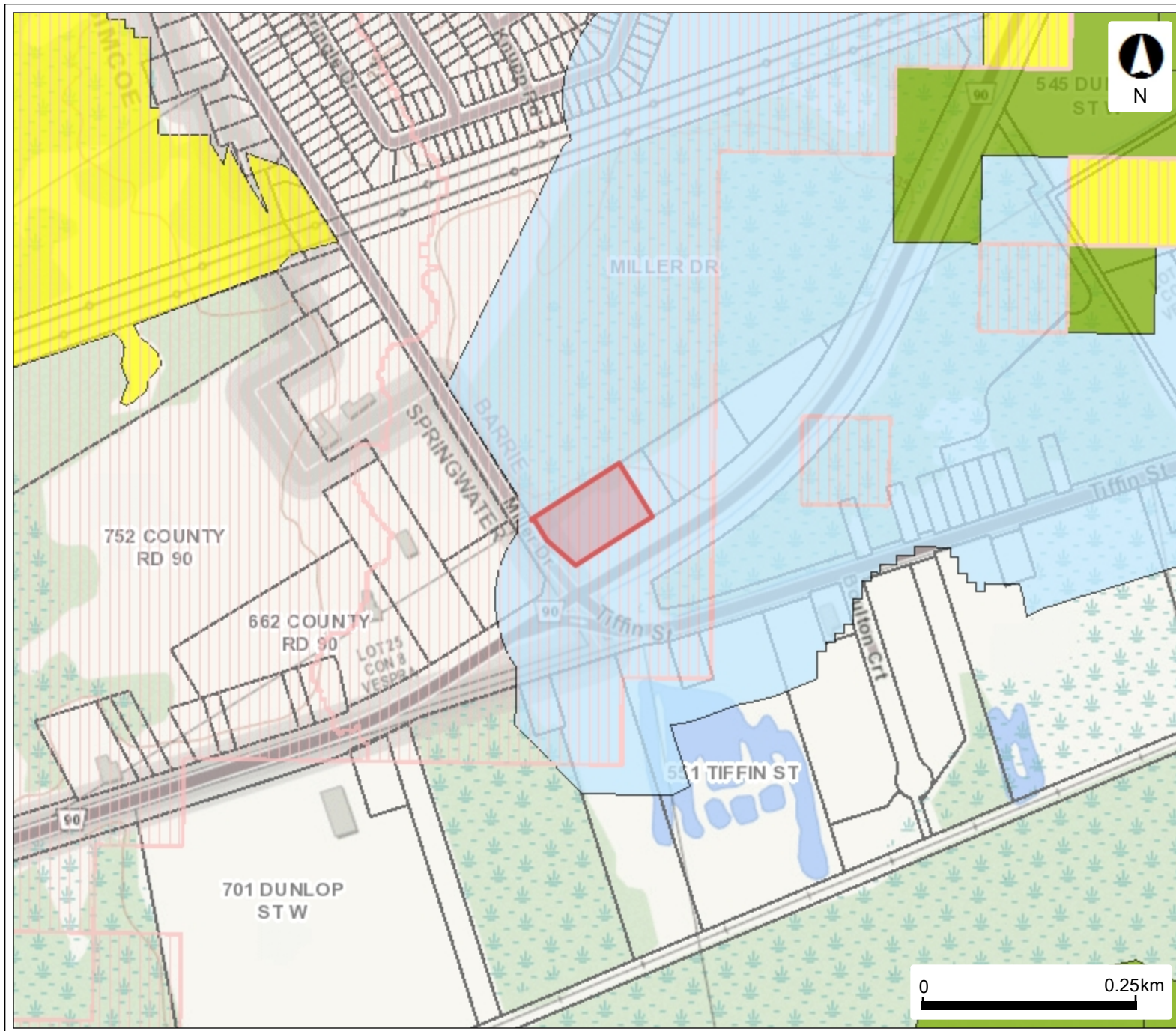


This map was produced by the Lake Simcoe Region Conservation Authority, lead agency of the South Georgian Bay Lake Simcoe Region Source Protection Region. Base data have been compiled from various sources, under data sharing agreements. While every effort has been made to accurately depict the base data, errors may exist.



Figure 9a-4

Source Area Protection Mapping



Legend

- Source Protection Areas
- Highly Vulnerable Aquifers
- Significant Groundwater Recharge Area
 - 0
 - 2
 - 4
 - 6
- Intake Protection Zone 1
- Intake Protection Zone 2
- Intake Protection Zone 3
- Assessment Parcel with Address

This map should not be relied on as a precise indicator of routes or locations, nor as a guide to navigation. The Ontario Ministry of Environment, Conservation and Parks (MECP) shall not be liable in any way for the use or any information on this map. of, or reliance upon, this map.

The map displays the Barrie, Ontario region. A red outline delineates the study area, which is situated in the northern part of the city. Major roads shown include Highway 400, Highway 27, and Highway 11. Water bodies such as Bear Creek and the Simcoe River are also visible. A scale bar at the bottom right indicates a distance of 1.97 km. A north arrow is located in the top right corner.

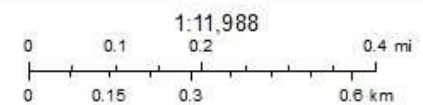
- Permits To Take Water: Active
- Intake Protection Zone Q
- Wellhead Protection Area Q1
- Wellhead Protection Area Q2

This map should not be relied on as a precise indicator of routes or locations, nor as a guide to navigation. The Ontario Ministry of Environment, Conservation and Parks (MECP) shall not be liable in any way for the use or any information on this map. of, or reliance upon, this map.

Toporama



March 3, 2022



Natural Resources
Canada

Ressources naturelles
Canada

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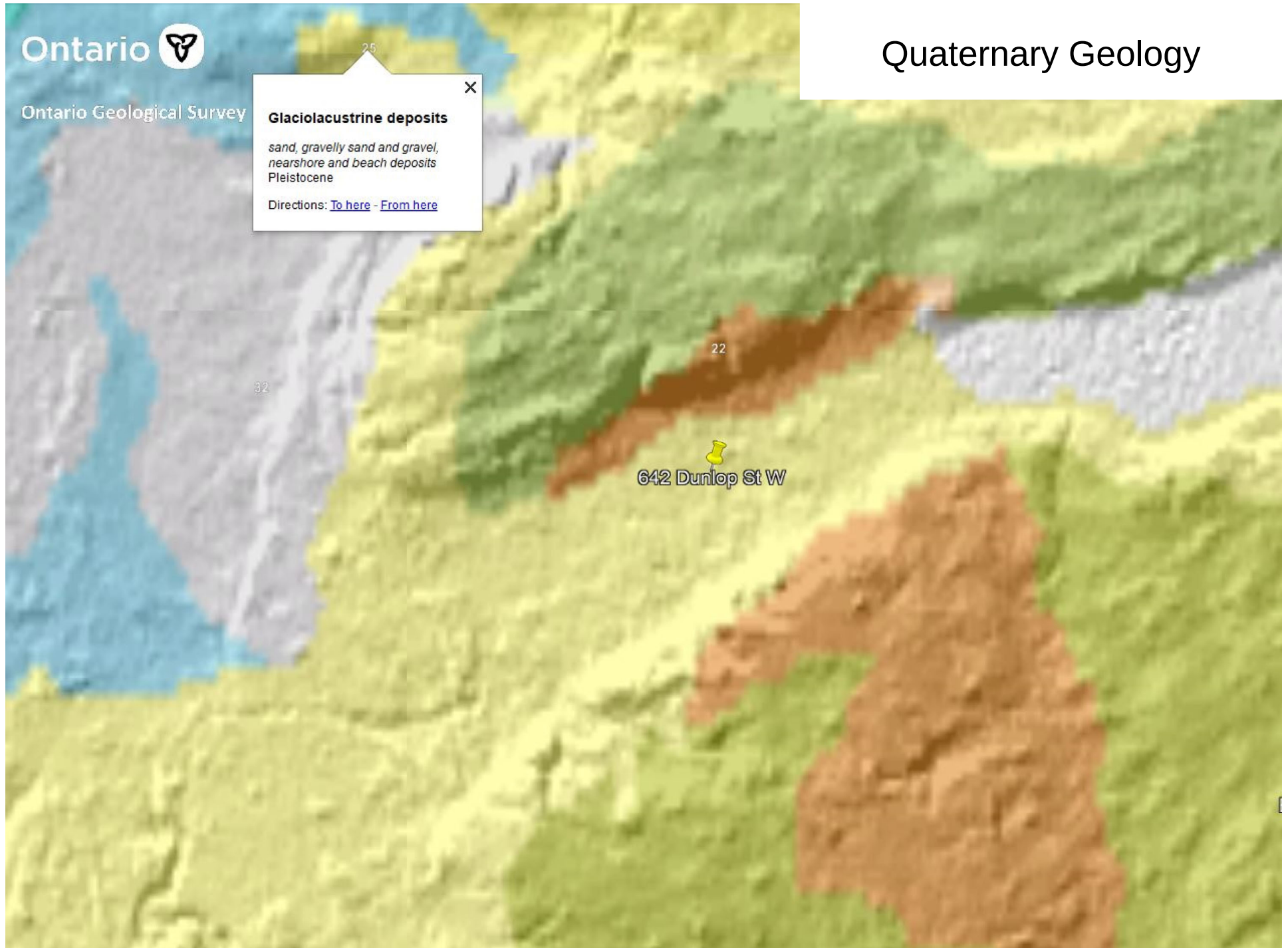
Canada

Glaciolacustrine deposits

*sand, gravelly sand and gravel,
nearshore and beach deposits
Pleistocene*

Directions: [To here](#) - [From here](#)

Quaternary Geology

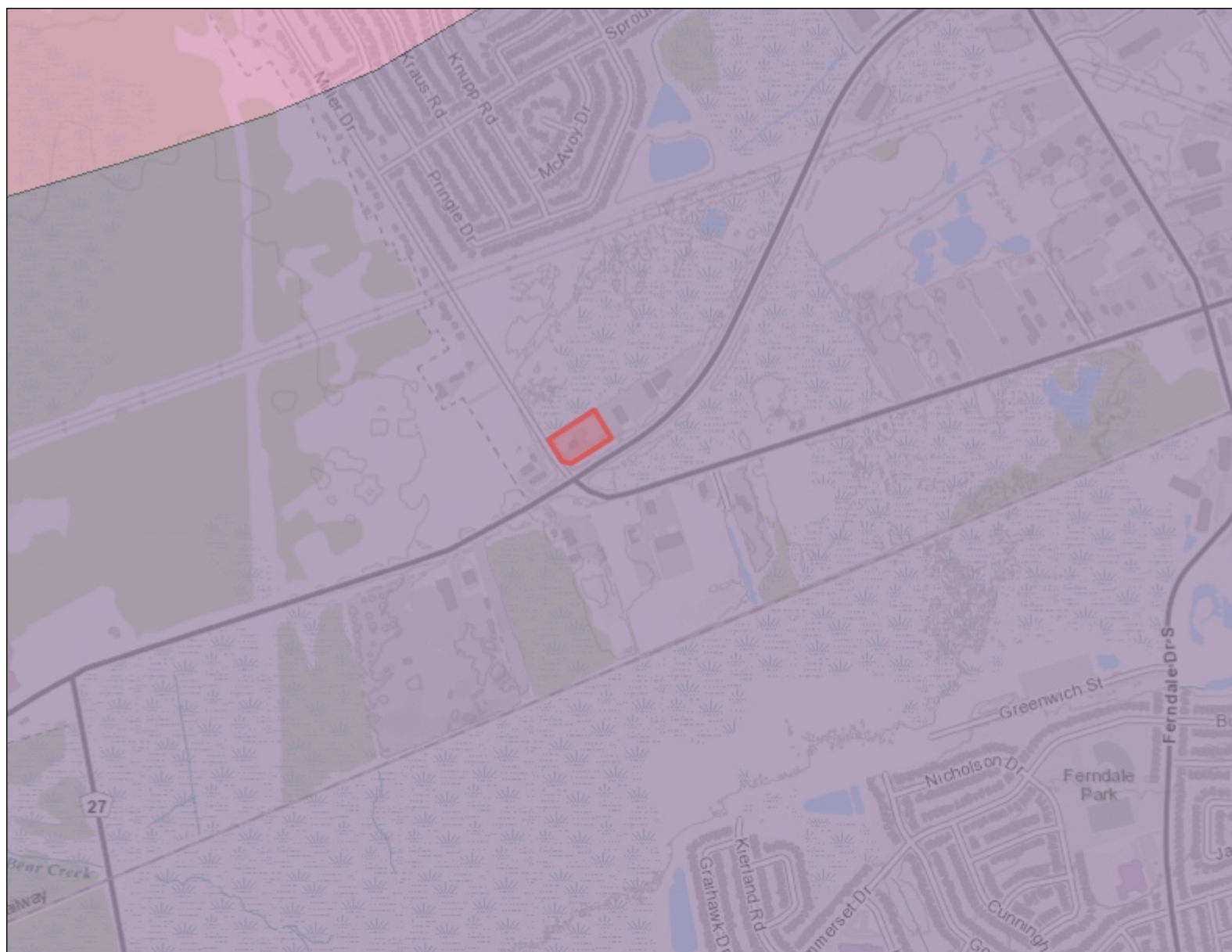


Physiographic Landforms

Legend

Physiographic Regions (OGS)

- Algonquin Highlands
- Beaver Valley
- Carden Plain
- Dummer Moraines
- Dundalk Till Plain
- Flamborough Plain
- Georgian Bay Fringe
- Guelph Drumlin Field
- Hillsburgh Sandhills
- Horseshoe Moraines
- Iroquois Plain
- Napanee Plain
- Niagara Escarpment
- Number 11 Strip
- Oak Ridges Moraine
- Peel Plain
- Peterborough Drumlin Field
- Prince Edward Peninsula
- Schomberg Clay Plains
- Simcoe Lowlands
- Simcoe Uplands
- South Slope
- Haldimand Clay Plain
- Norfolk Sand Plain



0.9 0 0.46 0.9 Km

1: 18,056



© Oak Ridges Moraine Groundwater Program, 2022

Map Compiled by the Oak Ridges Moraine Groundwater Program, 101 Exchange Avenue, Vaughan, Ontario, M3N 1S4

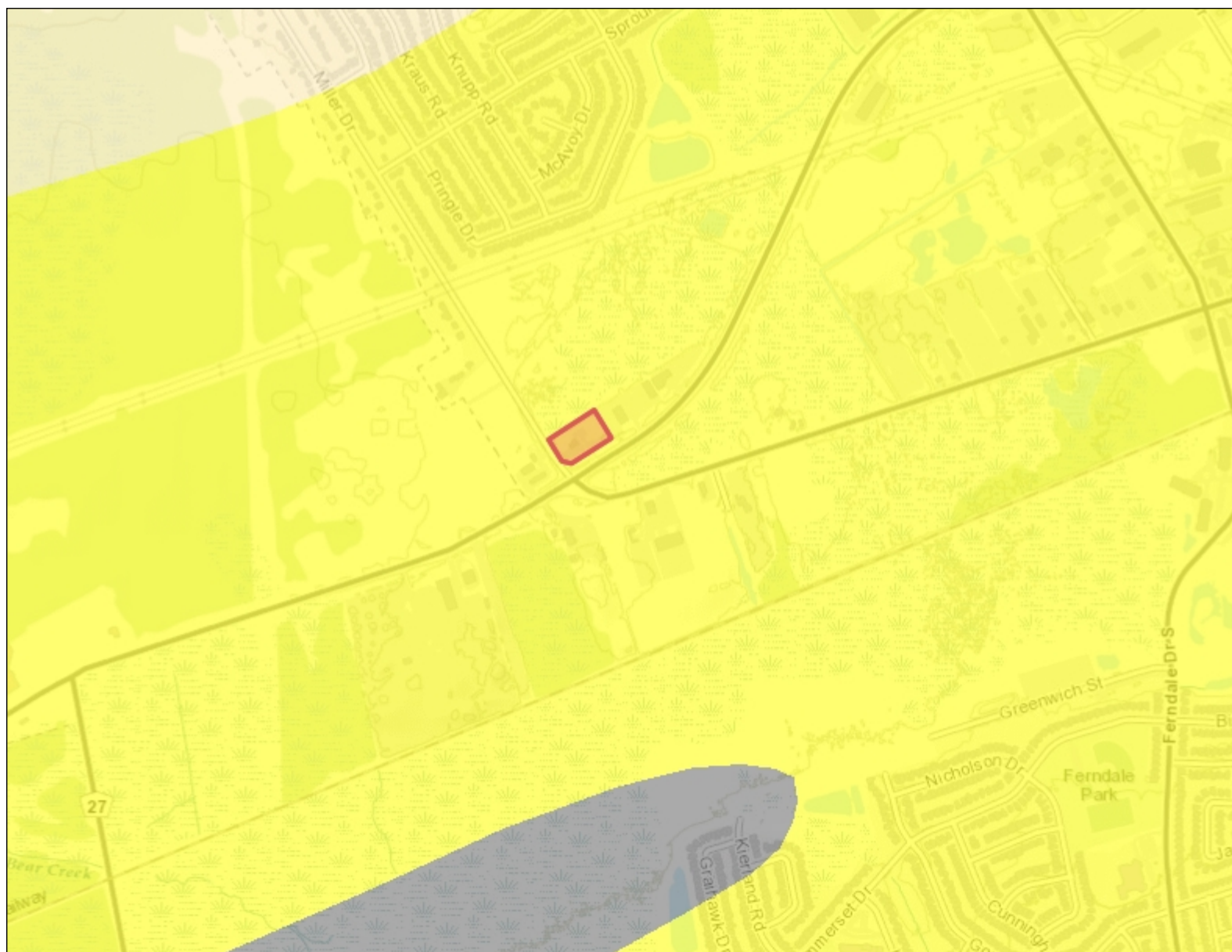
This map is for information purposes only and the Oak Ridges Moraine Groundwater Program takes no responsibility for, nor guarantees, the accuracy of all the information contained within the map.

SOURCE: ORMGP, 2022; MNRF, 2022;

PROJECTION: WGS_1984_Web_Mercator_Auxiliary_Sphere

DATE PRINTED: March 9, 2022

Physiographic Regions



Legend

Physiography (OGS)

- Bare Rock Ridges And Shallow Till
- Beaches
- Bevelled Till Plains
- Clay Plains
- Drumlins
- Escarpments
- Eskers
- Kame Moraines
- Limestone Plains
- Peat And Muck
- Sand Plains
- Shale Plains
- Shallow Till And Rock Ridges
- Spillways
- Till Moraines
- Till Plains (Drumlinized)
- Till Plains (Undrumlinized)



0.9 0 0.46 0.9 Km

1: 18,056



© Oak Ridges Moraine Groundwater Program, 2022

Map Compiled by the Oak Ridges Moraine Groundwater Program, 101 Exchange Avenue, Vaughan, Ontario, M3N 1S4

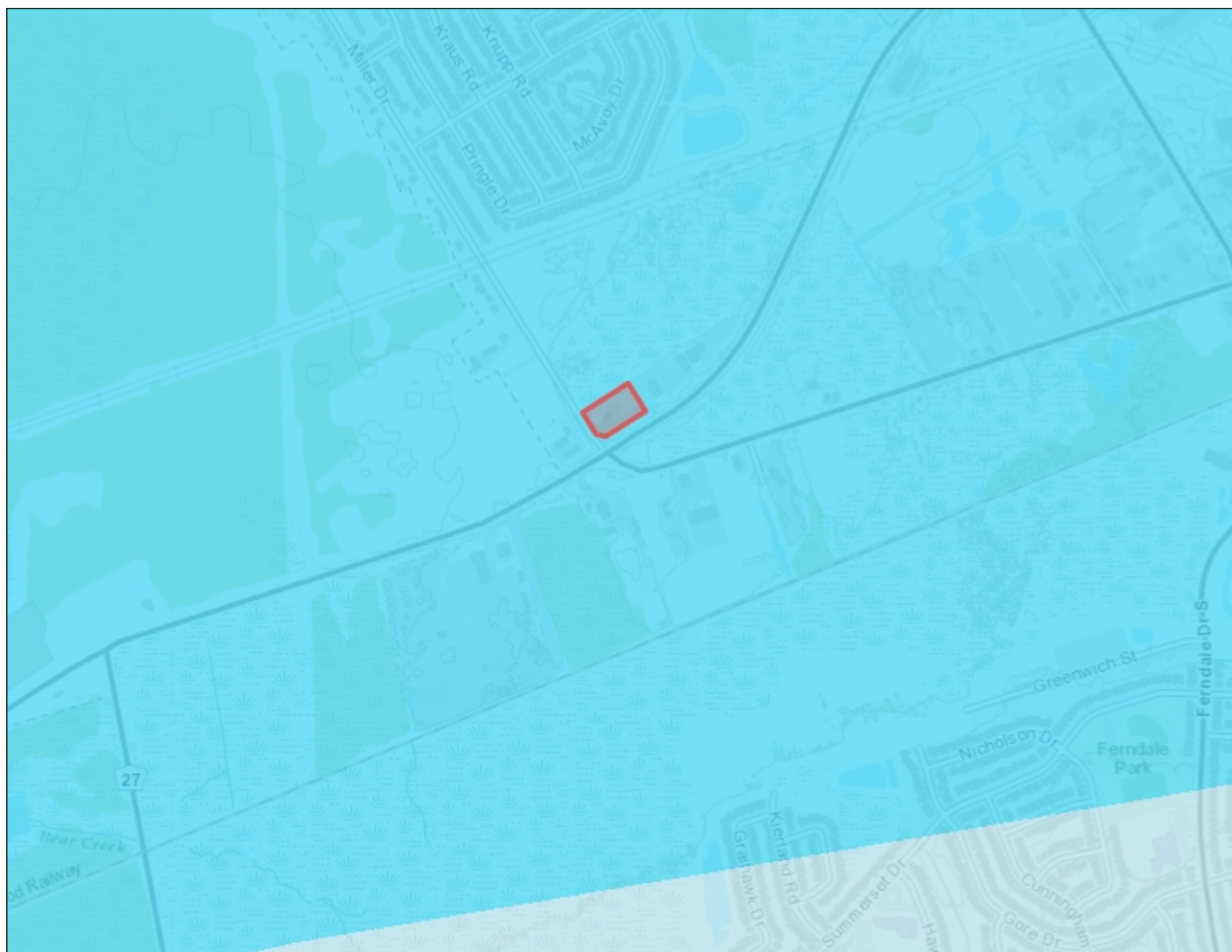
This map is for information purposes only and the Oak Ridges Moraine Groundwater Program takes no responsibility for, nor guarantees, the accuracy of all the information contained within the map.

SOURCE: ORMGP, 2022; MNRF, 2022;

PROJECTION: WGS_1984_Web_Mercator_Auxiliary_Sphere

DATE PRINTED: March 9, 2022

Bedrock Geology



Legend

- ORMGP Boundary
- ORM Plan Boundary
- Niagara Escarpment Plan Boundary
- Bedrock Geology (OGS)**
- 1: Precambrian
- 7: Shadow Lake
- 8: Gull River
- 9: Bobcaygeon
- 10: Verulam
- 11: Lindsay
- 12: Blue Mountain
- 14: Georgian Bay
- 16: Queenston
- 17: Clinton-Cataract Group
- 18: Lockport
- 19: Amabel
- 20: Guelph



0.9 0 0.46 0.9 Km

1: 18,056



© Oak Ridges Moraine Groundwater Program, 2022

Map Compiled by the Oak Ridges Moraine Groundwater Program, 101 Exchange Avenue, Vaughan, Ontario, M3N 1S4

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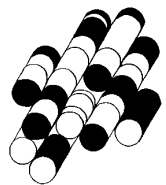
SOURCE: ORMGP, 2022; MNR, 2022;

PROJECTION: WGS_1984_Web_Mercator_Auxiliary_Sphere

DATE PRINTED: March 3, 2022

APPENDIX B

TERRAPROBE INC.



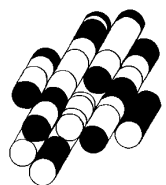
MECP Well Records Summary										
WELL ID	MECP* WWR ID	Construction Method	Well Depth (m)**	Well Usage		Water Found (mbgs)**	Static Water Level (mbgs)**	Top of Screen Depth (mbgs)**	Bottom of Screen Depth (mbgs)**	Date Completed
				Final Status	First Use					
1	5704843	Cable Tool	20.4	Water Supply	Domestic	18.3	1.8	18.9	20.4	May 6, 1959
2	5704844	Cable Tool	75.3	Water Supply	Domestic	67.7	6.1			October 21, 1964
3	5704866	Cable Tool	29.0	Water Supply	Domestic	29.0	2.7	27.5	29.0	September 18, 1958
4	5704867	Boring	4.3	Water Supply	Domestic	1.8	1.8			June 2, 1967
5	5709133	Cable Tool	34.1	Water Supply	Commercial		2.1	24.7	27.1	April 21, 1972
6	5711998	Boring	7.0	Water Supply	Domestic	4.0	4.0			July 5, 1974
7	5714854	Rotary (Convent.)	110.0	Observation Wells	Not Used		10.1	100.7	103.7	November 4, 1977
8	5714855	Rotary (Convent.)	108.8	Observation Wells	Not Used			95.8	102.5	November 11, 1977
9	5716164	Cable Tool	22.6	Water Supply	Commercial	20.4	3.1	21.7	22.6	July 12, 1979
10	5716689	Rotary (Convent.)	79.9	Water Supply	Industrial	75.6	9.8			June 8, 1979
11	5718060	Cable Tool	25.3	Water Supply	Domestic	22.3	22.3	25.3	26.2	April 23, 1982
12	5718624	Cable Tool	15.2	Water Supply	Commercial	5.8	2.4	12.5	13.4	August 3, 1983
13	5718988	Cable Tool	19.8	Water Supply	Domestic	16.5	4.0	18.3	19.8	January 26, 1984
14	5719487	Cable Tool	12.2	Water Supply	Domestic	12.2	1.8	10.4	11.3	November 7, 1984
15	5728600	Rotary (Convent.)	73.2	Water Supply	Domestic	72.3	10.7	72.3	73.2	October 18, 1991
16	5734829	Rotary (Convent.)	96.3	Water Supply	Test Hole		12.2	83.0	92.1	August 5, 1999
17	5734833	Rotary (Convent.)	99.1	Observation Wells	Municipal		13.7	71.4	77.5	May 11, 1999
18	5734835	Rotary (Convent.)	14.9	Observation Wells	Public		2.4	12.8	14.3	August 16, 1999
19	5739405	Cable Tool	5.5	Water Supply	Domestic	27.8	5.6	29.3	30.5	November 12, 2004
20	5740528	Rotary (Convent.)	91.1	Observation Wells	Not Used		11.7	76.2	85.3	February 10, 2006
21	5741280		26.5	Abandoned-Other	Domestic					September 5, 2006
22	7114203	Rotary (Reverse)	94.4	Water Supply	Municipal	12.0	13.7			December 7, 2007
23	7114205	Rotary (Convent.)	100.9	Test Hole	Test Hole		11.2			February 8, 2008
24	7124628		4.5	Observation Wells	Monitoring			1.5	4.5	April 24, 2009
25	7154757									October 14, 2010
26	7225837	Boring	4.6		Monitoring and Test Hole			2.7	4.3	August 1, 2014
27	7260721	Boring	4.6	Observation Wells	Test Hole	2.1		1.5	4.6	June 25, 2015
28	7286079	Rotary (Convent.)	4.6	Observation Wells	Monitoring	3.7	3.7	3.1	4.6	March 14, 2017

MECP*: Ministry of the Environment, Conservation and Parks

mbgs**: meters below ground surface

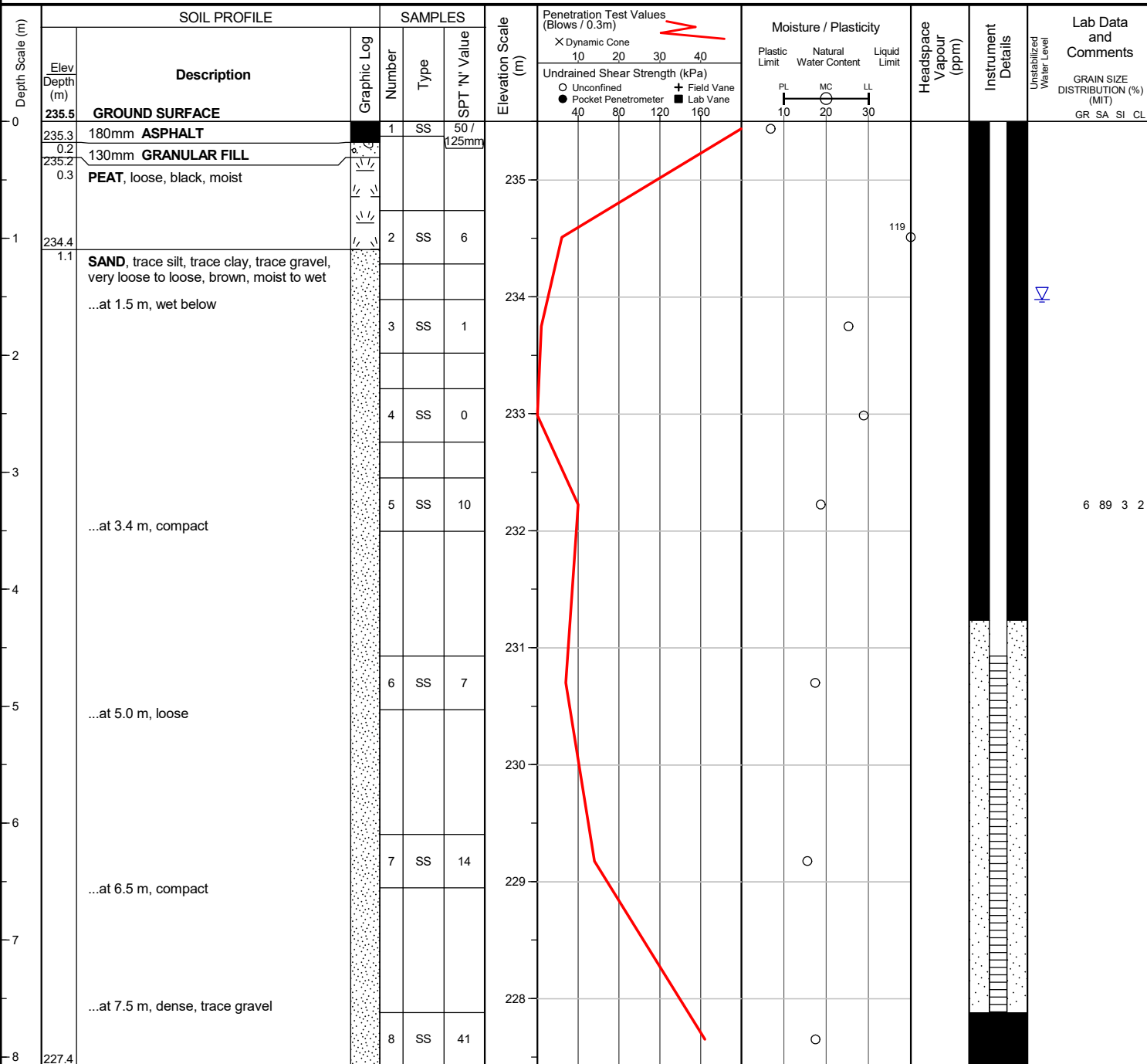
APPENDIX C

TERRAPROBE INC.



Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : SO
Date started : January 19, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601037, N: 4913351 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers



END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS
 Date Feb 1, 2022 Water Depth (m) Elevation (m) n/a



Originated by : CB

Compiled by : GO

Checked by : OB

Drilling Method : Hollow stem augers

file: 3-21-0182-01 bh logs.gpj

(continued next page)

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 20, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 2 of 2	Location : Barrie, Ontario	Checked by : OB

Position : E: 601094, N: 4913399 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone Undrained Shear Strength (kPa) O Unconfined ● Pocket Penetrometer	Moisture / Plasticity Plastic Limit Natural Water Content Liquid Limit PL MC LL	Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value						
		(continued)										
11		SAND, trace silt, trace clay, trace gravel, very loose, brown, moist to wet (continued)					224					
		...at 11.0 m, compact		10	SS	18						
12							223					
				11	SS	31	222					
221.9 12.6												

END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS		
Date	Water Depth (m)	Elevation (m)
Feb 1, 2022	1.7	232.8

Project No. : 3-21-0182-01

Client : 25707517 Ontario Inc.

Originated by : CB

Date started : January 18, 2022

Project : 642 Dunlop Street West

Compiled by : GO

Sheet No. : 1 of 1

Location : Barrie, Ontario

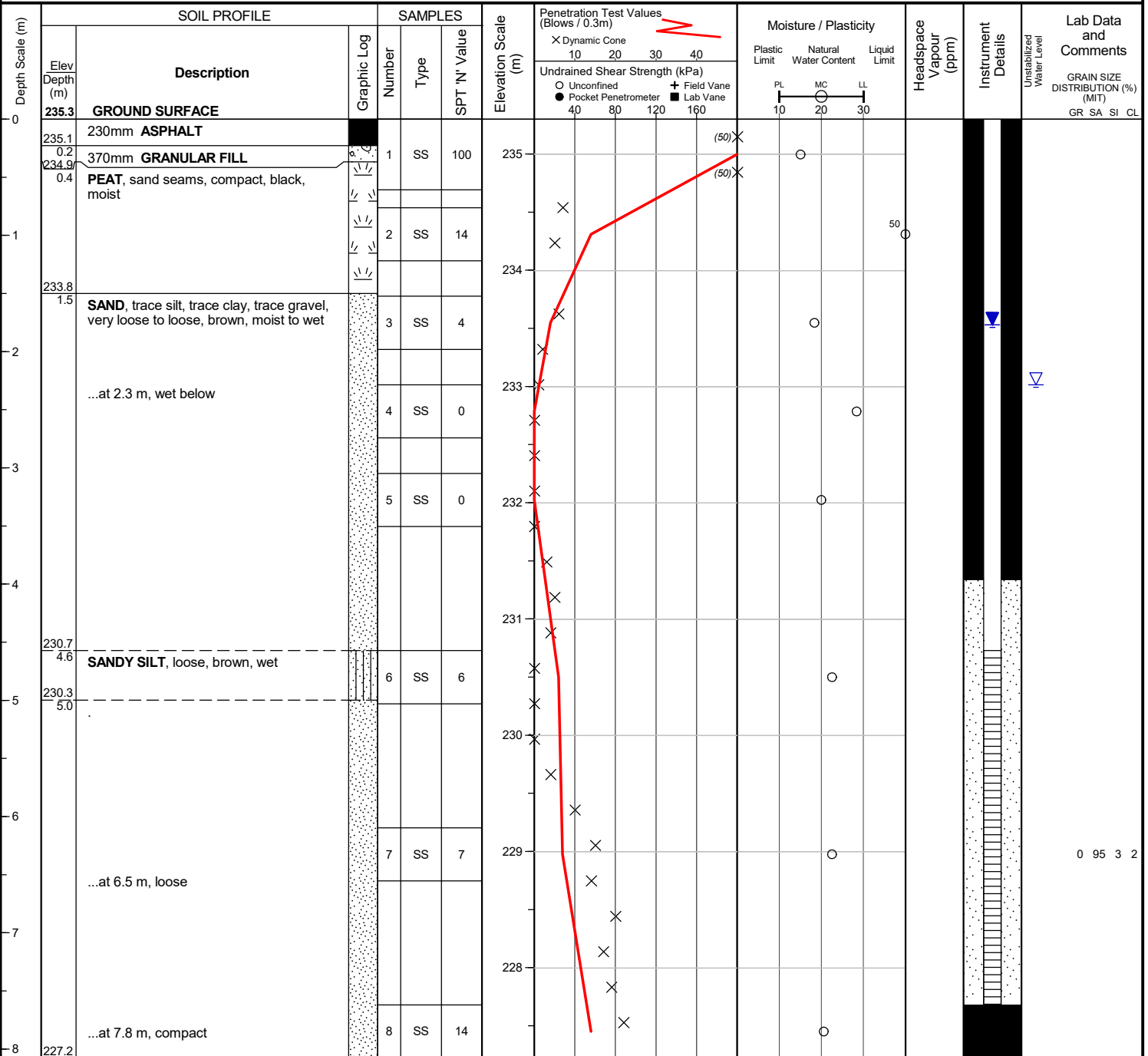
Checked by : OB

Position : E: 601077, N: 4913363 (UTM 17T)

Elevation Datum : Geodetic

Rig type : Track-mounted

Drilling Method : Hollow stem augers



END OF BOREHOLE

Unstabilized water level measured at 2.3 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS
 Date Feb 1, 2022
 Water Depth (m) 1.8
 Elevation (m) 233.5



Originated by : SO

Compiled by : GO

Checked by : OB

Drilling Method : Hollow stem augers

file: 3-21-0182-01 bh logs.gpj

(continued next page)

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : SO
Date started : January 19, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 2 of 2	Location : Barrie, Ontario	Checked by : OB

Position : E: 601049, N: 4913340 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone Undrained Shear Strength (kPa) ○ Unconfined ● Pocket Penetrometer	Moisture / Plasticity Plastic Limit Natural Water Content Liquid Limit PL MC LL	Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value						
		(continued)										
11		SAND, trace silt, trace clay, trace gravel, loose to compact, brown, moist to wet (continued)					225	(60)X				
		...at 10.8 m, dense		10	SS	46		(68)X				
12							224	(53)X				
								(56)X				
								(52)X				
								(50)X				
				11	SS	11	223	X				disturbed sample
222.8 12.6												

END OF BOREHOLE

Unstabilized water level measured at 2.3 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS		
Date	Water Depth (m)	Elevation (m)
Feb 1, 2022	1.9	233.5

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 19, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601106, N: 4913377 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

SOIL PROFILE			SAMPLES			Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments	
Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value	Elevation Scale (m)	X Dynamic Cone		Plastic Limit Natural Water Content Liquid Limit				
	10							20	30	40	PL	MC	LL	
0	235.2	GROUND SURFACE						Undrained Shear Strength (kPa)						
	235.0	130mm ASPHALT		1	SS	26	235					85		
	0.2	170mm GRANULAR FILL												
		PEAT, with silt, sand seams, loose, black, moist												
	234.4													
	0.8	SAND, trace silt, trace clay, trace gravel, very loose to compact, brown, moist to wet		2	SS	7	234							
		...at 1.5 m, wet below												
				3	SS	0								
				4	SS	0								
1														
2														
3														
4														
5														
6														
7														
8														

Unstabilized water level measured at 1.5 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

<u>Date</u>	<u>Water Depth (m)</u>	<u>Elevation (m)</u>
Feb 1, 2022	1.5	233.7

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 18, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601031, N: 4913349 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)	Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		X Dynamic Cone 10 20 30 40	Plastic Limit	Natural Water Content	Liquid Limit			
								Undrained Shear Strength (kPa) ○ Unconfined + Field Vane ● Pocket Penetrometer ■ Lab Vane 40 80 120 160	PL	MC	LL			
0	235.7	GROUND SURFACE												
	235.5	230mm ASPHALT		1	SS	17						86		
	235.5	250mm GRANULAR FILL												
	235.5	PEAT, compact, black, moist		2	SS	10						132		
1	234.5													
	233.9	SAND, trace silt, trace clay, trace gravel, trace organics, very loose, brown, moist to wet		3	SS	2								
														</

END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface; borehole was open upon completion of drilling.

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601084, N: 4913397 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		X Dynamic Cone	Undrained Shear Strength (kPa)	Plastic Limit	Natural Water Content	Liquid Limit			
0	234.3	GROUND SURFACE													
	234.1	150mm TOPSOIL		1	SS	22	234								
	0.2	FILL , peat/ sandy silt, some gravel, compact, black, moist													
	233.7	PEAT , sand seams,, very loose, black, wet		2	SS	2									
	0.6														
-1	232.9	SAND , trace silt, trace clay, trace gravel, trace organics, loose, brown, moist		3	SS	7	233								
	1.4														
	232.5														
	1.8														

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 3-21-0182-01 Client : 25707517 Ontario Inc. Originated by : CB
 Date started : January 18, 2022 Project : 642 Dunlop Street West Compiled by : GO
 Sheet No. : 1 of 1 Location : Barrie, Ontario Checked by : OB

Position : E: 601115, N: 4913390 (UTM 17T) Elevation Datum : Geodetic
 Rig type : Track-mounted Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m)		Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value		X Dynamic Cone	Undrained Shear Strength (kPa)	Plastic Limit	Natural Water Content	Liquid Limit			
0	235.1	GROUND SURFACE					235								
	234.9	230mm ASPHALT		1	SS	4									
	0.2	FILL, organics/peat, some gravel, sand, loose, black/brown, moist													
	234.5	PEAT, sand seams, loose, black, moist													
	0.6														
	234.3	SAND, trace silt, trace clay, trace gravel, some organics, loose, brown, moist		2	SS	5									
	0.8														
1				3	SS	4									
	233.3														
	1.8														

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 3-21-0182-01	Client : 25707517 Ontario Inc.	Originated by : CB
Date started : January 18, 2022	Project : 642 Dunlop Street West	Compiled by : GO
Sheet No. : 1 of 1	Location : Barrie, Ontario	Checked by : OB

Position : E: 601082, N: 4913416 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Hollow stem augers

Depth Scale (m)	SOIL PROFILE			SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone 10 20 30 40 Undrained Shear Strength (kPa) ○ Unconfined + Field Vane ● Pocket Penetrometer ■ Lab Vane 40 80 120 160	Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments	
	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value			Plastic Limit	Natural Water Content	Liquid Limit				
0	234.4	GROUND SURFACE													
1		Borehole augered to 5m depth without sampling for installation of monitoring well					234								
2								233							
3								232							
4								231							
5	229.4							230							

END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface; borehole was open upon completion of drilling.

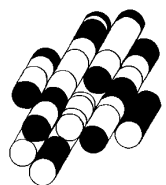
50 mm dia. monitoring well installed.

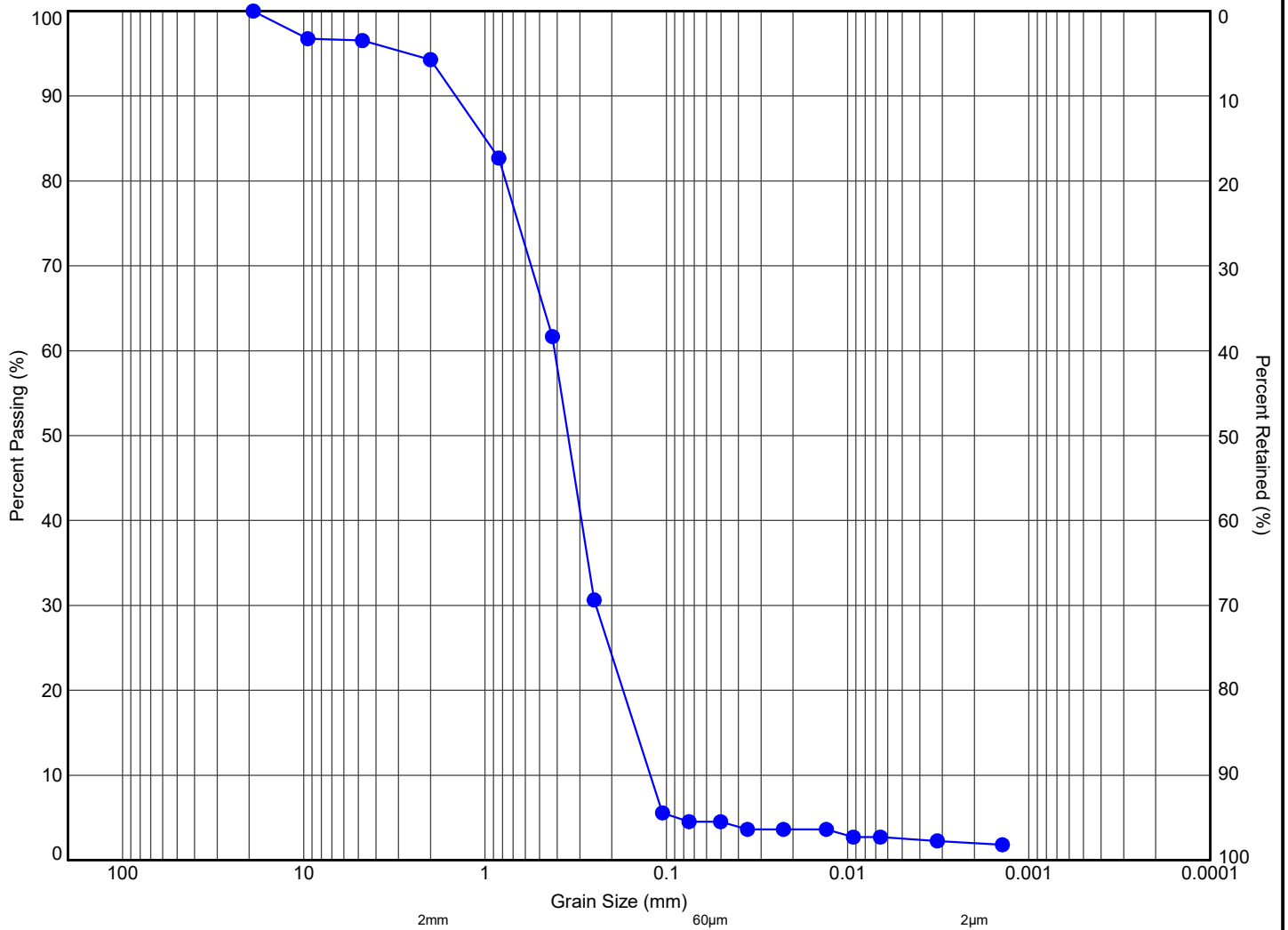
WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Feb 1, 2022	1.7	232.7

APPENDIX D

TERRAPROBE INC.





MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● 1	SS5	3.3	232.2	6	89	3	2		



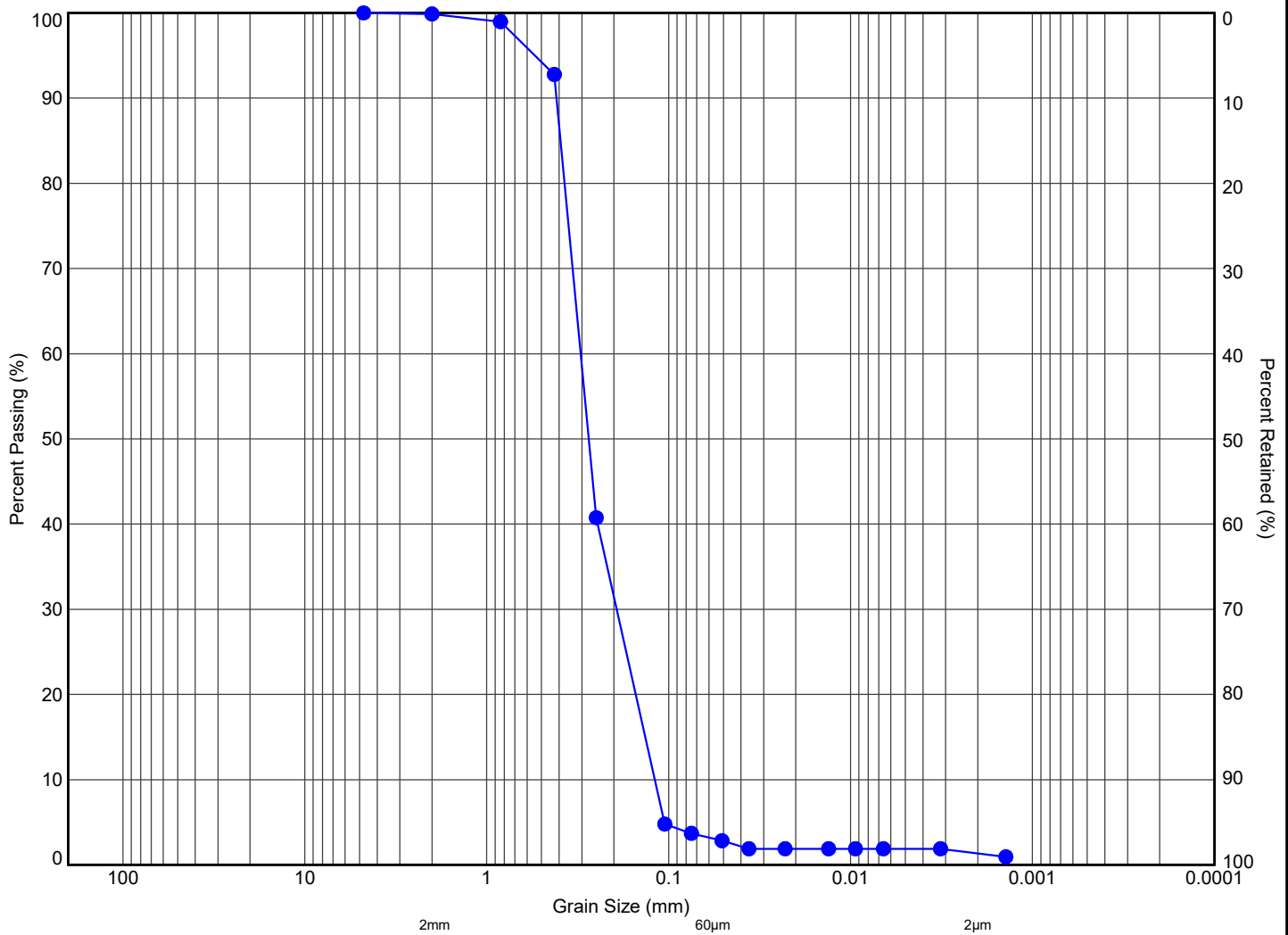
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

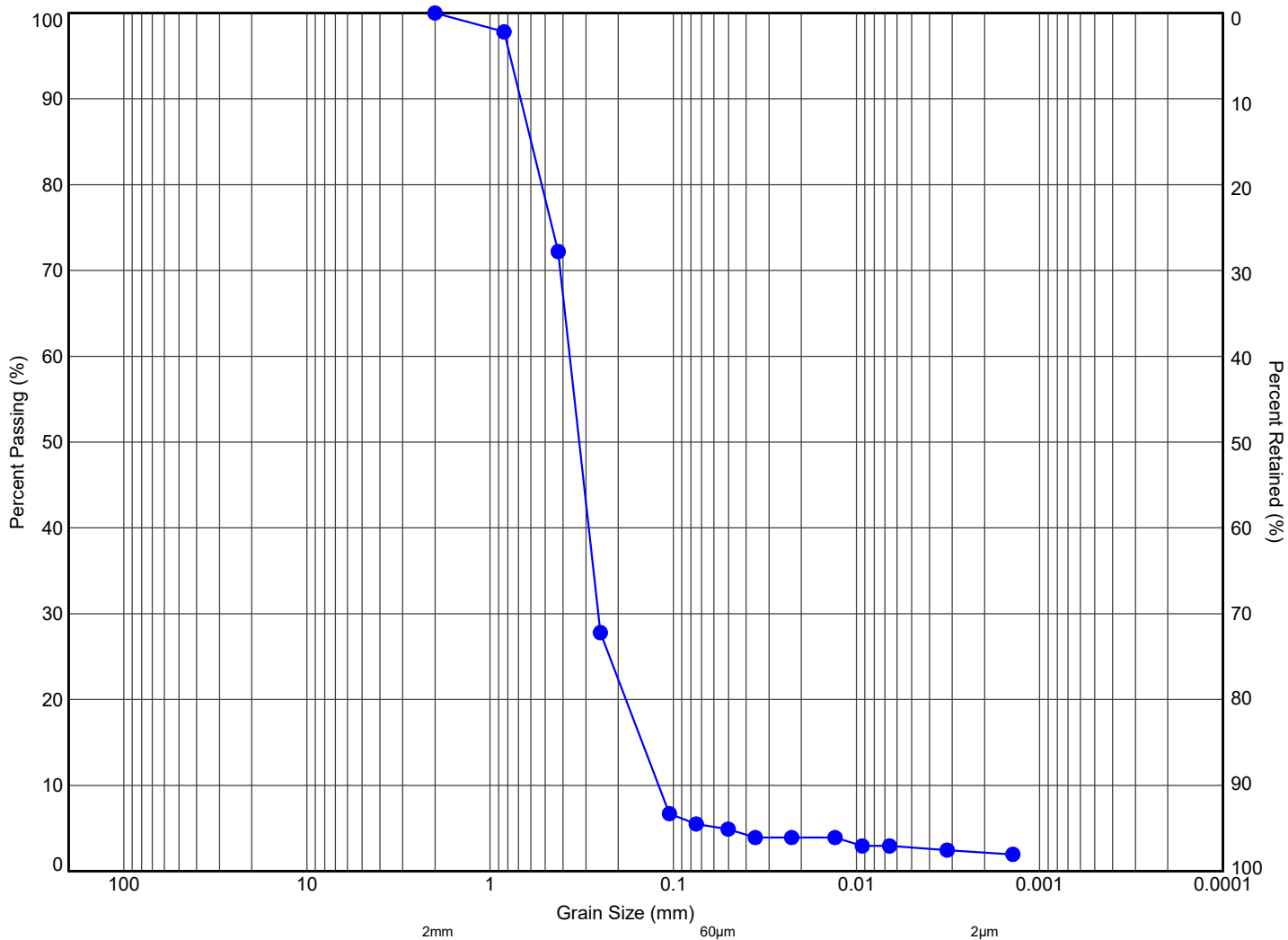
Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01





MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM

Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)
3	SS7	6.3	229.0	0	95	3	2	



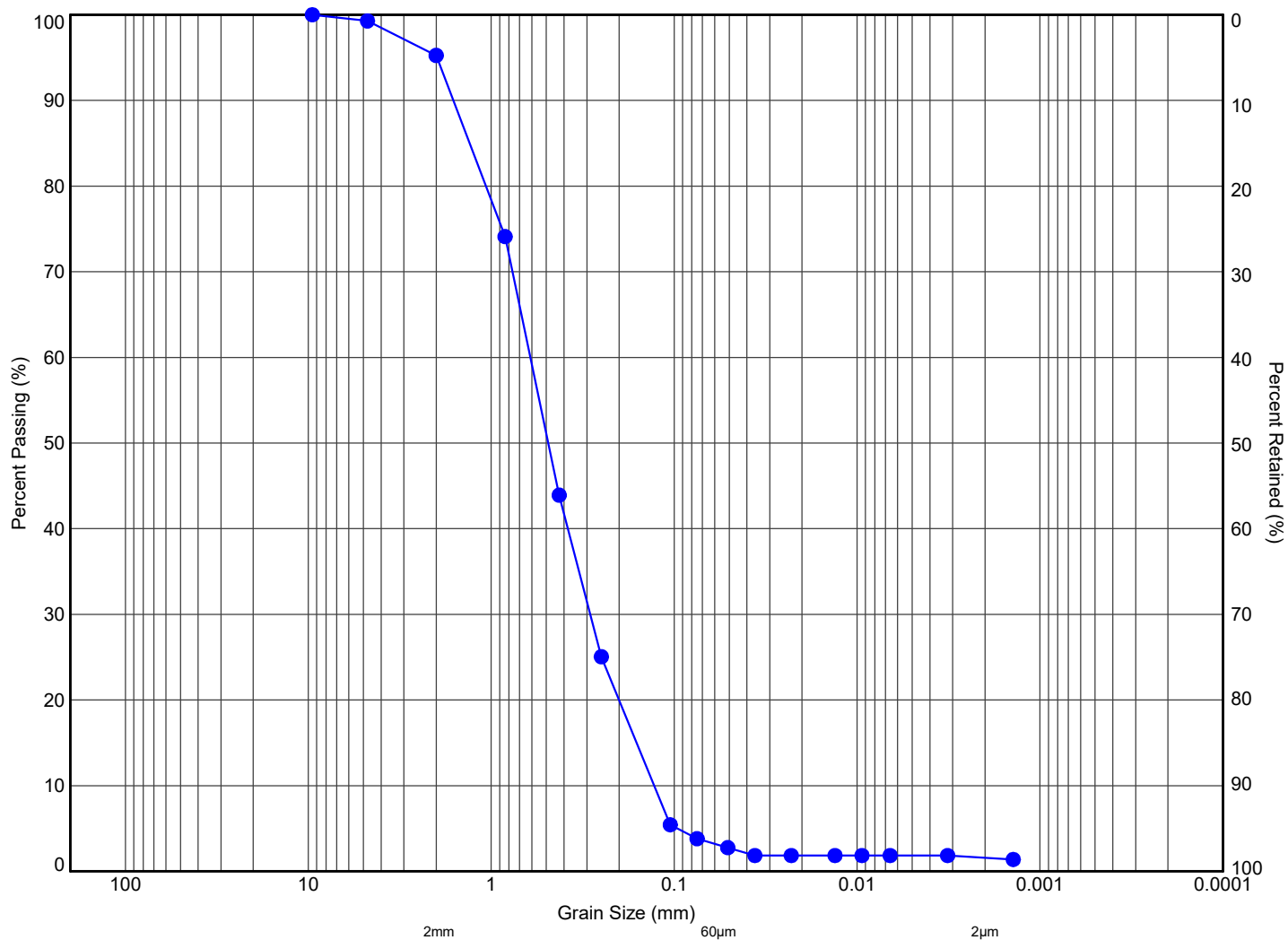
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● 4	SS10	10.9	224.5	5	91	2	2		



11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

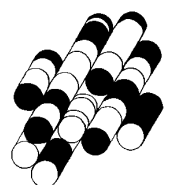
**GRAIN SIZE DISTRIBUTION
SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY**

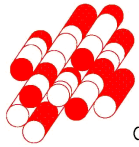
File No.:

3-21-0182-01

APPENDIX E

TERRAPROBE INC.





Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

Slug Test Analysis Report

Project: 642 Dunlop Street West

Number: 3-21-0182-46

Client: 25707517 Ontario Inc.

Location: Barrie, ON

Slug Test: BH 2

Test Well: BH 2

Test Conducted by: CB

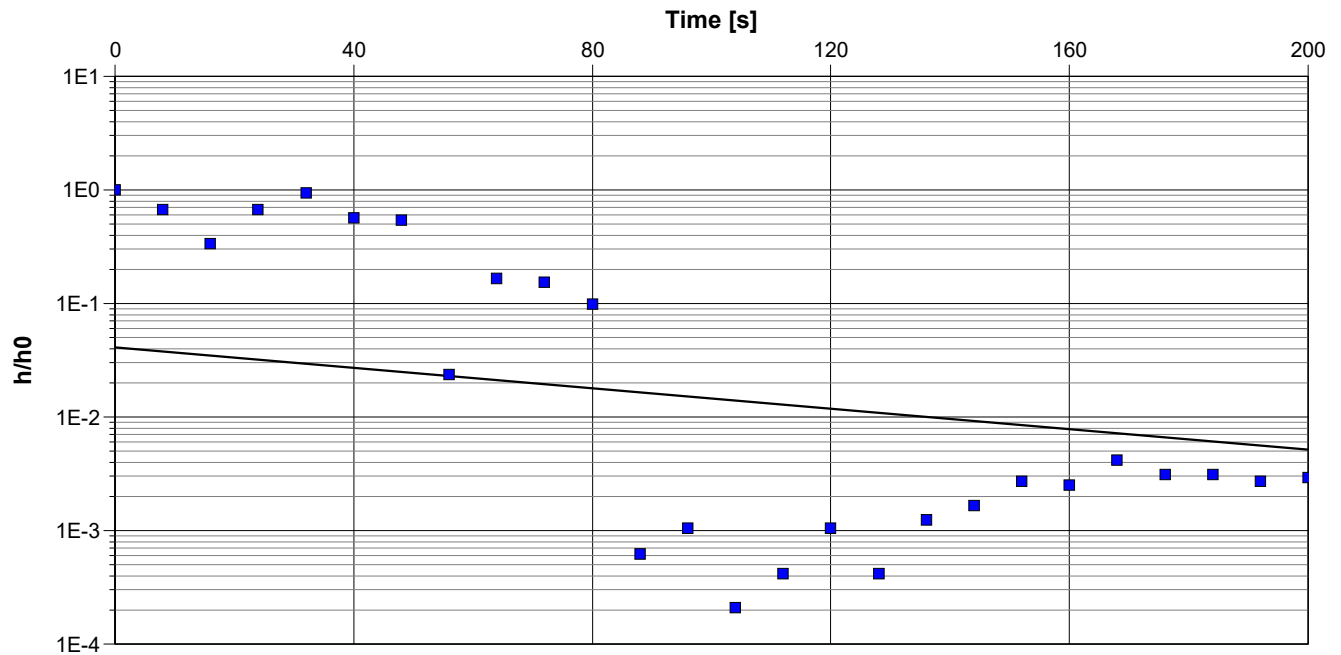
Test Date: 2022-02-01

Analysis Performed by: UA

BH 2

Analysis Date: 2022-03-21

Aquifer Thickness: 12.00 m



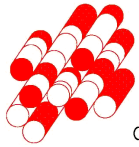
Calculation using Bouwer & Rice

Observation Well

Hydraulic Conductivity
[m/s]

BH 2

4.08×10^{-6}



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

Slug Test Analysis Report

Project: 642 Dunlop Street West

Number: 3-21-0182-46

Client: 25707517 Ontario Inc.

Location: Barrie, ON

Slug Test: BH 3

Test Well: BH 3

Test Conducted by: CB

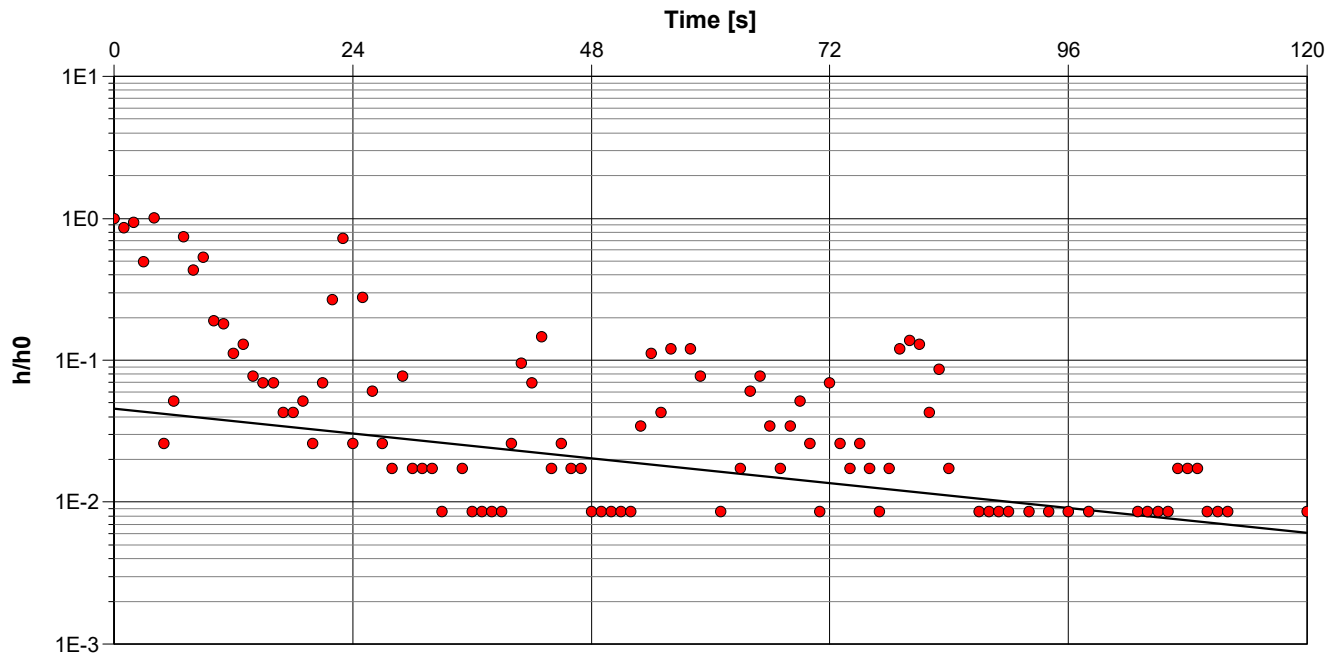
Test Date: 2022-02-01

Analysis Performed by: UA

BH 3

Analysis Date: 2022-03-21

Aquifer Thickness: 12.00 m



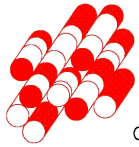
Calculation using Bouwer & Rice

Observation Well

Hydraulic Conductivity
[m/s]

BH 3

5.75×10^{-6}



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

Slug Test Analysis Report

Project: 642 Dunlop Street West

Number: 3-21-0182-46

Client: 25707517 Ontario Inc.

Location: Barrie, ON

Slug Test: BH 4

Test Well: BH 4

Test Conducted by: CB

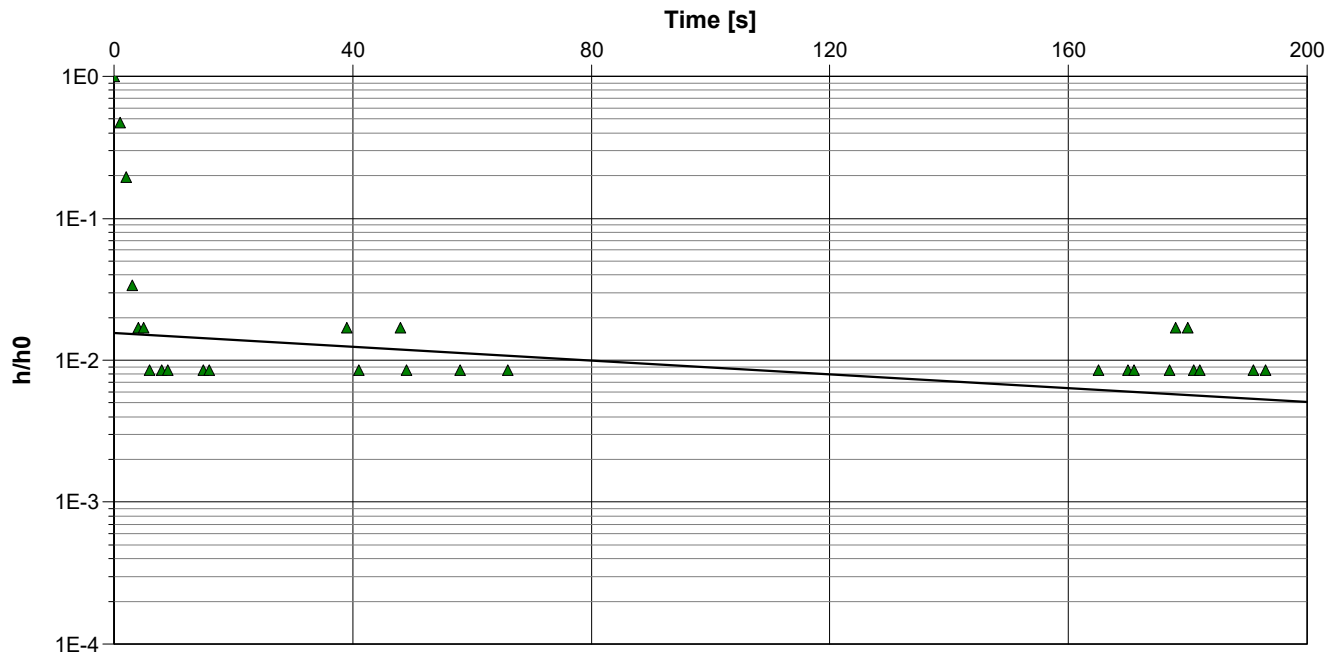
Test Date: 2022-01-01

Analysis Performed by: UA

BH 4

Analysis Date: 2022-03-21

Aquifer Thickness: 12.00 m



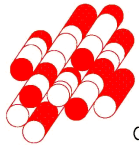
Calculation using Bouwer & Rice

Observation Well

Hydraulic Conductivity
[m/s]

BH 4

2.19×10^{-6}



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

Slug Test Analysis Report

Project: 642 Dunlop Street West

Number: 3-21-0182-46

Client: 25707517 Ontario Inc.

Location: Barrie, ON

Slug Test: BH 5

Test Well: BH 5

Test Conducted by:

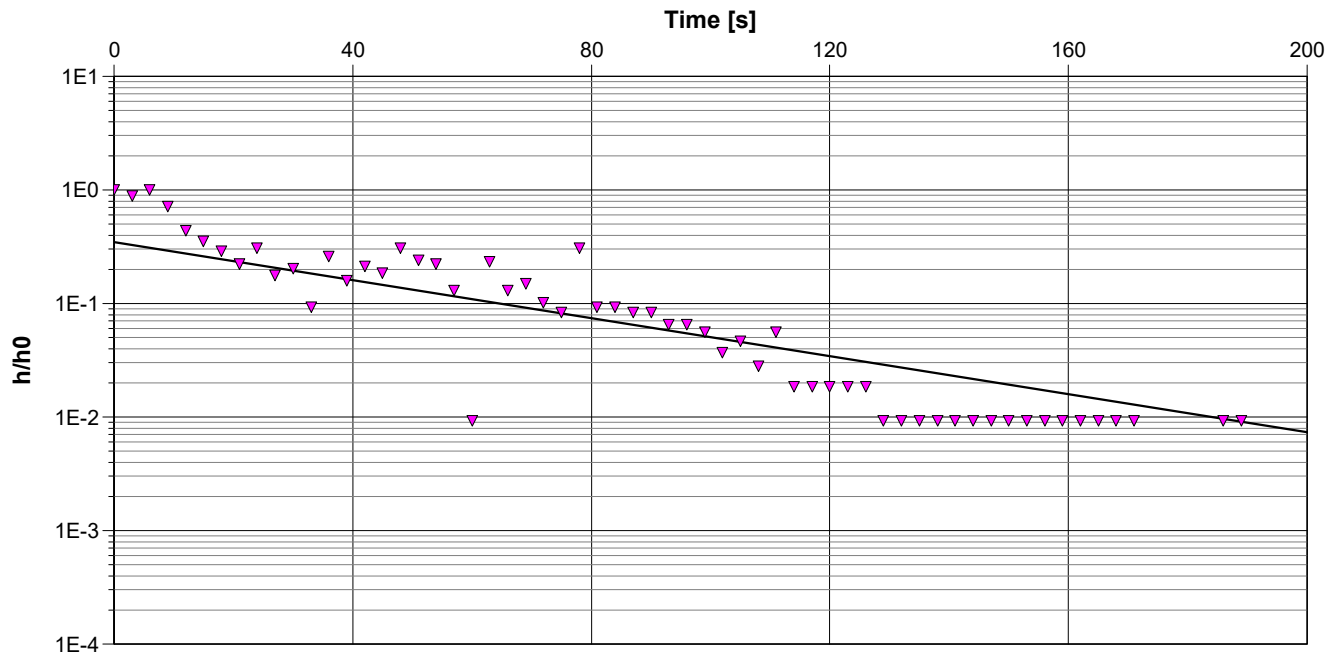
Test Date: 2022-03-21

Analysis Performed by: UA

BH 5

Analysis Date: 2022-03-21

Aquifer Thickness: 12.00 m



Calculation using Bouwer & Rice

Observation Well

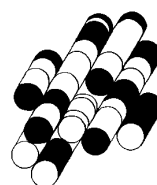
Hydraulic Conductivity
[m/s]

BH 5

6.05×10^{-6}

APPENDIX F

TERRAPROBE INC.



CLIENT NAME: TERRAPROBE INC.
220 BAYVIEW DRIVE, UNIT 25
BARRIE, ON L4N4Y8
(705) 739-8355

ATTENTION TO: Justin Rumneey

PROJECT: 3-21-0182-46

AGAT WORK ORDER: 22T859620

WATER ANALYSIS REVIEWED BY: Yris Verastegui, Report Reviewer

DATE REPORTED: Feb 08, 2022

PAGES (INCLUDING COVER): 10

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*Notes

Disclaimer:

- All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.
- All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.
- AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.
- This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
- The test results reported herewith relate only to the samples as received by the laboratory.
- Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.
- All reportable information as specified by ISO/IEC 17025:2017 is available from AGAT Laboratories upon request.



Certificate of Analysis

AGAT WORK ORDER: 22T859620

PROJECT: 3-21-0182-46

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: TERRAPROBE INC.

SAMPLING SITE:Barrie

ATTENTION TO: Justin Rumneey

SAMPLED BY:Justin Rumney

Water Quality Assessment - PWQO (mg/L)

DATE RECEIVED: 2022-02-02

DATE REPORTED: 2022-02-08

		SAMPLE DESCRIPTION:		BH2
		SAMPLE TYPE:		Water
		DATE SAMPLED:		2022-01-02 12:40
Parameter	Unit	G / S	RDL	3474015
Electrical Conductivity	µS/cm		2	979
pH	pH Units	6.5-8.5	NA	7.87
Saturation pH (Calculated)				6.86
Langelier Index (Calculated)				1.01
Hardness (as CaCO3) (Calculated)	mg/L		0.5	349
Total Dissolved Solids	mg/L		10	564
Alkalinity (as CaCO3)	mg/L		5	307
Bicarbonate (as CaCO3)	mg/L		5	307
Carbonate (as CaCO3)	mg/L		5	<5
Hydroxide (as CaCO3)	mg/L		5	<5
Fluoride	mg/L		0.05	<0.05
Chloride	mg/L		0.12	108
Nitrate as N	mg/L		0.05	0.80
Nitrite as N	mg/L		0.05	<0.05
Bromide	mg/L		0.05	<0.05
Sulphate	mg/L		0.10	51.0
Ortho Phosphate as P	mg/L		0.10	<0.10
Ammonia as N	mg/L		0.02	<0.02
Ammonia-Un-ionized (Calculated)	mg/L	0.02	0.000002	<0.000002
Total Phosphorus	mg/L	*	0.02	0.08
Total Organic Carbon	mg/L		0.5	15.9
True Colour	TCU		5	24
Turbidity	NTU		0.5	13.2
Total Calcium	mg/L		0.32	120
Total Magnesium	mg/L		0.34	12.0
Total Potassium	mg/L		1.15	2.15
Total Sodium	mg/L		0.45	59.1
Aluminum-dissolved	mg/L	*	0.004	0.010
Total Antimony	mg/L	0.020	0.001	<0.001

Certified By:

José Verástegui



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 22T859620

PROJECT: 3-21-0182-46

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: TERRAPROBE INC.

SAMPLING SITE:Barrie

ATTENTION TO: Justin Rumneey

SAMPLED BY:Justin Rumney

Water Quality Assessment - PWQO (mg/L)

DATE RECEIVED: 2022-02-02

DATE REPORTED: 2022-02-08

		SAMPLE DESCRIPTION:		BH2
		SAMPLE TYPE:		Water
		DATE SAMPLED:		2022-01-02 12:40
Parameter	Unit	G / S	RDL	3474015
Total Arsenic	mg/L	0.1	0.003	<0.003
Total Barium	mg/L		0.002	0.083
Total Beryllium	mg/L	*	0.001	<0.001
Total Boron	mg/L	0.2	0.010	0.031
Total Cadmium	mg/L	0.0002	0.0001	<0.0001
Total Chromium	mg/L		0.003	<0.003
Total Cobalt	mg/L	0.0009	0.0005	0.0006
Total Copper	mg/L	0.005	0.001	0.005
Total Iron	mg/L	0.3	0.010	1.48
Total Lead	mg/L	*	0.001	0.001
Total Manganese	mg/L		0.002	0.145
Dissolved Mercury	mg/L	0.0002	0.0001	<0.0001
Total Molybdenum	mg/L	0.040	0.002	0.008
Total Nickel	mg/L	0.025	0.003	<0.003
Total Selenium	mg/L	0.1	0.002	<0.002
Total Silver	mg/L	0.0001	0.0001	<0.0001
Total Strontium	mg/L		0.005	0.257
Total Thallium	mg/L	0.0003	0.0003	<0.0003
Total Tin	mg/L		0.002	0.005
Total Titanium	mg/L		0.010	0.036
Total Tungsten	mg/L	0.030	0.010	<0.010
Total Uranium	mg/L	0.005	0.002	<0.002
Total Vanadium	mg/L	0.006	0.002	0.003
Total Zinc	mg/L	0.030	0.020	<0.020
Total Zirconium	mg/L	0.004	0.004	<0.004
Lab Filtration Aluminum Dissolved				2022/2/4
Lab Filtration mercury				2022/2/4

Certified By:

José Verástegui



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 22T859620

PROJECT: 3-21-0182-46

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: TERRAPROBE INC.

SAMPLING SITE: Barrie

ATTENTION TO: Justin Rumney

SAMPLED BY: Justin Rumney

Water Quality Assessment - PWQO (mg/L)

DATE RECEIVED: 2022-02-02

DATE REPORTED: 2022-02-08

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to PWQO * Variable - refer to guideline reference document
Guideline values are for general reference only. The guidelines provided may or may not be relevant for the intended use. Refer directly to the applicable standard for regulatory interpretation.
3474015 Dilution required, RDL has been increased accordingly.
Un-ionized Ammonia detection limit is a calculated RDL. The calculation of Un-ionized Ammonia is based on lab measured parameters (ammonia as N, pH and temperature). Values are reported as calculated.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

Iris Veraistegui



AGAT Laboratories

Exceedance Summary

AGAT WORK ORDER: 22T859620

PROJECT: 3-21-0182-46

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: TERRAPROBE INC.

ATTENTION TO: Justin Rumneey

SAMPLEID	SAMPLE TITLE	GUIDELINE	ANALYSIS PACKAGE	PARAMETER	UNIT	GUIDEVALUE	RESULT
3474015	BH2	ON PWQO	Water Quality Assessment - PWQO (mg/L)	Total Iron	mg/L	0.3	1.48

Quality Assurance

CLIENT NAME: TERRAPROBE INC.

PROJECT: 3-21-0182-46

SAMPLING SITE:Barrie

AGAT WORK ORDER: 22T859620

ATTENTION TO: Justin Rumney

SAMPLED BY:Justin Rumney

Water Analysis															
RPT Date: Feb 08, 2022			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Water Quality Assessment - PWQO (mg/L)															
Electrical Conductivity	3474986		30700	30900	0.6%	< 2	93%	90%	110%						
pH	3474986		7.63	7.67	0.5%	NA	101%	90%	110%						
Total Dissolved Solids	3474015	3474015	564	588	4.2%	< 10	102%	80%	120%						
Alkalinity (as CaCO3)	3474986		143	146	2.1%	< 5	99%	80%	120%						
Bicarbonate (as CaCO3)	3474986		143	146	2.1%	< 5	NA								
Carbonate (as CaCO3)	3474986		<5	<5	NA	< 5	NA								
Hydroxide (as CaCO3)	3474986		<5	<5	NA	< 5	NA								
Fluoride	3474015	3474015	<0.05	<0.05	NA	< 0.05	109%	70%	130%	106%	80%	120%	99%	70%	130%
Chloride	3474015	3474015	108	100	7.7%	< 0.10	91%	70%	130%	108%	80%	120%	103%	70%	130%
Nitrate as N	3474015	3474015	0.80	0.77	3.8%	< 0.05	96%	70%	130%	106%	80%	120%	104%	70%	130%
Nitrite as N	3474015	3474015	<0.05	<0.05	NA	< 0.05	93%	70%	130%	92%	80%	120%	98%	70%	130%
Bromide	3474015	3474015	<0.05	<0.05	NA	< 0.05	101%	70%	130%	109%	80%	120%	106%	70%	130%
Sulphate	3474015	3474015	51.0	48.0	6.1%	< 0.10	98%	70%	130%	106%	80%	120%	104%	70%	130%
Ortho Phosphate as P	3474015	3474015	<0.10	<0.10	NA	< 0.10	92%	70%	130%	99%	80%	120%	98%	70%	130%
Ammonia as N	3473280		0.03	0.03	NA	< 0.02	105%	70%	130%	103%	80%	120%	92%	70%	130%
Total Phosphorus	3474015	3474015	0.08	0.08	NA	< 0.02	101%	70%	130%	102%	80%	120%	103%	70%	130%
Total Organic Carbon	3474015	3474015	15.9	16.3	2.5%	< 0.5	97%	90%	110%	101%	90%	110%	NA	80%	120%
True Colour	3470143		<5	<5	NA	< 5	102%	90%	110%						
Turbidity	3474015	3474015	13.2	13.3	0.8%	< 0.5	101%	80%	120%						
Total Calcium	3473485		15.8	15.7	0.6%	< 0.10	93%	70%	130%	96%	80%	120%	100%	70%	130%
Total Magnesium	3473485		1.85	1.84	0.5%	< 0.10	96%	70%	130%	98%	80%	120%	103%	70%	130%
Total Potassium	3473485		<0.50	<0.50	NA	< 0.50	94%	70%	130%	97%	80%	120%	103%	70%	130%
Total Sodium	3473485		1.69	1.70	0.6%	< 0.10	95%	70%	130%	98%	80%	120%	104%	70%	130%
Aluminum-dissolved	3474015	3474015	0.010	0.008	NA	< 0.004	100%	70%	130%	103%	80%	120%	104%	70%	130%
Total Antimony	3478434		0.009	0.009	0.0%	< 0.001	99%	70%	130%	101%	80%	120%	104%	70%	130%
Total Arsenic	3478434		<0.003	<0.003	NA	< 0.003	97%	70%	130%	103%	80%	120%	108%	70%	130%
Total Barium	3478434		0.128	0.129	0.8%	< 0.002	99%	70%	130%	100%	80%	120%	104%	70%	130%
Total Beryllium	3478434		<0.001	<0.001	NA	< 0.001	101%	70%	130%	99%	80%	120%	112%	70%	130%
Total Boron	3478434		0.121	0.129	6.4%	< 0.010	99%	70%	130%	94%	80%	120%	108%	70%	130%
Total Cadmium	3478434		0.0002	0.0002	NA	< 0.0001	101%	70%	130%	101%	80%	120%	103%	70%	130%
Total Chromium	3478434		0.051	0.051	0.0%	< 0.003	101%	70%	130%	102%	80%	120%	100%	70%	130%
Total Cobalt	3478434		0.0018	0.0017	NA	< 0.0005	101%	70%	130%	105%	80%	120%	104%	70%	130%
Total Copper	3478434		0.009	0.010	10.5%	< 0.001	100%	70%	130%	104%	80%	120%	100%	70%	130%
Total Iron	3478434		1.31	1.32	0.8%	< 0.010	95%	70%	130%	101%	80%	120%	93%	70%	130%
Total Lead	3478434		<0.001	<0.001	NA	< 0.001	97%	70%	130%	98%	80%	120%	93%	70%	130%
Total Manganese	3478434		0.037	0.036	2.7%	< 0.002	99%	70%	130%	102%	80%	120%	102%	70%	130%
Dissolved Mercury	3474015	3474015	<0.0001	<0.0001	NA	< 0.0001	102%	70%	130%	97%	80%	120%	94%	70%	130%
Total Molybdenum	3478434		0.036	0.037	2.7%	< 0.002	102%	70%	130%	104%	80%	120%	109%	70%	130%
Total Nickel	3478434		0.045	0.045	0.0%	< 0.003	102%	70%	130%	107%	80%	120%	102%	70%	130%

Quality Assurance

CLIENT NAME: TERRAPROBE INC.

PROJECT: 3-21-0182-46

SAMPLING SITE: Barrie

AGAT WORK ORDER: 22T859620

ATTENTION TO: Justin Rumney

SAMPLED BY: Justin Rumney

Water Analysis (Continued)

RPT Date: Feb 08, 2022			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Total Selenium	3478434		0.005	0.003	NA	<0.002	106%	70%	130%	113%	80%	120%	110%	70%	130%
Total Silver	3478434		<0.0001	<0.0001	NA	< 0.0001	100%	70%	130%	103%	80%	120%	99%	70%	130%
Total Strontium	3478434		1.89	1.90	0.5%	< 0.005	98%	70%	130%	102%	80%	120%	105%	70%	130%
Total Thallium	3478434		<0.0003	<0.0003	NA	< 0.0003	95%	70%	130%	103%	80%	120%	99%	70%	130%
Total Tin	3478434		0.006	0.006	NA	< 0.002	110%	70%	130%	102%	80%	120%	113%	70%	130%
Total Titanium	3478434		<0.010	<0.010	NA	< 0.010	97%	70%	130%	104%	80%	120%	103%	70%	130%
Total Tungsten	3478434		<0.010	<0.010	NA	< 0.010	88%	70%	130%	85%	80%	120%	95%	70%	130%
Total Uranium	3478434		<0.002	<0.002	NA	< 0.002	97%	70%	130%	102%	80%	120%	101%	70%	130%
Total Vanadium	3478434		0.008	0.008	NA	< 0.002	100%	70%	130%	101%	80%	120%	105%	70%	130%
Total Zinc	3478434		0.039	0.048	NA	< 0.020	99%	70%	130%	105%	80%	120%	100%	70%	130%
Total Zirconium	3478434		<0.004	<0.004	NA	< 0.004	102%	70%	130%	103%	80%	120%	110%	70%	130%

Comments: NA signifies Not Applicable.

If the RPD value is NA, the results of the duplicates are under 5X the RDL and will not be calculated.

Matrix spike: Spike level < native concentration. Matrix spike acceptance limits do not apply.

Certified By:



Method Summary

CLIENT NAME: TERRAPROBE INC.

PROJECT: 3-21-0182-46

SAMPLING SITE:Barrie

AGAT WORK ORDER: 22T859620

ATTENTION TO: Justin Rumney

SAMPLED BY:Justin Rumney

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Water Analysis			
Electrical Conductivity	INOR-93-6000	modified from SM 2510 B	PC TITRATE
pH	INOR-93-6000	modified from SM 4500-H+ B	PC TITRATE
Saturation pH (Calculated)		SM 2320 B	CALCULATION
Langelier Index (Calculated)		SM 2330B	CALCULATION
Hardness (as CaCO ₃) (Calculated)	MET-93-6105	modified from EPA SW-846 6010C & 200.7 & SM 2340 B	CALCULATION
Total Dissolved Solids	INOR-93-6028	modified from EPA 1684,ON MOECC E3139,SM 2540C,D	BALANCE
Alkalinity (as CaCO ₃)	INOR-93-6000	Modified from SM 2320 B	PC TITRATE
Bicarbonate (as CaCO ₃)	INOR-93-6000	modified from SM 2320 B	PC TITRATE
Carbonate (as CaCO ₃)	INOR-93-6000	modified from SM 2320 B	PC TITRATE
Hydroxide (as CaCO ₃)	INOR-93-6000	modified from SM 2320 B	PC TITRATE
Fluoride	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Chloride	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Nitrate as N	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Nitrite as N	INOR-93-6004	SM 4110 B	ION CHROMATOGRAPH
Bromide	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Ortho Phosphate as P	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Ammonia as N	INOR-93-6059	modified from SM 4500-NH ₃ H	LACHAT FIA
Ammonia-Un-ionized (Calculated)		MOE REFERENCE, PWQOs Tab 2	CALCULATION
Total Phosphorus	INOR-93-6022	modified from SM 4500-P B and SM 4500-P E	SPECTROPHOTOMETER
Total Organic Carbon	INOR-93-6049	modified from SM 5310 B	SHIMADZU CARBON ANALYZER
True Colour	INOR-93-6074	modified from SM 2120 B	LACHAT FIA
Turbidity	INOR-93-6044	modified from SM 2130 B	NEPHELOMETER
Total Calcium	MET-93-6105	modified from EPA 6010D	ICP/OES
Total Magnesium	MET-93-6105	modified from EPA 6010D	ICP/OES
Total Potassium	MET-93-6105	modified from EPA 6010D	ICP/OES
Total Sodium	MET-93-6105	modified from EPA 6010D	ICP/OES
Aluminum-dissolved	MET-93-6103	modified from EPA 200.8 and EPA 3005A	ICP-MS
Total Antimony	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Arsenic	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Barium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Beryllium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Boron	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Cadmium	MET -93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Chromium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Cobalt	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Copper	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Iron	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS

Method Summary

CLIENT NAME: TERRAPROBE INC.

PROJECT: 3-21-0182-46

SAMPLING SITE: Barrie

AGAT WORK ORDER: 22T859620

ATTENTION TO: Justin Rumney

SAMPLED BY: Justin Rumney

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Total Lead	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Manganese	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Dissolved Mercury	MET-93-6100	modified from EPA 245.2 and SM 3112 B	CVAAS
Total Molybdenum	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Nickel	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Selenium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Silver	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Strontium	INOR-93-6003	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Thallium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Tin	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Titanium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Tungsten	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Uranium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Vanadium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Zinc	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Total Zirconium	MET-93-6103	modified from EPA 200.8, 3005A, 3010A & 6020B	ICP-MS
Lab Filtration Aluminum Dissolved	SR-78-9001		FILTRATION
Lab Filtration mercury	SR-78-9001		FILTRATION



Laboratories

* Faircliff

5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
web@earth.agatlabs.com

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Report Information:

Company: Terraprobe
Contact: Justin Rummey
Address: 220 Bayview Dr., Barrie
Phone: _____ Fax: _____
Reports to be sent to: Justin@terraprobe.ca
1. Email: _____
2. Email: _____

Project Information:

Project: 3-21-0152-46
Site Location: Barrie
Sampled By: Justin Rummey
AGAT Quote #: _____ PO: _____
Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Company: _____
Contact: Lorina Rossi
Address: _____
Email: Lrossi@terraprobe.ca
Bill To Same: Yes ☐ No ☐

Regulatory Requirements:

(Please check all applicable boxes)

☐ Regulation 153/04 ☐ Excess Soils R406 ☐ Sewer Use
☐ Sanitary ☐ Storm
Table Indicate One Table Indicate One Region _____
☐ Ind/Com ☐ Res/Park ☒ Prov. Water Quality
☐ Agriculture Objectives (PWQO)
Soil Texture (Check One) ☐ CCME ☐ Other
☐ Coarse ☐ Fine
☐ Fine

Is this submission for a Record of Site Condition?

☐ Yes ☐ No

Report Guideline on Certificate of Analysis

☒ Yes ☐ No

Sample Matrix Legend

B Biota
GW Ground Water
O Oil
P Paint
S Soil
SD Sediment
SW Surface Water

Field Filtered - Metals, Hg, CrVI, DOC

O. Reg 153

Metals & Inorganics
Metals - ☐ CrVI, ☐ Hg, ☐ HWSB
BTEX, F1-F4 PHCs
Analyze F4G if required ☐ Yes ☐ No

PAHs

PCBs

VOC

O. Reg 558

Landfill Disposal Characterization TCLP:
TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ Bery ☐ PCBs

Excess Soils SPLP Rainwater Leach

SPLP: ☐ Metals ☐ VOCs ☐ SVOCs

Excess Soils Characterization Package

pH, ICPS Metals, BTEX, F1-F4

Salt - EC/SAR

PWQO

X

Potentially Hazardous or High Concentration (Y/N)

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Y/N	Metals & Inorganics	Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	BTEX, F1-F4 PHCs	Analyze F4G if required ☐ Yes ☐ No	PAHs	PCBs	VOC	Landfill Disposal Characterization TCLP: TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ Bery ☐ PCBs	Excess Soils SPLP Rainwater Leach	SPLP: ☐ Metals ☐ VOCs ☐ SVOCs	Excess Soils Characterization Package pH, ICPS Metals, BTEX, F1-F4	Salt - EC/SAR	PWQO	Potentially Hazardous or High Concentration (Y/N)
BH2	11/21/22	12:40 AM	7	GW																
		AM																		
		PM																		
		AM																		
		PM																		
		AM																		
		PM																		
		AM																		
		PM																		
		AM																		
		PM																		
		AM																		
		PM																		

Samples Relinquished By (Print Name and Sign): <u>Justin Rummey</u>	Date: <u>11/21/22</u>	Time: _____	Samples Received By (Print Name and Sign): <u>Anthony Basile</u>	Date: <u>11/21/22</u>	Time: _____
Samples Relinquished By (Print Name and Sign): _____	Date: _____	Time: _____	Samples Received By (Print Name and Sign): _____	Date: _____	Time: _____
Samples Relinquished By (Print Name and Sign): _____	Date: _____	Time: _____	Samples Received By (Print Name and Sign): _____	Date: _____	Time: _____

Page _____ of _____
N: T **130347**

Laboratory Use Only

Work Order #: 22T859620
Cooler Quantity: 1 small
Arrival Temperatures: 2.5 | 6.2 | 5.6
Custody Seal Intact: ☐ Yes ☐ No ☐ N/A
Notes: Ice Pack

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

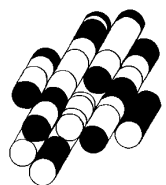
OR Date Required (Rush Surcharges May Apply): _____

Please provide prior notification for rush TAT
*TAT is exclusive of weekends and statutory holidays

For 'Same Day' analysis, please contact your AGAT CPM

APPENDIX G

TERRAPROBE INC.



APPENDIX F - Based Water Balance (Entire Site) - 642 Dunlop Street West, Barrie

File No. 3-21-0182-46

1. Climate Information

Precipitation ¹	952 mm/a	0.95 m/a
Actual Evapotranspiration ¹	420 mm/a	0.42 m/a
Precipitation Surplus	532 mm/a	0.53 m/a

2. Infiltration Rates**Table 3 Approach - Infiltration Factors²***(Pre and Post Development)*

Soil: Peat		
Infiltration	399 mm/a	0.399 m/a
Run-off	133 mm/a	0.133 m/a

3. Drainage A (Catchment Area) - Property Statistics

			Percentage
Pre- Development Site Coverage			
Building Area/Coverage	200 m ²	0.02 ha	3 %
Hardscape/Impervious	1,200 m ²	0.12 ha	20 %
Softscape/Landscape (Undeveloped area)	4,600 m ²	0.46 ha	77 %
TOTAL	6,000.00 m²	0.60 ha	100 %

4. Entire Site Property Statistics

			Percentage
Pre- Development Site Coverage			
Building Area/Coverage	200 m ²	0.02 ha	2 %
Hardscape/Impervious	2,800 m ²	0.28 ha	35 %
Softscape/Landscape (Undeveloped area)	5,100 m ²	0.51 ha	63 %
TOTAL	8,100.00 m²	0.81 ha	100 %
Post-Development Coverage			
Building Area (Roof Runoff)	2,400 m ²	0.24 ha	30 %
Hardscape/Impervious (Paved area- Run-off Discharge to Municipal Strom	3,000 m ²	0.30 ha	37 %
Softscape/Landscape (B1 (Pervious Area)	2,700 m ²	0.27 ha	33 %
TOTAL:	8,100.00 m²	0.81 ha	100 %

Notes/References:

Water Balance Analysis Method: Thornthwaite and Matther approach

1 Lake Simcoe Climate Data Reference Document (April 2017)

2 MOEE Technical Information Requirements for Land Development Applications (1995)

APPENDIX F - Water Balance (Entire Site)- 642 Dunlop Street West, Barrie

File No. File No. 3-21-0182-46

5. Drainage A- Catchment Area)Annual Water Balance (Pre Development)

Land Use	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Building Area/Coverage	200	190	nil	19	nil	171
Hardscape/Impervious	1,200	1,142	nil	114	nil	1,028
Softscape/Landscape	4,600	4,379	1,932	nil	1,835	612
TOTAL	6,000	5,712	1,932	133	1,835	1,811

4. Annual Water Balance (Pre Development)

Land Use	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Building Area/Coverage	200	190	nil	19	nil	171
Hardscape/Impervious	2,800	2,666	nil	267	nil	2,399
Softscape/Landscape	5,100	4,855	2,142	nil	2,035	678
TOTAL	8,100	7,711	2,142	286	2,035	3,249

6. Annual Water Balance (Post Development)

Land Use	Area (m ²)	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Building Area/Coverage	2,400	2,285	nil	228	nil	2,056
Hardscape/Impervious areas	3,000	2,856	nil	286	nil	2,570
Softscape/Landscape	2,700	2,570	1,134	nil	1,077	359
TOTAL	8,100	7,711	1,134	514	1,077	4,986

7. Comparison of Pre-Development (before buidling additions) and Post-Development (after building additions)

	Precipitation (m ³)	Evapotranspiration (m ³)	Evaporation (m ³)	Infiltration (m ³)	Run-Off (m ³)
Pre-Development	7,711	2,142	286	2,035	3,249
Post-Development	7,711	1,134	514	1,077	4,986
Difference	0	-1,008	228	-958	1,737

8. Requirement for Infiltration Maintaenance from roof run-off

Volume of roof run-off captured (90%) (roof run-off available in post development)	1,851 m ³
Volume of roof run-off required to match pre-development Run-off to Adjacent North Wetland (Runoff deficit in post development)	1,811 m ³
Percentage of roof run-off required to match pre-development infiltration	98%
Volume of roof run-off required to match pre-development Infiltration rates (site infiltration deficit in post development)	958 m ³
Volume of Run-Off remaining after post development run-off available for wetland adjacent north of the Site	39.4
Final-Volume of Site Infiltration deficit in post development condition	918 m ³

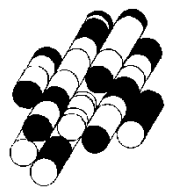
Appendix A: Climate Data Tables

Barrie Creeks Subwatershed

Hydrologic Soil Group		Subwatershed Area (km ²)	Mean Annual Precipitation (mm/yr.)	Actual Evapotranspiration (mm/yr.)	Precipitation Surplus (mm/yr.)
Urban Lawns/Golf Courses					
Fine Sand	A	1.74	952	525	427
Fine Sandy Loam	B		952	539	413
Silt Loam	C		952	573	380
Clay	D		952	643	310
Forest					
Fine Sand	A	4.12	952	521	431
Fine Sandy Loam	B		952	540	412
Silt Loam	C		952	434	518
Clay	D		952	598	354
Pasture & Shrubs					
Fine Sand	A	0.40	952	565	387
Fine Sandy Loam	B		952	546	406
Silt Loam	C		952	558	394
Clay	D		-	-	-
Non-Intensive Agriculture (e.g. Hay)					
Fine Sand	A	0.92	952	528	424
Fine Sandy Loam	B		952	636	316
Silt Loam	C		-	-	-
Clay	D		-	-	-
Intensive Agriculture (e.g. Row crop)					
Fine Sand	A	0.43	952	556	396
Fine Sandy Loam	B		952	532	420
Silt Loam	C		-	-	-
Clay	D		-	-	-
Open Alvar					
Fine Sand	A	-	-	-	-
Fine Sandy Loam	B		-	-	-
Silt Loam	C		-	-	-
Clay	D		-	-	-
Aggregates					
Fine Sand	A	0.62	952	471	481
Fine Sandy Loam	B		952	456	496
Silt Loam	C		-	-	-
Clay	D		-	-	-
Mean Annual			952	446	506
Notes: Precipitation and Actual Evapotranspiration values are the AVERAGE ANNUAL estimates obtained from the Lake Simcoe PRMS model (Earthfx, 2010).					

APPENDIX H

TERRAPROBE INC





Terraprobe

File No. 3-21-0182-46.1

642 Dunlop St. W., Barrie, ON

Monitoring Well I.D.	MW1		MW 2		MW 3		MW 4		MW 5		MW 9	
Stick Up (magl)	0.0		0.9		0.0		0.0		0.0		0.9	
Depth (mbgl)	-		11.7		7.2		11.8		4.4		4.4	
Geodetic Elevation (masl)	235.5		234.5		235.3		235.4		235.2		234.4	
Drilling Date	19-Jan-22		20-Jan-22		18-Jan-22		19-Jan-22		19-Jan-22		18-Jan-22	
Data Logger Installation Date	-		-		-		-		-		-	
Date	Water Level (mbgl)	Water Elevation (masl)	Water Level (mbgl)	Water Elevation (masl)	Water Level (mbgl)	Water Elevation (masl)	Water Level (mbgl)	Water Elevation (masl)	Water Level (mbgl)	Water Elevation (masl)	Water Level (mbgl)	Water Elevation (masl)
1-Feb-22	#	Not Measured	1.74	232.76	1.77	233.53	1.91	233.49	1.49	233.71	1.73	232.67
31-Mar-22	#	Not Measured	1.10	233.40	1.77	233.53	1.24	234.16	0.78	234.42	1.06	233.34
14-Apr-22	#	Not Measured	1.02	233.48	1.09	234.21	1.24	234.16	0.78	234.42	1.07	233.33
2-May-22	#	Not Measured	1.11	233.39	1.22	234.08	1.37	234.03	0.91	234.29	1.17	233.23
22-Jun-22	#	Not Measured	1.46	233.04	1.57	233.73	1.57	233.83	1.23	233.97	1.54	232.86
4-Jul-22	#	Not Measured	0.66	233.84	1.78	233.52	1.91	233.49	1.43	233.77	1.74	232.66
2-Aug-22	#	Not Measured	1.96	232.54	2.10	233.20	2.25	233.15	2.89	232.31	2.04	232.36
14-Sep-22	#	Not Measured	2.03	232.47	2.12	233.18	2.27	233.13	1.82	233.38	1.65	232.75
3-Oct-22	#	Not Measured	1.93	232.57	2.07	233.23	1.73	233.67	1.76	233.44	1.65	232.75
1-Nov-22	#	Not Measured	1.98	232.52	2.14	233.16	1.81	233.59	1.83	233.37	2.14	232.26
1-Dec-22	#	Not Measured	2.06	232.44	2.22	233.08	1.90	233.50	1.92	233.28	2.23	232.17

Note:

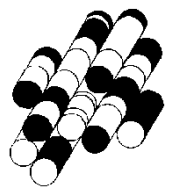
mbgl - Metres Below Ground Level

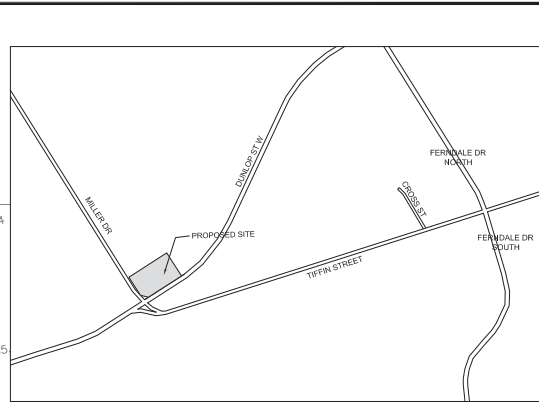
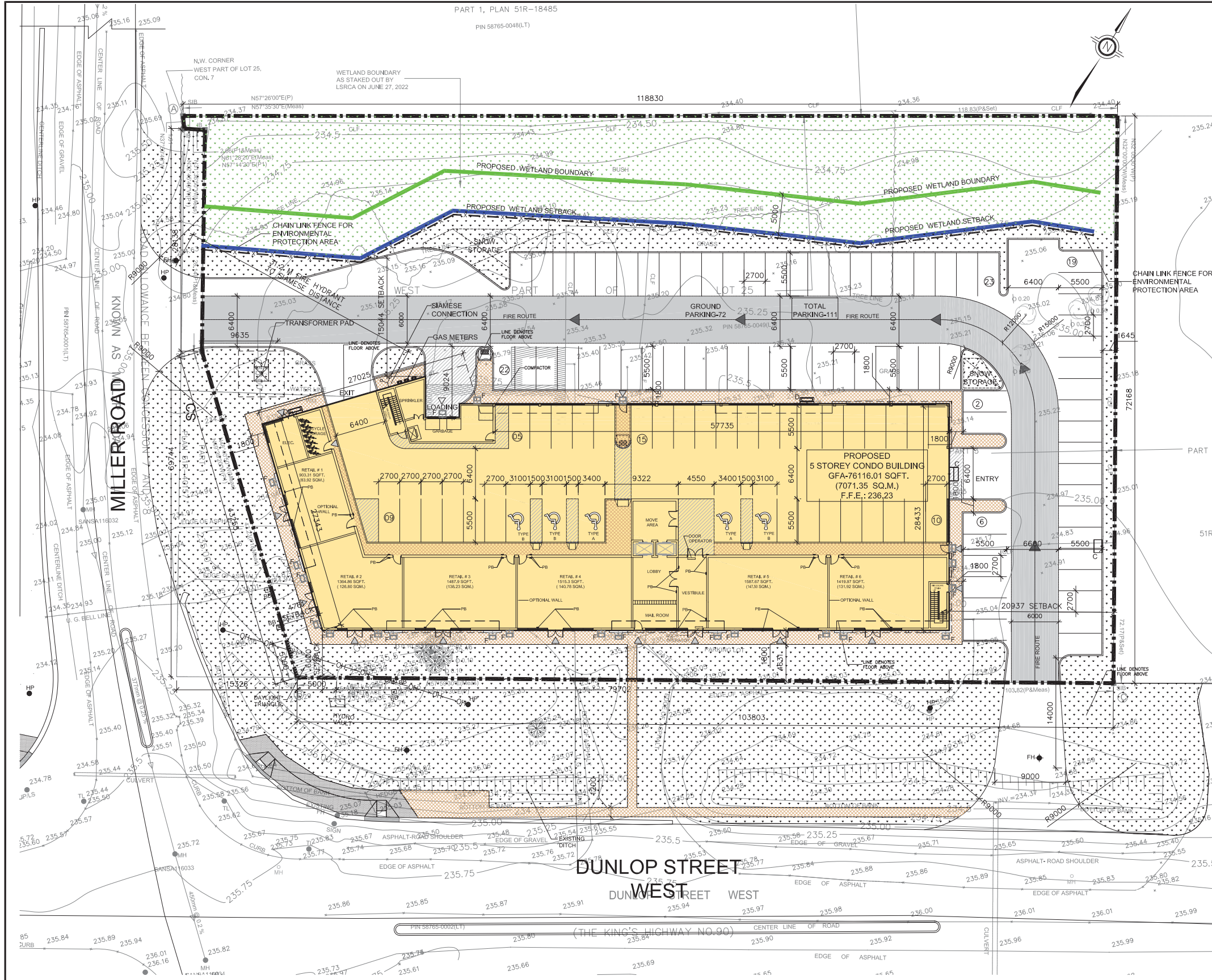
masl - Metres Above Sea Level

MW1 was not found, first covered in snow, later in water flooded area.

APPENDIX I

TERRAPROBE INC





KEY PLAN

PROJECT STATISTICS		
ADDRESS 642 DUNLOP STREET WEST, BARRIE, ON.		
ZONING	GENERAL COMMERCIAL (C4)	
	REQUIRED	PROPOSED
LOT(SITE) AREA (m²)	450 SQ.M.	8084.59 SQ.M.
RETAIL		778.89 SQ.M.
RESIDENTIAL UNITS		6233.07 SQ.M.
AMENITY AREA		341.95 SQ.M.
TOTAL G.F.A.		7071.35 SQ.M.
COVERAGE	50% MAX	2420.23 SQ.M. (29.93%)
LANDSCAPE AREA		2811.57 SQ.M. (34.77%)
PAVED AREA		522.91 SQ.M. (6.47%)
BUILDING HEIGHT (MAX.)	14.0 M.	13.95 M.
TOTAL PARKING (MIN.)	128 CARS	111 CARS
BARRIER FREE PARKING (MIN.)	5 CARS	5 CARS
BICYCLE PARKING (MIN.)	4	7
LOADING SPACE (MIN.)	1	1
LOT FRONTAGE (MIN.)	15 M.	69.7 M.
DUNLOP STREET WEST		

BUILDING SETBACKS		
	REQUIRED	PROPOSED
FRONT YARD (WEST)	6.0M	4.76 M.
EXTERIOR SIDE YARD (SOUTH)	5.0M	6.0 M.
INTERIOR SIDE YARD (NORTH)	3.0M	15.04 M.
REAR YARD (EAST)	7.0M	20.93 M.

PARKING CALCULATIONS				
ITEM	AREA(SQM.)	RATE	REQUIRED	PROVIDED
RETAIL	769.15	1 PER 24 SQ.M.	32	16
RESIDENTIAL UNITS (72)	6233.07	1.25 PER DWELLING	90	90
H.C. PARKING (OVER 100 - 1+3% OF REQ. PARKING)			5	5
TOTAL			127	111
CAR SPACE SIZE			2.7X5.5M	2.7X5.5M
H/C CAR SPACE SIZE (TYPE-A)			3.4X5.5M	3.4X5.5M
(TYPE-B)			3.1X5.5M	3.1X5.5M

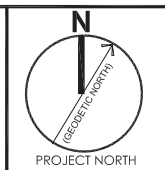
- LEGEND
- NEW BUILDING
 - ASPHALT
 - LANDSCAPE
 - CONC. PAVEMENT, 150MM RAISED
 - HANDICAP PARKING
 - MAIN ENTRANCE
 - OVERHEAD DOOR
 - TREE
 - BARRIER CURB
 - BARRIER FREE CURB
 - PB BARRIER FREE PUSH BUTTON

MUNICIPAL ADDRESS & LEGAL DESCRIPTION:
PART 3 OF LOT 25 REGISTERED PLAN F45 CITY OF BARRIE COUNTY OF SIMCOE

APPLICANT:
n Architecture Inc., 9120 Leslie St., Suite 208, Richmond Hill, Ontario L4B 3J9
T: 905.597.5937 F:1.866.340.5265
E: info@narchitecture.com
WEB: www.narchitecture.com

SURVEY INFORMATION TAKEN FROM:	
OMARI MWINYI SURVEYING LTD. 1905 PINE GROVE AVENUE PICKERING ONTARIO, L1V 6T1 Ph: (905) 728-8863 Fax:(905) 492 5485	
TOTAL UNITS:	
TWO BED ROOM UNITS	33
ONE BED ROOM UNITS	27
ACCESSIBLE ONE BED ROOM UNITS	06
ACCESSIBLE TWO BED ROOM UNITS	06
TOTAL	72

n Architecture Inc
PRINCIPAL: NITIN MALHOTRA, ARCHITECT.
9120 Leslie Street, Suite-208
Richmond Hill, Ontario. L4B 3J9
T : 4 1 6 . 2 5 6 . 9 7 4 1
E: info@narchitecture.com
www.narchitecture.com



13th MARCH 2023
NOT FOR CONSTRUCTION

No.	Date	Version	Dwn.
3.		ISSUED FOR SPA	SB
2.	13-JUN-2018	ISSUED FOR PRE-CONSULTATION	SB
1.	04-APR-2016	ISSUED FOR PRE-CONSULTATION	FK

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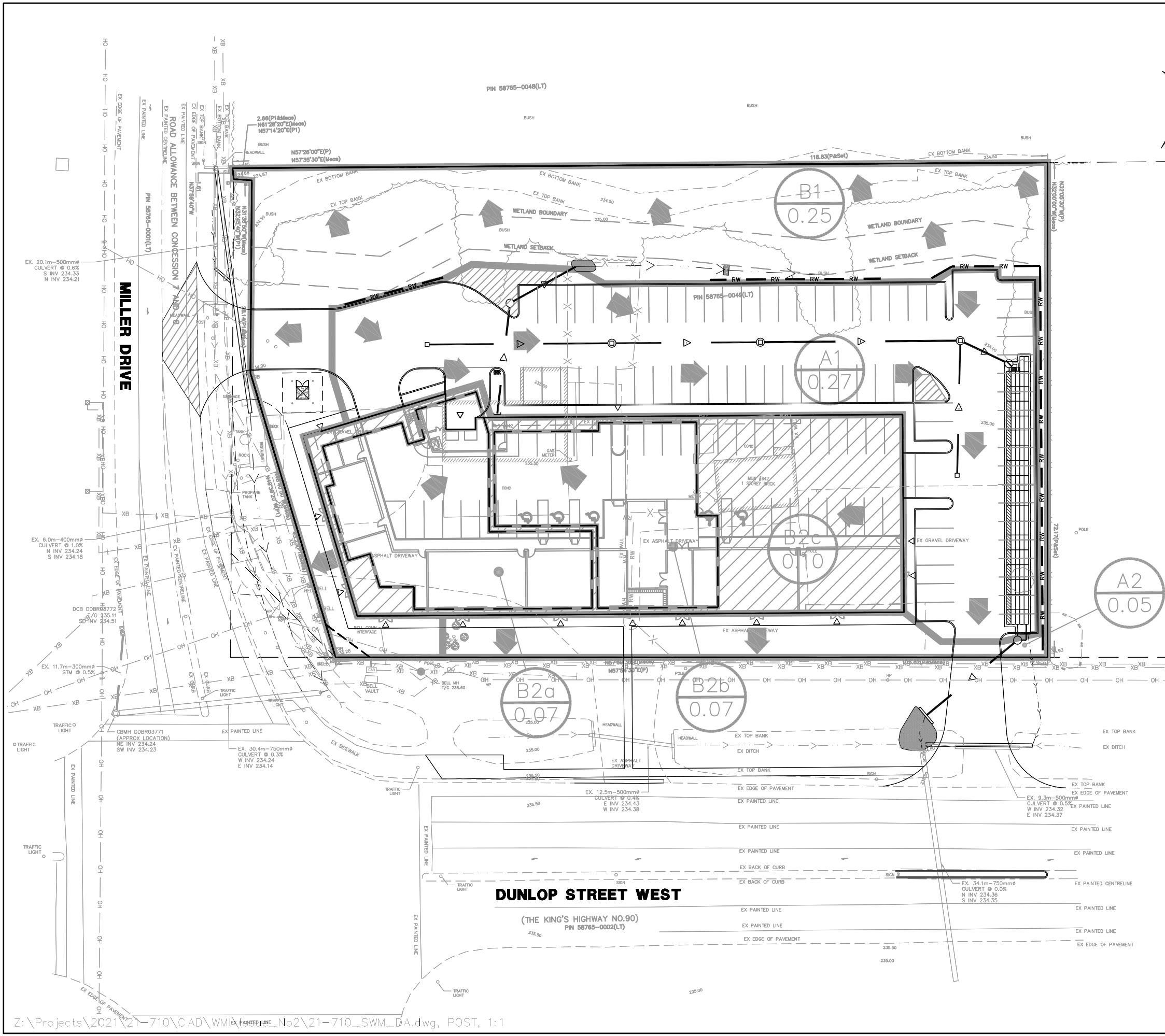
PROJECT:

**PROPOSED 5-STOREY CONDO BUILDING
642 DUNLOP STREET W
BARRIE, ON.**

DRAWING TITLE:	
SITE PLAN	
DRAWN BY: FK	DATE: DEC 02 2015
CHECKED BY: NM	SCALE: AS NOTED
PROJECT NO.:	DRAWING NO.:

15-48 A-1.0

1 SITE PLAN
A-1.0 SCALE: 1:250



Legend:

0
0.00

CATCHMENT IDENTIFICATION

CATCHMENT AREA (HA)

LIMITS OF CATCHMENT AREA

OVERLAND FLOW DIRECTION

Drawing Title

POST-DEVELOPMENT DRAINAGE PLAN

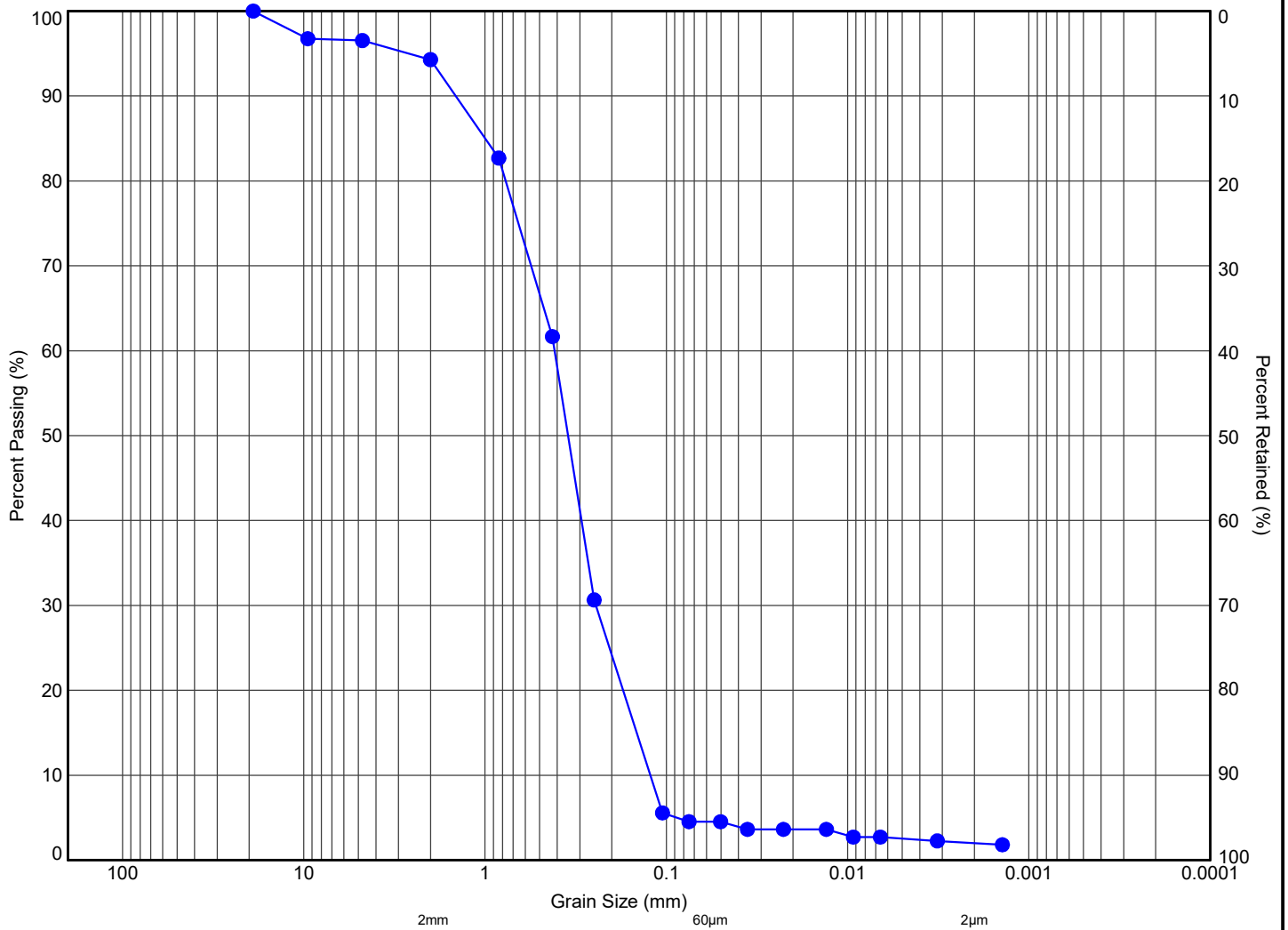
Project Title

642 DUNLOP STREET WEST

WMI & Associates Limited
119 Collier Street
Barrie, Ontario
L4M 1H5
705-797-2027
www.wmiengineering.ca

Drawn By	AB	Checked By	JR	Figure No.
Scale	N.T.S.	Project No.	21-710	

FIG 3



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
1	SS5	3.3	232.2	6	89	3	2		



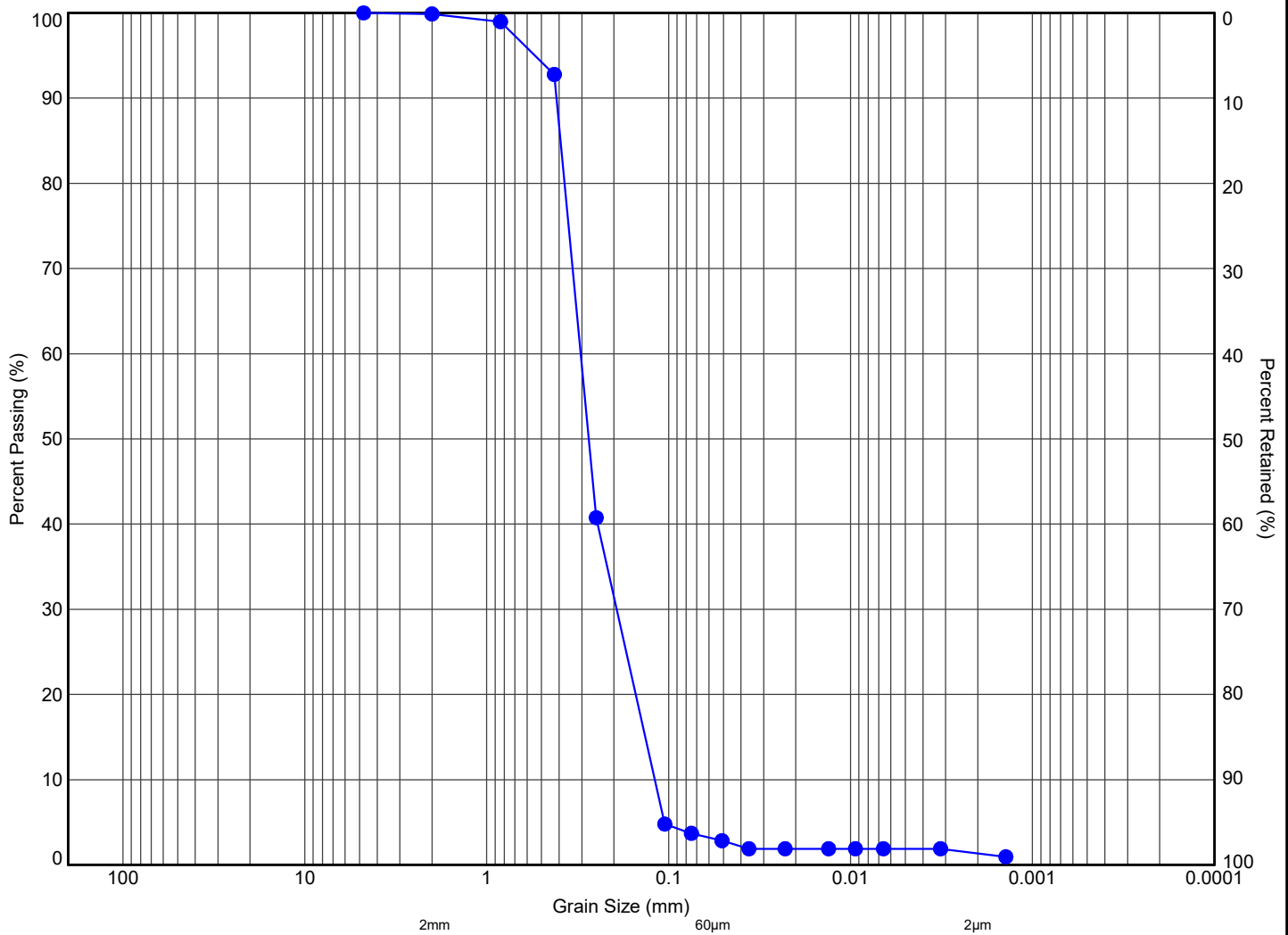
11 Indell Lane, Brampton Ontario L6T 3Y3
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Title:

**GRAIN SIZE DISTRIBUTION
SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY**

File No.:

3-21-0182-01



T-Time Analysis

The results of the soil gradation are appended and estimated Hydraulic Conductivity and T-Time are summarized below:

Borehole No. Sample No.	Sampling Depth below Grade (m)	Percentage (by mass)				Descriptions (MIT System)
		Gravel	Sand	Silt	Clay	
Borehole 1 Sample 5	3.3	6	89	3	2	SAND, trace gravel trace silt, trace clay
Borehole 2 Sample 4	2.5	0	97	2	1	SAND, trace silt, trace clay

Based on the results of the grain size analysis, the estimated hydraulic conductivity is as follows,

Borehole No. Sample No.	Sampling Depth below Grade (m)	Sampling Elevation (m)	Particle Size Corresponding to 10% Finer (D ₁₀) (mm)	Estimated Hydraulic Conductivity (cm/s)	Estimated T- Time(mins/cm)
Borehole 1 Sample 5	3.3	232.2	0.13	1.7×10^{-2}	5
Borehole 2 Sample 4	2.5	232.0	0.13	1.7×10^{-2}	5

Based on the grain size analysis results, the estimated hydraulic conductivity of the site soil samples (Sand) analyzed is estimated to be 1.7×10^{-2} cm/sec, which corresponds to a T-Time of 5 mins/cm based on the Supplementary Standards to the Ontario Building Code 2012 document Percolation Time and Soil Descriptions (SB-6).

It should be noted that the soil permeability/T-Time as noted above is estimated based on the composition of the soil sample tested. It should also be noted that the soil conditions may vary between and beyond the boreholes and there can also be variation within the sand layer.