Site Servicing & Stormwater Management Report

265 Whitfield Crescent Town of Midland

July 2019
WMI File # 19-543

Prepared by

WMI & Associates Limited
119 Collier Street, Barrie Ontario L4M 1H5

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1.0 Introduction

1.1 General

WMI & Associates Limited was retained by the Jason Redman to prepare a Site Servicing and Stormwater Management Report for the proposed commercial development located at 265 Whitfield Crescent, in the Town of Midland.

1.2 Background

The subject site is situated on approximately 0.42 hectares of land on the west side of Whitfield Crescent. The general location of this property is illustrated on **FIG 1** in **Appendix A** (Site Location Plan) and will be referred to as the "site" within the context of this report. The Site Plan for the project has been prepared by WMI & Associates Ltd. (dated June 20, 2019) and is included in **Appendix A.**

The property is legally described as being Part 8 Plan 51R-19678 which is Part of Lot 100, Concession 2, Town of Midland, and Part of Lot 100, Concession 2, Township of Tay, County of Simcoe.

The 0.42ha subject property currently consists of a large gravelled area (where the overlying soils have been stripped for use as fill off-site), with the remaining area consisting of unimproved land overgrown with vegetation.

It is proposed to construct three (3) 1-storey self-storage buildings, as well as a gravel parking area accessed by a proposed site entrance from the Whitfield Crescent right-of-way (ROW) that spans east-west parallel to the southern property line.

The stormwater management features that have been designed for this site consist of a grass swale, storm sewer, and a dry detention basin which will form an integrated treatment train providing both stormwater quality and quantity control for the proposed development.

2.0 Pre-Development Condition

2.1 General

All pre-development runoff from the site takes place in the form of overland sheet flow over the unimproved land. The elevation drop across the site is approximately 10m. In general, the site slopes from the northwest to the southeast causing runoff to drain primarily to the ditch located within the Whitfield Crescent ROW. All site runoff is collected by the existing ditches within the Whitfield Crescent right-of-way (ROW). One pre-development catchment (PRE = 0.42ha) was used to analyze the existing condition.

A 0.78ha external drainage area currently contributes runoff to the site's northern property line. A cut-off swale will be constructed along the southern property line of the neighbouring property to the north of the subject site by the respective land owner (based on discussions with the proponent). All other existing grades slope away from the site and as a result, the site is considered to be self-contained for the purposes of the stormwater management design.

The pre-development condition drainage boundaries have been confirmed through a combination of topographic survey, Simcoe County Interactive Maps GIS topographic contours, and a site visit.

Refer to **FIG2** in **Appendix A** for the Pre-Development Drainage Plan.

2.2 Soil Conditions

According to the Soils Map of Simcoe County, Ontario, Soil Survey Report prepared for the Department of Agriculture, the subject site consists primarily of Tioga and Vasey Sandy Loam. These soil types are within Hydrologic Soil Group 'A-AB' and are considered to be good draining soils.

A Geotechnical Investigation Report prepared by Cambium Inc., (April 1, 2019) has been prepared for the site. Two (2) test-pits were dug on-site and revealed varying layers of topsoil overtop of native gravelly silty sand soils. The report concludes that the native gravelly silty sands on the site have high permeability, specifically estimating the T-time to be 20min/cm (Infiltration rate of 30mm/hr).

Out of the two (2) test-pits dug on-site, groundwater was not observed. The Cambium Report states that the groundwater was encountered in boreholes located at 1000 William Street just across from the subject site, but that it is likely perched due to the native soils' low permeability and the recent addition of fill overtop of the native soils. Bedrock was not encountered in any of the test-pits.

The runoff coefficients and curve numbers associated with the site drainage area were determined by calculating weighted values based on corresponding land uses and soil type. The Hydrologic Soil Group was determined in accordance with the Ontario Ministry of Transportation (MTO) Soil Classification System.

2.3 Stormwater Management Design Criteria

The stormwater management design for the site will incorporate the policies and criteria of a number of agencies, including the Ministry of the Environment Conservation and Parks (MECP), Severn Sound Environmental Association (SSEA), and the Town of Midland (Town). Additional design guidance has been provided by the Low Impact Development Stormwater Management Planning & Design Guide (LID) prepared by the Credit Valley Conservation (CVC) and the Toronto and Region Conservation Authority (TRCA), Version 1.0, dated 2010.

The above noted agencies stormwater design criteria for the proposed development are summarized below:

- Stormwater quality controls will be provided based on the guidelines described in the Ministry of the Environment, Stormwater Management Planning and Design Manual dated March 2003 and the Low Impact Development Stormwater Management Planning & Design Guide (LID) prepared by the Credit Valley Conservation (CVC) and the Toronto and Region Conservation Authority (TRCA), Version 1.0, dated 2010. Following the Ministry of Environment Conservation and Parks (MECP) and LID Guidelines noted above, the stormwater management design utilized for the site will provide water quality control at an Enhanced Level of Protection (minimum of 80% Total Suspended Solids removal efficiency).
- Stormwater quantity control will be provided via the use of an on-site dry detention basin sized to accommodate the storage volume required to attenuate post-development peak flows to corresponding pre-development target rates or less for each of the 2-100 year design storm events. The dry detention basin will include an outlet structure consisting of an orifice plate installed on an outlet pipe which will control all outflows from the site to the Whitfield Crescent ROW.
- Stormwater quality control will be provided via the use of a dry detention basin in conjunction with a grass swale and a storm sewer system outfitted with Nyloplast Envirohoods and deep sumps within each structure upstream for pre-treatment. This proposed treatment train approach is premised on the stormwater being both filtrated as well as infiltrated into the in-situ soils while the vegetation will also provide nutrient uptake and evapotranspiration benefits.

- The Ministry of Transportation Rainfall Intensity-Duration-Frequency (IDF) Lookup data, was used to determine the peak design flows and runoff volumes for each of the design storm events analyzed.
- Erosion and Sediment Control measures will be implemented prior to and during the construction of the development and maintained until the site is stabilized.

3.0 Post-Development Condition

3.1 General

With the intention of maintaining pre-development peak flow rates and water quality, post-development drainage patterns have generally been kept consistent with that of the pre-development condition. Due to the increase in impervious area in the post-development condition, an integrated treatment train of Low Impact Development Best Management Practices (LID BMP's) have been proposed to fully address stormwater quality control and water balance. In the post-development condition, the site remains as a single catchment, referred to as POST (0.42ha).

3.2 Post-Development Drainage

The site will be comprised of three (3) 1-storey buildings (slab on grade), as well as a gravel parking area accessed by the site entrance located at the eastern property line from the Whitfield Crescent right-of-way (ROW). Stormwater will be captured and conveyed by the proposed LIDs to the site outlet. The site will be graded to provide positive drainage towards each of the proposed LIDs.

A grass swale located along the northern property boundary is proposed to convey flows mainly from the sloped landscape area north of the swale to a dry detention basin located in the northeast corner of the site. The grass swale will provide filtration and evapotranspiration benefits, as well as nutrient uptake and opportunity for infiltration into the in-situ soils. The grass swale will run perpendicular to the direction of overland flow to intercept runoff and provide maximum opportunity for quality control and water balance benefits prior to discharging into the downstream dry detention basin.

The remainder of the site will drain to catchbasins within the access road and be conveyed to the dry detention basin by a storm sewer. The catchbasins will have deep sumps and fitted with Nyloplast Envirohoods to prevent sediment and floating debris/oil from entering the storm sewer.

The dry detention basin will be designed to provide further stormwater quality control and at-source groundwater recharge for water balance purposes as well as an outlet structure that will provide stormwater quantity control.

Refer to **FIG3** (Post-Development Drainage Plan) and **SGR** (Site Servicing and Grading Plan) in **Appendix A**.

4.0 Hydrologic Analysis

4.1 Pre-Development Condition Results

Using the site drainage area as illustrated on **FIG2** and the Rational Method, the total flows were determined for the 2, 5, 10, 25, 50 & 100-year design storm events. These flows are summarized in **Table 1** below. The stormwater management design calculations including the Rational Method peak flow values can be found in **Appendix B**.

Table 1: Pre-Development Peak Flows

Catchment	Area (ha)	Pre-Development Peak Flows					
		2 yr. m^3/s	5 yr. m^3/s	10 yr. m^3/s	25 yr. m^3/s	50 yr. m^3/s	100 yr. m^3/s
PRE	0.42	0.006	0.009	0.010	0.013	0.016	0.018

4.2 Post-Development Condition Results

The post-development peak flows are summarized in **Table 2** below.

Table 2: Post-Development Uncontrolled Peak Flows

Catchment	Area (ha)	Post-Development Uncontrolled Peak Flows					
		2 yr. m^3/s	5 yr. m^3/s	10 yr. m^3/s	25 yr. m^3/s	50 yr. m^3/s	100 yr. m^3/s
POST	0.42	0.041	0.055	0.064	0.082	0.100	0.114

By comparing **Tables 1** and **2** for the total site drainage area, it is evident that the post-development peak flows exceed the pre-development levels. Attenuation of post-development peak flows to pre-development levels or less will be provided as discussed in **Section 5.0** below.

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

5.0 Stormwater Quantity Control

The table below (**Table 3)** summarizes the storage volume requirements for the stormwater management basin (dry detention basin) and the corresponding inflow-outflows and estimated water levels. The storage volumes were determined using the Modified Rational Method and the calculations can be found in **Appendix B**.

Table 3: Dry Detention Basin Characteristics

Storm Event (Year)	Dry Detention Basin Post-Development Controlled Peak Flows (m^3/s) & Storage Volumes (m^3)				
	Drainage Area (ha)	Inflow (m^3/s) (Table 2)	Outflow (m^3/s)	Storage Provided (m^3)	Estimated Water Levels (m)
2	0.42	0.041	0.0083	37.32	186.68
5		0.055	0.0095	54.38	186.83
10		0.064	0.0102	66.02	186.91
25		0.082	0.0113	92.67	187.09
50		0.100	0.0122	119.09	187.22
100		0.114	0.0129	142.02	187.32

The proposed dry detention basin has been designed to incorporate stormwater quantity control as well as provide additional quality control prior to releasing runoff to the existing stormwater outlet (the existing west ditch within in the Whitfield Crescent ROW).

Details of the proposed dry detention basin are summarized below:

- The site's internal grading has been designed such that during a 100-year design storm event, all stormwater runoff is safely conveyed both overland and via storm sewer towards the proposed dry detention basin. The proposed storm sewer, grass swale, and gravel drive aisles have been designed and graded to safely convey the site's major system flows to the dry detention basin prior to being attenuated and released to the existing stormwater outlet.
- The dry detention basin consists of 3:1 (H:V) side slopes, a maximum depth of 1.3m and a total storage capacity of 187.2m^3.
- The proposed dry detention basin will consist of a concentrated flow overland inlet stemming directly from the grass swale at its northwest corner and a concentrated flow inlet stemming from the storm sewer pipe at its southwest corner. These proposed inlets will convey all contributing design storm peak flows up to and including the 100-year design storm from the site directly into the dry detention basin. All inlets will be lined with filter cloth and rip-rap for erosion protection.

- The proposed outlet structure (orifice plate) will cause runoff to pool within the basin as it attenuates post-development peak flows. This design will utilize the full volume provided within the basin and force runoff to contact the entire base area before being released to the existing outlet. Runoff contacting the base area will experience the full potential of the quality control benefits via vegetative filtration, evapotranspiration, infiltration, and nutrient uptake via vegetative cover. Moreover, the pooling of runoff in the basin will allow further sedimentation of suspended solids as intended prior to runoff being released to the site's outlet.
- The proposed dry detention basin is designed to attenuate the stormwater runoff generated by the development prior to releasing it to the site outlet. An orifice plate installed on the upstream end of the basin's outlet pipe is proposed to control the 2-100 year design storm peak flows directed into the dry detention basin. The orifice plate will have a diameter of 0.075m and have an invert elevation of 186.20m. A perforated riser pipe complete with rip-rap cover will be constructed to prevent sediment and debris from obstructing the orifice. This outlet configuration has been designed to provide sufficient stormwater attenuation within the dry detention basin to control the post-development peak flows to the corresponding pre-development target rates or less for each of the 2-100 year design storm events.
- In the event that there is an obstruction of the outlet structure or during storm events less frequent than the 100-year design storm, an earthen trapezoidal overflow weir will be constructed in the eastern bank of the dry detention basin. The weir will have capacity to convey the uncontrolled 100-year peak flow (0.114m^3/s, conservative) from the basin safely to the site's outlet, the existing west ditch in the Whitfield Crescent ROW.
- The dry detention basin will have a maximized base area of 51m^2 in an effort to provide at-source groundwater recharge for water balance purposes.

Refer to **Appendix A** for the engineering drawing set detailing the proposed dry detention basin, as well as **Appendix B** for supporting calculations and design details of the basin.

6.0 Stormwater Quality Control

6.1 Total Suspended Solids Removal Initiatives

In determining the best approach to provide quality control for the proposed development, various factors were considered, as follows:

- Existing land characteristics and uses (soils, topography, treatment area, location, etc.);
- Local requirements and maintenance considerations with regard to quality control;
- Facility feasibility & proximity to a suitable stormwater outlet.
- Utilizing an 'integrated treatment train' approach to treat stormwater runoff;
- Ability to utilize landscaped areas for nutrient uptake and evapotranspiration benefits;

Based on the above noted factors, the application of a dry detention basin in conjunction with a grass swale and a storm sewer system outfitted with a Nyloplast Envirohood and deep sump within each structure upstream for pre-treatment. This proposed treatment train approach is premised on the stormwater being both filtrated as well as infiltrated into the in-situ soils while the vegetation will also provide nutrient uptake and evapotranspiration benefits.

Referencing the LID & MOE Guidelines, the site's impervious area (rooftops and gravel parking lot) is directed to Low Impact Development Best Management Practices (LID BMP) capable of providing quality control benefits. An 'Enhanced' Level of Protection, as defined in the MOE's Stormwater Management Planning & Design Manual will be achieved through filtration practices.

The dry detention basin is proposed to capture and release all stormwater runoff from the property and has been designed based on the same principles of an enhanced grass swale (as suggested in the LID Manual). Enhanced grass swales (dry detention basin) are considered advantageous as they can be integrated into the various landscape areas proposed throughout a site. From a performance perspective they are beneficial in that they can function adequately when graded into areas of varying slope and will provide exceptional capture due to the longitudinal dimension and location relative to the proposed site grading (perpendicular overland to the direction of flow). The design of an enhanced grass swale (dry detention basin) is highly conducive to providing optimal capture of a site's stormwater runoff while facilitating a reduction in flow velocity prior to discharging to the site outlet. The dry detention basin is outlined in **Section 5.0** and will continue to provide treatment of all stormwater generated on the property by means of infiltration, vegetative filtration, nutrient uptake and evapotranspiration.

Runoff generated on the building rooftops is considered to be ‘clean’ and free of contaminants. Since nearly a quarter of the site area consists of building rooftops, the contaminant load over the site will be much less than what the total impervious area on site suggests. This reduced loading will allow the proposed LIDs to be more effective at treating stormwater from a quality control perspective.

A storm sewer upstream of the dry detention basin is proposed to capture flows from the parking area and convey them to the basin. The gravel parking lot has been graded to allow runoff to flow into an inverted crown drive aisle which will convey runoff into the sewer. The storm sewer system will run west-east and will consist of a catchbasin and catchbasin manhole both located within the centre of the drive aisle. The storm sewer is sized to convey the 100-year peak flow, Refer to **Appendix B** for additional details.

Both the grass swale and dry detention basin proposed on-site will provide similar filtration and evapotranspiration benefits, as well as nutrient uptake and opportunity for infiltration into the in-situ soils. Based on the information provided in the LID Guide, the median pollutant mass removal rates of enhanced grass swales are considered to be 76% for total suspended solids, 55% for total phosphorus and 50% for total nitrogen based on available performance studies. A Nyloplast Envirohood and deep sump within each structure in the storm sewer will provide pre-treatment prior to discharging into the basin. The deep sumps will allow sediment to settle out of the captured stormwater while the Nyloplast Envirohood will prevent floating oils, trash, and debris from entering the sewer pipes.

Considering the above treatment train of a storm sewer, grass swale, and a dry detention basin, a minimum of 80% TSS removal efficiency is considered to be achievable on-site, as enhanced grass swales alone have been found to provide 76% TSS removal efficiency as per the LID guide.

Refer to **Drawing SSG** (Site Servicing and Grading Plan), **Drawing DS1** (Details Sheet) located in **Appendix A** as well as to the supporting calculations provided in **Appendix B** for additional details related to the stormwater management design.

6.2 Total Phosphorous Removal Initiatives

Phosphorus removal initiatives are also proposed for the subject site.

The various BMPs proposed for the site which will provide phosphorus loading reduction benefits are the grass swale and the dry detention basin. These stormwater management features will retain pollutants and nutrients, such as phosphorus, during minor rainfall events as they have been designed to accept the all of the site's runoff.

As noted in Section 4.4 of the LID Manual, any stormwater that is infiltrated or evaporated by LIDs prevents pollutants in the stormwater (such as phosphorus) from leaving the site. Moreover, the contaminated stormwater continues to be treated as it is infiltrated by the native soils. Both the grass swale and dry detention basin will provide opportunity for infiltration into the native soils, as well as filtration, nutrient uptake and evapotranspiration benefits.

6.3 Water Balance Initiatives

As noted in the Hydrogeological Study and Water Balance Analysis prepared by Ian D. Wilson & Associates Ltd. (May 24, 2019), the predominant underlying gravelly silty sand soils on-site are considered to have a high permeability (considering their infiltration rate - determined to be 30mm/hr based on the T-time of 20min/cm as stated in the Geotechnical Investigation Report prepared by Cambium Inc., dated April 1, 2019).

The Wilson report concludes that based on the conservative T-time of 20min/cm for the native gravelly silty sand soils, LID measures with a total site footprint of 35m^2 are required to meet the on-site water balance requirements.

The proposed dry detention basin provides a total base surface area of 51m^2. As a result, the dry detention basin alone exceeds the minimum requirements for water balance on-site as indicated above.

7.0 Sediment and Erosion Controls

In accordance with Town policy, effective erosion and sediment controls must be established prior to construction commencement and maintained until the site has been stabilized. Exposure of the soil during construction should be minimized to avoid erosion and sedimentation. The site's erosion potential may be mitigated through the use of sound erosion and sedimentation control measures. The following measures shall be carried out prior to construction and maintained until disturbed areas have regained a significant grass cover:

Topsoil Stripping: Topsoil stripping will be reduced as much as possible on-site. Where grading is necessary, the exposed soil will be stabilized by seeding immediately upon being set to grade. Should topsoil stockpiling be required, the stockpiles will be kept at manageable levels for grass/weed cutting purposes.

Silt Fence: Silt fence will be placed along the down slope of all excavated material and along the perimeter of the site to prevent sediment transport. Periodic inspections and repairs to the silt fence should be performed regularly, as well as after every rainfall event.

Mud Mat: Mud tracking from construction traffic must be controlled through the use of a mud-mat consisting of clear stone located at the site's construction entrances/ exits.

Vegetated Buffers: Existing grassland vegetation/wooded and lawn areas along the development limits are to be maintained wherever possible. These areas will provide a natural barrier to filter potentially sediment-laden overland flow before it is released from the site.

Conveyance Protection: Straw bale check dams will be placed within all swales immediately after being constructed and should be removed only after the area has been fully stabilized.

Finally, the Site Engineer will be responsible for completing routine inspections of the sediment and erosion control structures throughout the construction phase of the development, particularly after rainfall events. All damaged or clogged control devices or fencing must be repaired immediately.

8.0 Water Servicing

The proposed water servicing is detailed on the Site Servicing and Grading Plan **(SSG)** provided in **Appendix A**.

Based on the record information provided by the Town of Midland, there is an existing 250mmø watermain located in the west boulevard of Whitfield Crescent. There's also an existing water service located on Whitfield Crescent that will be used to service the proposed office space.

The size of the existing water service is unknown and will need to be verified during/prior to construction. It is assumed that the existing water service is 19mmø. Based on this assumption it has been confirmed that a 19mmø water service is more than adequate to accommodate the proposed single powder room within the office space. The water service will be complete with a shut-off valve located at the property line.

In terms of fire protection there's an existing fire hydrant in front of the site along Whitfield Crescent. An additional fire hydrant is proposed within the site between Building 'A' and Building 'B'.

The Town of Midland has provided fire hydrant flow data for both existing hydrants located on Whitfield Crescent. The Town records for the existing 250mmø watermain on Whitfield Crescent has a static pressure of 100psi with a flow of 94.7 L/s and a residual pressure of 78psi. It has been determined that a 150mmø fire service connected to the existing 250mmø watermain on Whitfield Crescent is adequate to provide fire protection within the proposed development.

9.0 Wastewater Servicing

There is an existing 200mmø sanitary sewer along the east boulevard of the Whitfield Crescent ROW as well as an existing 150mmø sanitary lateral at property line. It is proposed to connect to the existing 150mmø lateral to service the proposed office space as shown on the Site Servicing and Grading Plan (**SSG**) provided in **Appendix A**.

10.0 Utilities & Electrical Servicing

Existing bell, hydro, gas and cable services are all present at the property frontage based on visual inspection. The servicing drawings are in the process of being circulated to the utility agencies to confirm that existing services are adequate. Considering the area and the location of the existing developments within the Whitfield Crescent ROW, we do not anticipate any issues with utility servicing. The site plan has been provided to Walker's Electric 2000 to determine the electrical servicing requirements for the development.

11.0 Summary and Conclusions

This Site Servicing and Stormwater Management Report demonstrates how the proposed development can be integrated into the existing community, without imposing any adverse stormwater/servicing impacts. Specifically, we note the following:

- Stormwater quantity control will be provided via the use of an on-site dry detention basin sized to accommodate the storage volume required to attenuate post-development peak flows to corresponding pre-development target rates or less. The dry detention basin will include an outlet structure consisting of an orifice plate installed on the upstream end of the outlet pipe which will control runoff prior to being discharged off-site.
- Stormwater quality control will be provided via an integrated treatment train which will help minimize any negative impacts the proposed development may have on the existing quality of stormwater runoff. An 'Enhanced' Level of Protection, as defined in the MOE's Stormwater Management Planning & Design Manual, will be provided through the use of a dry detention basin complete with a grass swale, and storm sewer system located upstream. The storm sewer system will be equipped with a Nyloplast Envirohood and deep sump in all structures for pre-treatment purposes. This approach is premised on the stormwater being both filtrated as well as infiltrated into the in-situ soils while the vegetation will also provide nutrient uptake and evapotranspiration benefits which will all inherently provide additional water balance and phosphorus loading reduction benefits. Moreover, sedimentation/separation of contaminants (i.e. oils, floating debris, etc.) will occur within each storm sewer inlet structure.
- The use of silt fencing, existing vegetated buffers, straw bale check dams, and a construction mud mat will ensure downstream stormwater quality is maintained during construction.

- Site servicing will be provided via water (domestic and fire supply) and sanitary service connections to existing infrastructure within the adjacent Whitfield Crescent ROW. Similarly, utility and electrical servicing will be provided from the Whitfield Crescent ROW as well.

The site servicing and stormwater management design as described above, can be constructed and maintained as a functional method of servicing the site as well as treating all stormwater run-off generated by the proposed development. This Site Servicing and Stormwater Management Report and the associated engineering design drawings are based on information provided at the time of their preparation and are considered only applicable to the proposed works as described in this report. Any changes subsequent to the report and drawings date of issuance should be reviewed by WMI & Associates Ltd. to ensure applicability of the design contained within the documents.

Based on the above, we request that this report be received by the Town in support of detailed design and ultimately the construction of the proposed self-storage development.

Respectfully submitted,

WMI & Associates Limited

Benjamin Daniels, B.Eng.

Jeremy W. Lightheart, P.Eng.

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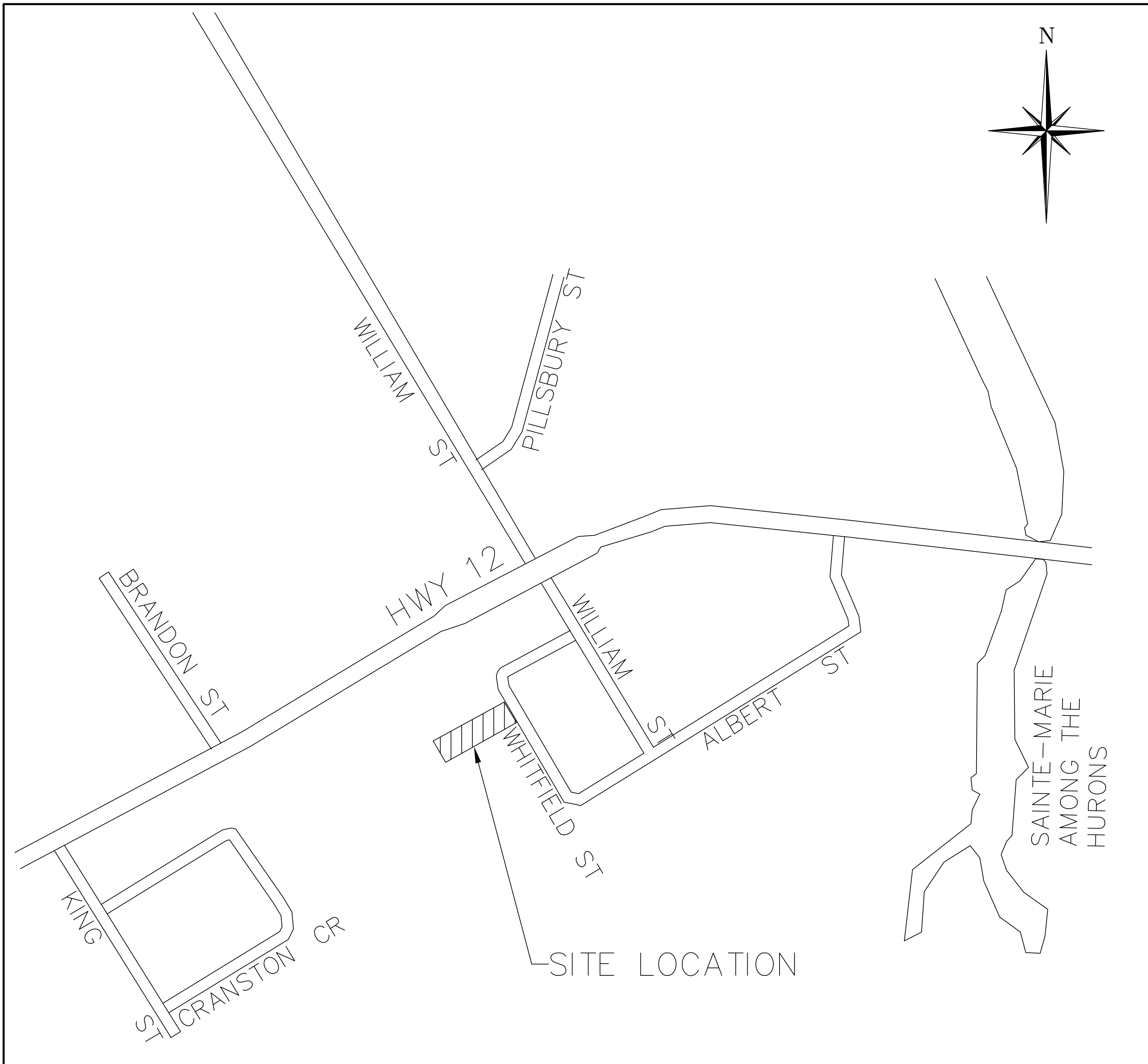
APPENDIX A

FIGURES



Drawing Title

SITE LOCATION PLAN

Project Title

265 WHITFIELD CRESCENT

WMI & Associates Limited
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Barrie, Ontario
L4M 1H5
705-797-2027
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Drawn By	Checked By	Figure No.
AW	RDW	FIG1
Scale: N.T.S.	Project No.: 19-543	

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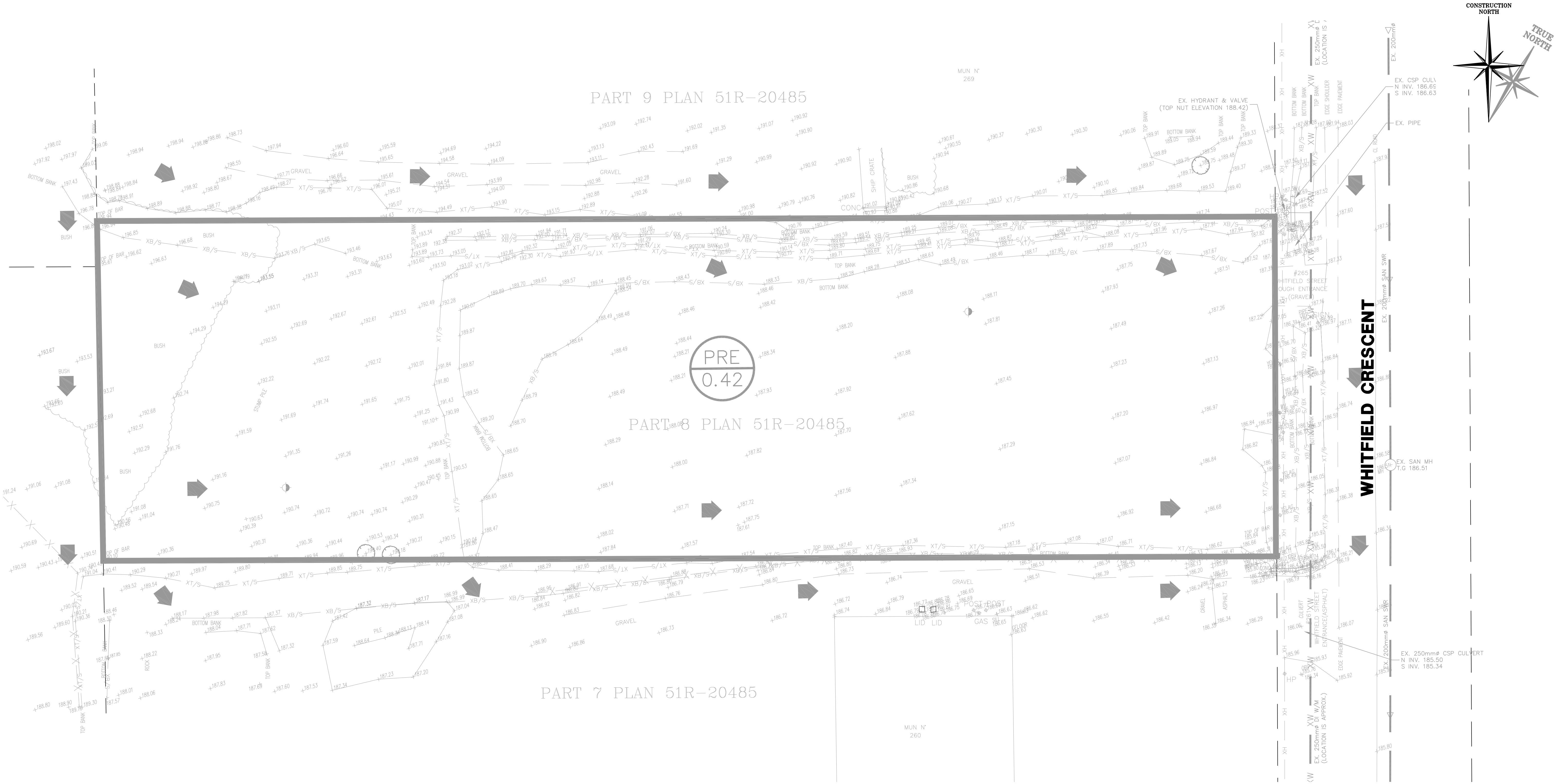

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	LIMITS OF CATCHMENT AREA
	OVERLAND FLOW DIRECTION

Drawing Title

PRE-DEVELOPMENT DRAINAGE PLAN

Project Title

265 WHITFIELD CRESCENT

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Drawn By	Checked By	Figure No.
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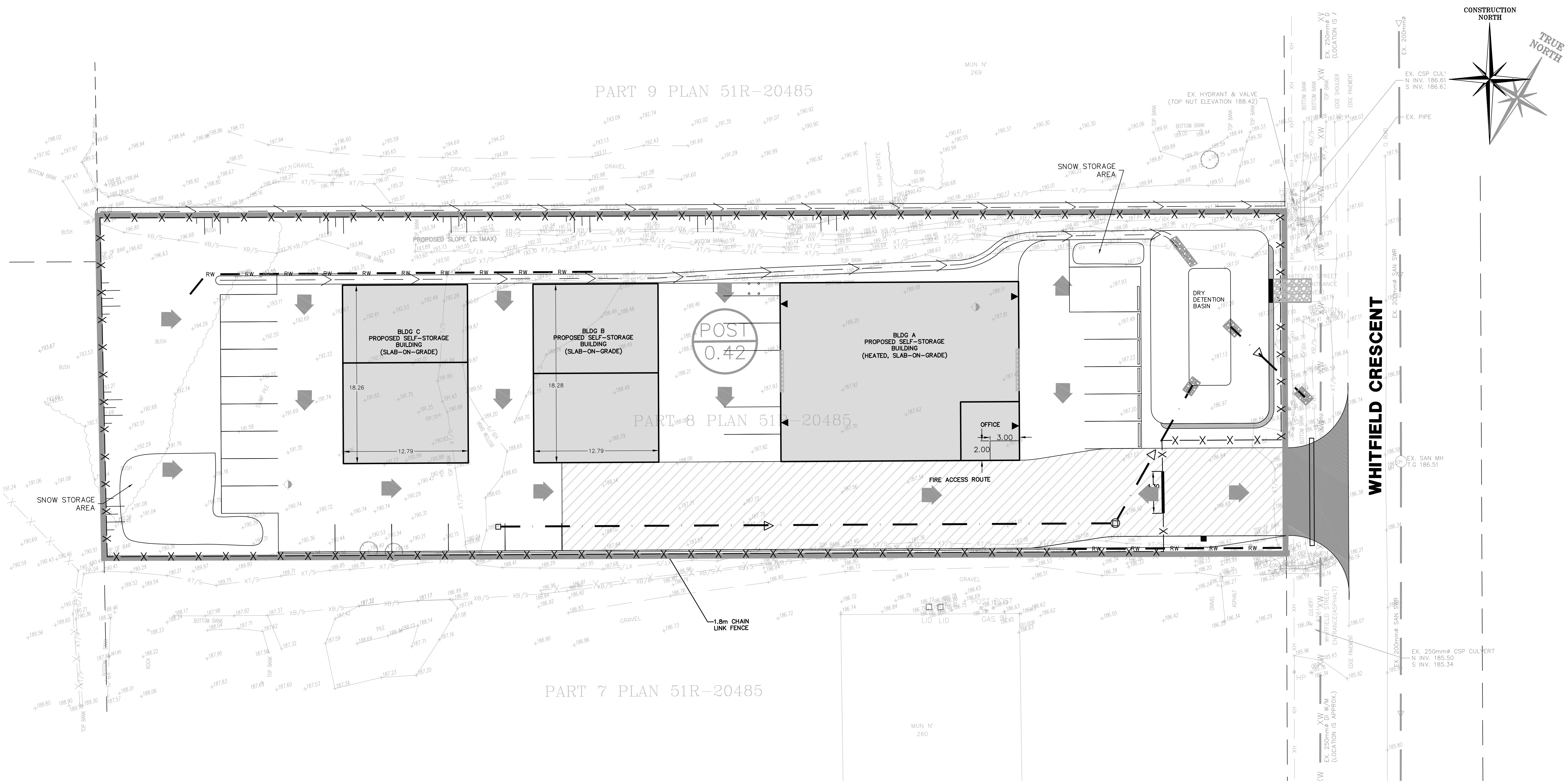

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CATCHMENT AREA (HA) (0.00)

LIMITS OF CATCHMENT AREA

OVERLAND FLOW DIRECTION

Drawing Title

POST-DEVELOPMENT DRAINAGE PLAN

Project Title

265 WHITFIELD CRESCENT

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Drawn By	Checked By	Figure No.
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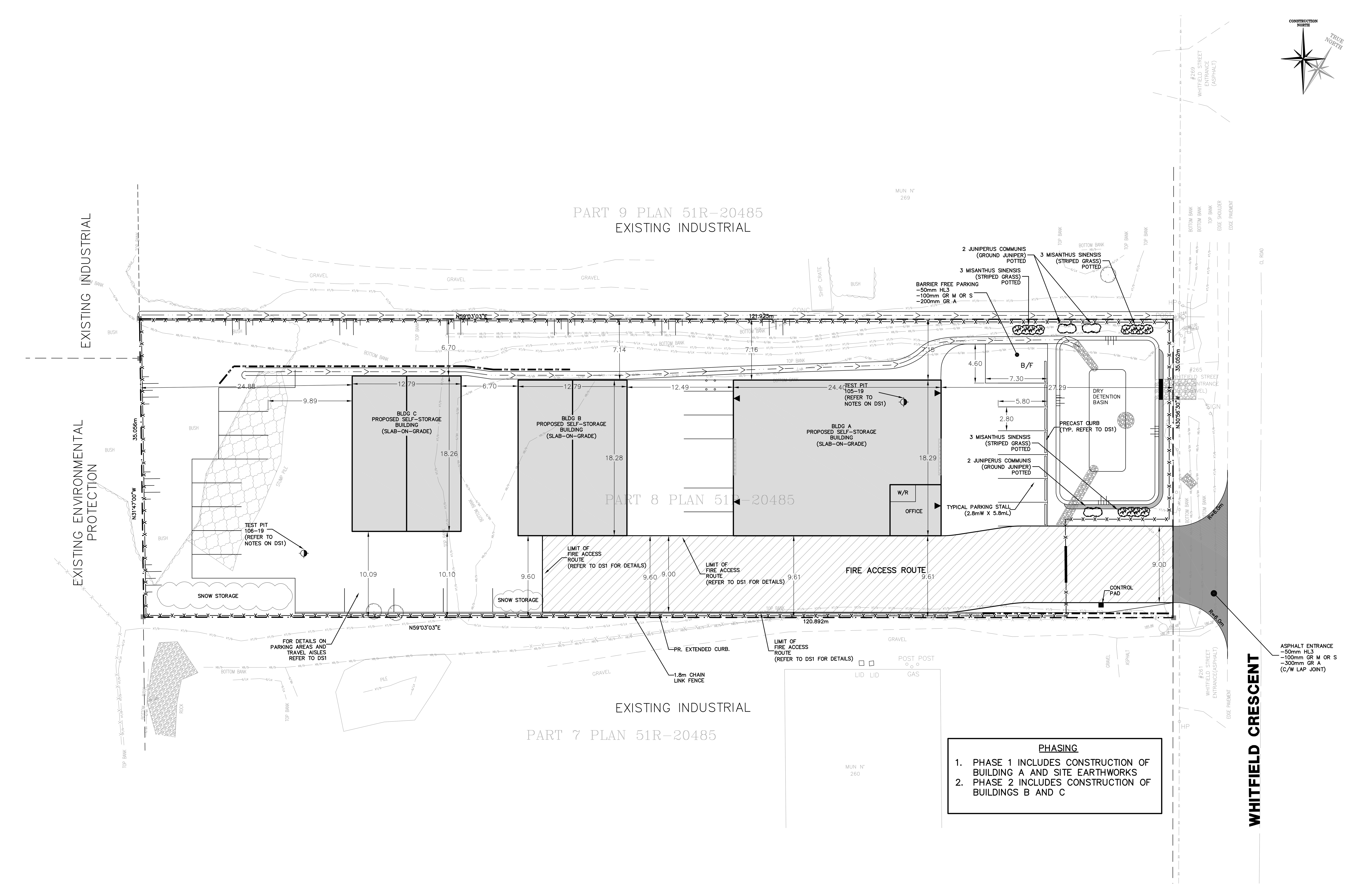

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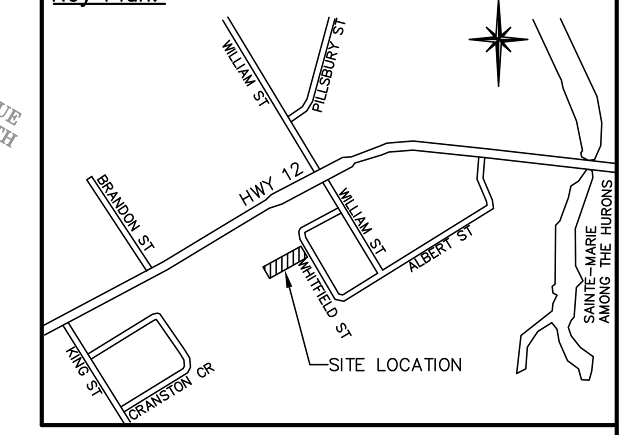

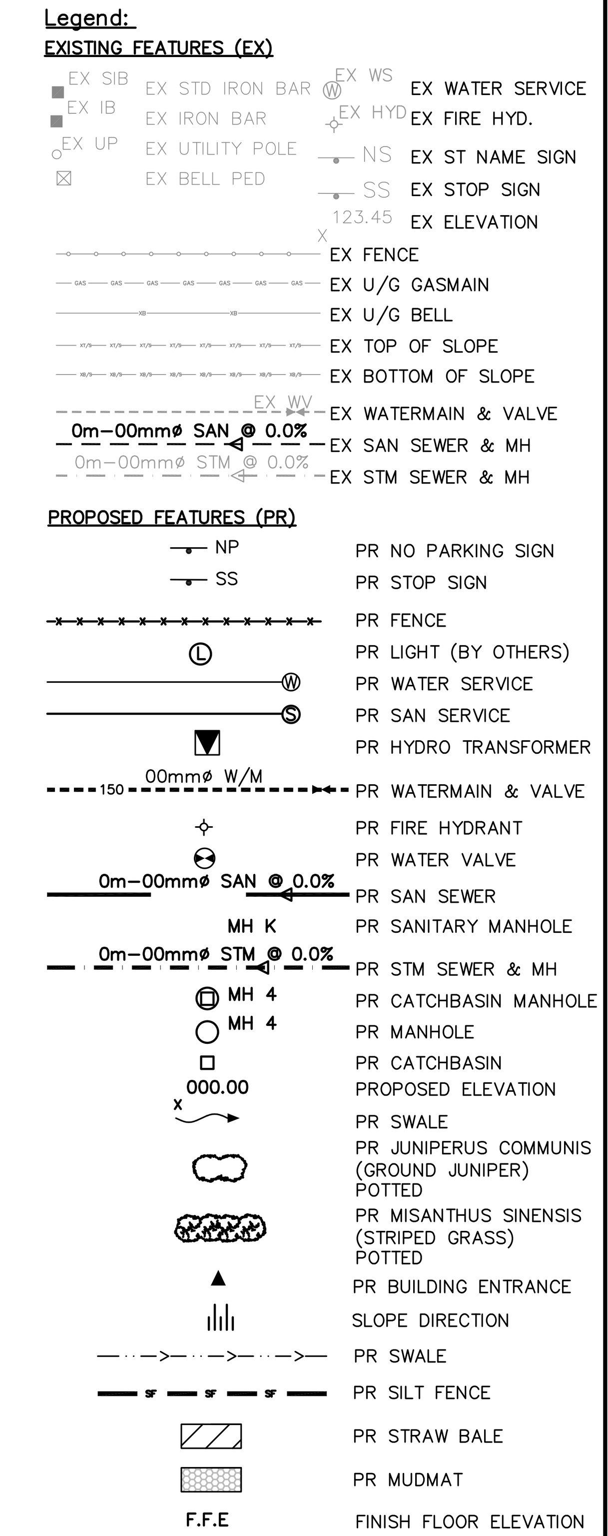

SITE STATISTICS

ZONING – (M1) INDUSTRIAL

	REQUIRED	PROVIDED
MINIMUM LOT AREA	0.4 ha	0.42ha
MINIMUM FRONTAGE	30.0 m	35.05m
MAXIMUM COVERAGE	60%	21.7%
MINIMUM SETBACKS		
FRONT	7.5m	27.8m
REAR	8.8m	21.7m
INTERNAL SIDE	6.0m	6.0m
BUILDING AREA		915sqm
PARKING	1 PER 40sqm (23)	22
B/F	1	1
TOTAL		23

GENERAL NOTES:

1. PROPERTY LINE INFORMATION IS TAKEN FROM SURVEY PLAN 51R-20485 DATED FEBRUARY 7, 1990 PREPARED BY HERB MELLISH, O.L.S.
2. TOPOGRAPHIC SURVEY INFORMATION IS TAKEN FROM DEMTECH SERVICES INC. DATED MARCH 2019.
3. TEST PIT INFORMATION TAKEN FROM GEOTECHNICAL INVESTIGATION REPORT PREPARED BY CAMBIUM INC., DATED APRIL 2019.
4. FOR DETAILS ON EXTERIOR LIGHTING REFER TO THE "LIGHTING CALCULATIONS STUDY" PREPARED BY IDEAL SUPPLY, DATED APRIL 16, 2019,

CAUTION
CONTRACTOR TO DETERMINE LOCATION OF EXISTING UTILITIES PRIOR TO CONSTRUCTION.

Notes:

1. Unless noted otherwise, the measurements and distances shown on this drawing are shown in meters.
2. Do not scale drawings.
3. It is the contractor's responsibility to verify all dimensions, levels and datums on site and report any discrepancies or omissions to WMI & Associates Ltd. prior to construction.
4. This drawing is to be read and understood in conjunction with all other relevant documents applicable to this project.
5. This drawing is the exclusive property of WMI & Associates Ltd. and the reproduction of any part of this document without prior written consent is strictly prohibited.

Benchmark: 202.945m

TOP NUT OF EXISTING FIRE HYDRANT LOCATED ON THE NORTH SIDE OF HIGHWAY No.12, APPROXIMATELY 200m WEST OF THE INTERSECTION OF WILLIAM STREET AND HIGHWAY No.12.

No.	Issue / Revision	Date
1	OWNER REVIEW	MAY 14, 2019
2	FIRST SUBMISSION	JULY 8, 2019

265 WHITFIELD CRESCENT

SITE PLAN

Client:
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Drawn By	Checked By	Drawing No.
AW	DW	SP
Scale 1:200	Project No. 19-543	

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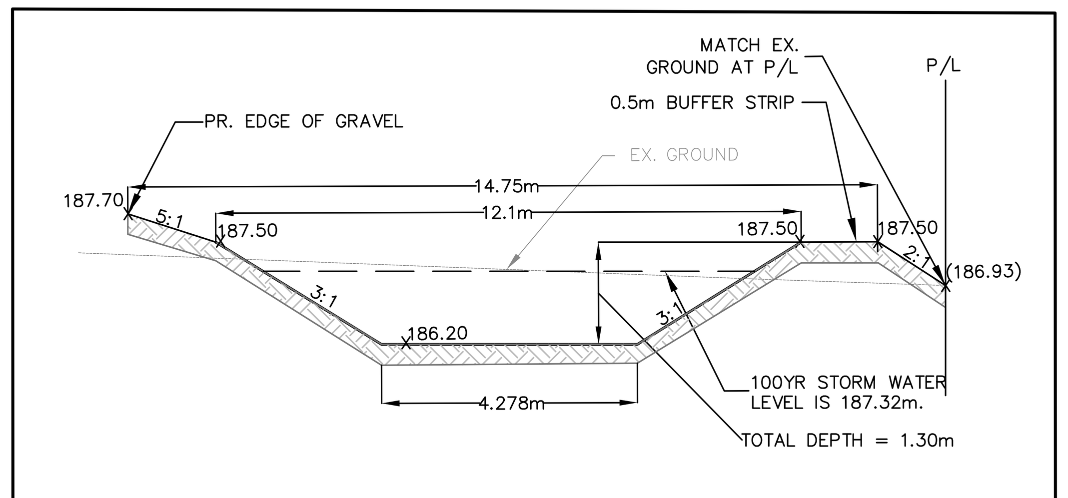
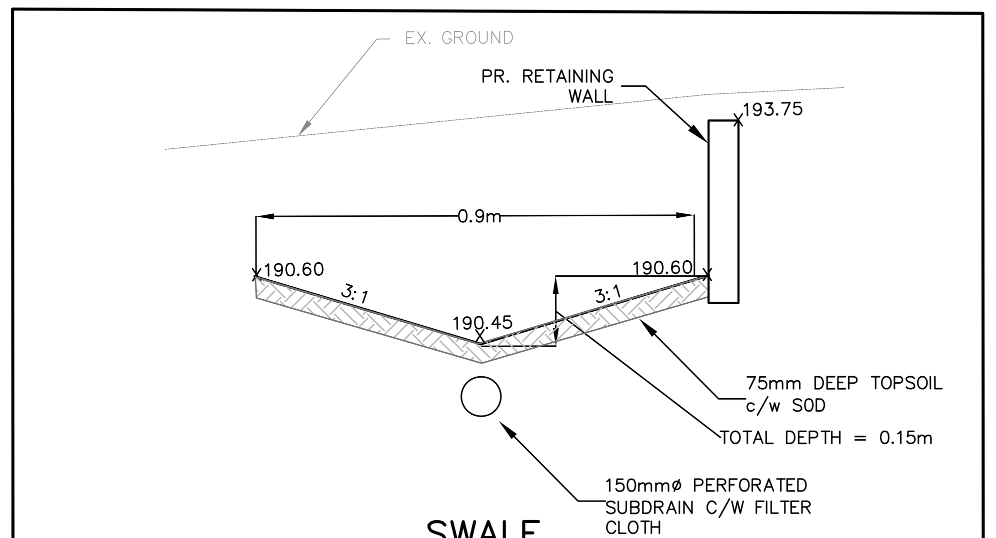


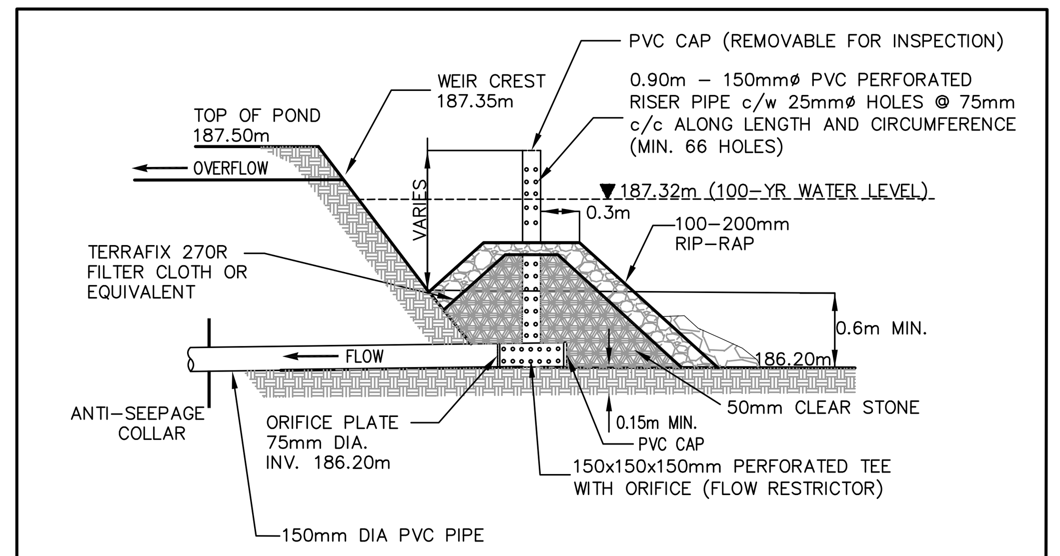
DRY DETENTION BASIN CROSS SECTION A–A

N.T.S.



SWALE CROSS SECTION B–B

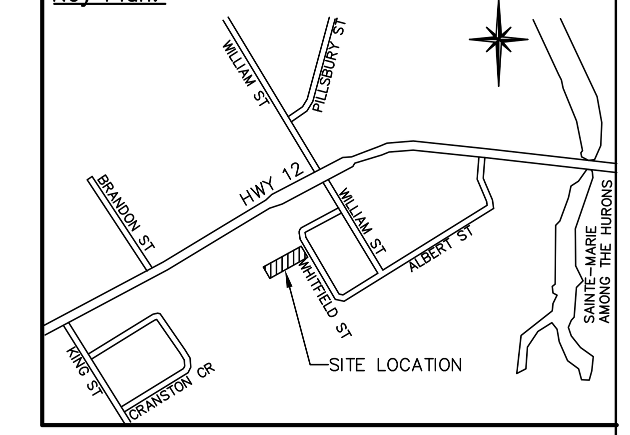
N.T.S.



DRY DETENTION BASIN OUTLET DETAIL

N.T.S.

Key Plan:



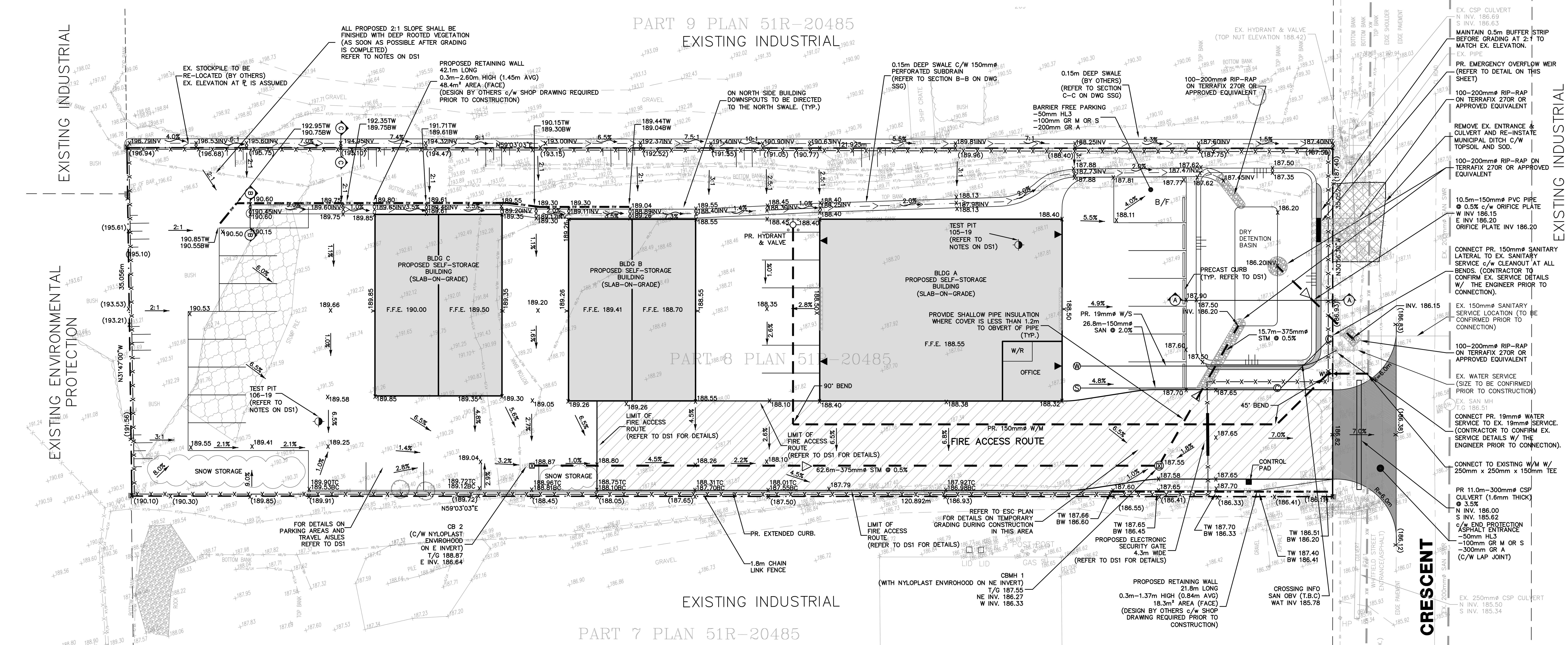
Legend:

EXISTING FEATURES (EX)

- EX SIB — EX STD IRON BAR
- EX IB — EX IRON BAR
- EX UP — EX UTILITY POLE
- EX BELL PED
- EX HYDRO TRANSFORMER
- EX WS — EX WATER SERVICE
- EX HYD — EX FIRE HYD.
- NS — EX ST NAME SIGN
- SS — EX STOP SIGN
- x 123.45 — EX ELEVATION
- EX FENCE
- EX U/G GASMAIN
- EX U/G BELL
- EX W — EX WATERMAIN & VALVE
- EX SAN @ 0.0% — EX SAN SEWER & MH
- EX STM @ 0.0% — EX STM SEWER & MH

PROPOSED FEATURES (PR)

- NS — PR STREET NAME SIGN
- SS — PR STOP SIGN
- PR SILT FENCE
- PR STREET LIGHT
- PR WATER SERVICE
- PR SAN SERVICE
- PR STM SERVICE
- PR HYDRO TRANSFORMER
- 00mmØ W/M — PR WATERMAIN
- PR FIRE HYDRANT
- PR WATER VALVE
- 0m–00mmØ SAN @ 0.0% — PR SAN SEWER
- MH K — PR SANITARY MANHOLE
- 0m–00mmØ STM @ 0.0% — PR STM SEWER
- CBMH 4 — PR CATCHBASIN MANHOLE
- MH 4 — PR MANHOLE
- CB 4 — PR CATCHBASIN
- 000.00 — PROPOSED ELEVATION
- (000.00) — PR. MATCH EXISTING ELEVATION (INTERPOLATED)
- PR JUNIPERUS COMMUNIS (GROUND JUNIPER) POTTED
- PR MISANTHUS SINENSIS (STRIPED GRASS) POTTED
- PR BUILDING ENTRANCE
- SLOPE DIRECTION
- PR SWALE
- PR SILT FENCE
- PR STRAW BALE
- PR MUDMAT
- F.F.E — FINISH FLOOR ELEVATION
- PR RETAINING WALL
- PR FIRE ACCESS ROUTE



PHASING

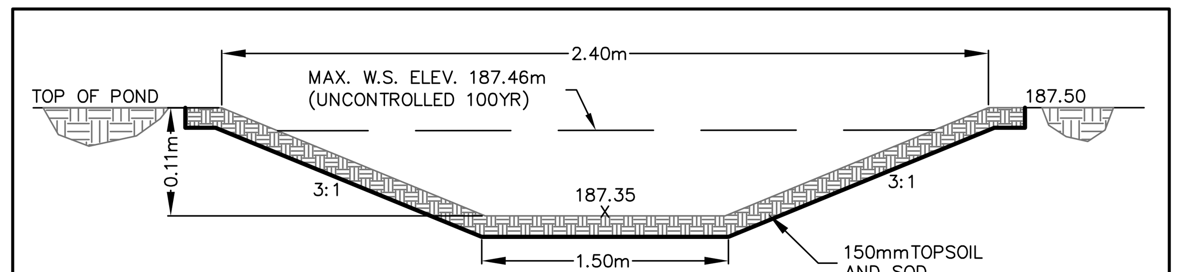
1. PHASE 1 INCLUDES CONSTRUCTION OF BUILDING A AND SITE EARTHWORKS
2. PHASE 2 INCLUDES CONSTRUCTION OF BUILDINGS B AND C

GENERAL NOTES:

1. PROPERTY LINE INFORMATION IS TAKEN FROM SURVEY PLAN 51R–20485 DATED FEB. 7, 1990 PREPARED BY HERB MELLISH, O.L.S.
2. TOPOGRAPHIC SURVEY INFORMATION IS TAKEN FROM DEMTECH SERVICES INC. DATED MARCH 2019.
3. TEST PIT INFORMATION TAKEN FROM GEOTECHNICAL INVESTIGATION REPORT PREPARED BY CAMBIUM INC., DATED APRIL 2019.
4. FOR DETAILS ON EXTERIOR LIGHTING REFER TO THE "LIGHTING CALCULATIONS STUDY" PREPARED BY IDEAL SUPPLY, DATED APRIL 16, 2019.

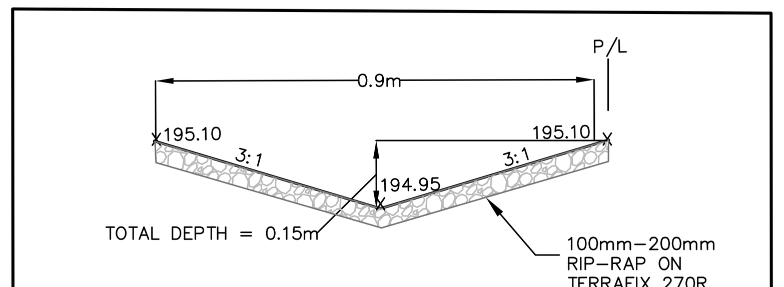
CAUTION

CONTRACTOR TO DETERMINE LOCATION OF EXISTING UTILITIES PRIOR TO CONSTRUCTION.



EMERGENCY OVERFLOW WEIR CROSS SECTION

N.T.S.



LOT LINE SWALE CROSS SECTION C–C

N.T.S.

FEATURE	ELEVATION	VOLUME
BASE OF DRY DETENTION BASIN (ACTIVE STORAGE VOLUMES)	186.20m	0m³
2 YR HWL	186.68m	37.20m³
5 YR HWL	186.83m	54.38m³
10 YR HWL	186.91m	66.02m³
25 YR HWL	187.09m	92.67m³
50 YR HWL	187.22m	119.09m³
100 YR HWL	187.32m	142.02m³
TOP OF BASIN	187.50m	187.20m³

Notes:

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Benchmark: 202.945m

TOP NUT OF EXISTING FIRE HYDRANT LOCATED ON THE NORTH SIDE OF HIGHWAY No.12, APPROXIMATELY 200m WEST OF THE INTERSECTION OF WILLIAM STREET AND HIGHWAY No.12.

REGISTERED PROFESSIONAL ENGINEER — S. R. MORASH — July 8, 2019 — PROVINCE OF ONTARIO

No.	Issue / Revision	Date
1	INTERNAL REVIEW	JUNE 21, 2019
2	FIRST SUBMISSION	JULY 8, 2019

265 WHITFIELD CRESCENT

SITE SERVICING & GRADING PLAN

Client:

Jason Redman
699 Aberdeen Blvd. Unit 902,
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L4M 1H2

WMI & Associates Limited
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Drawn By	Checked By	Drawing No.
AW	DW	SSG
Scale 1:200	Project No. 19–543	

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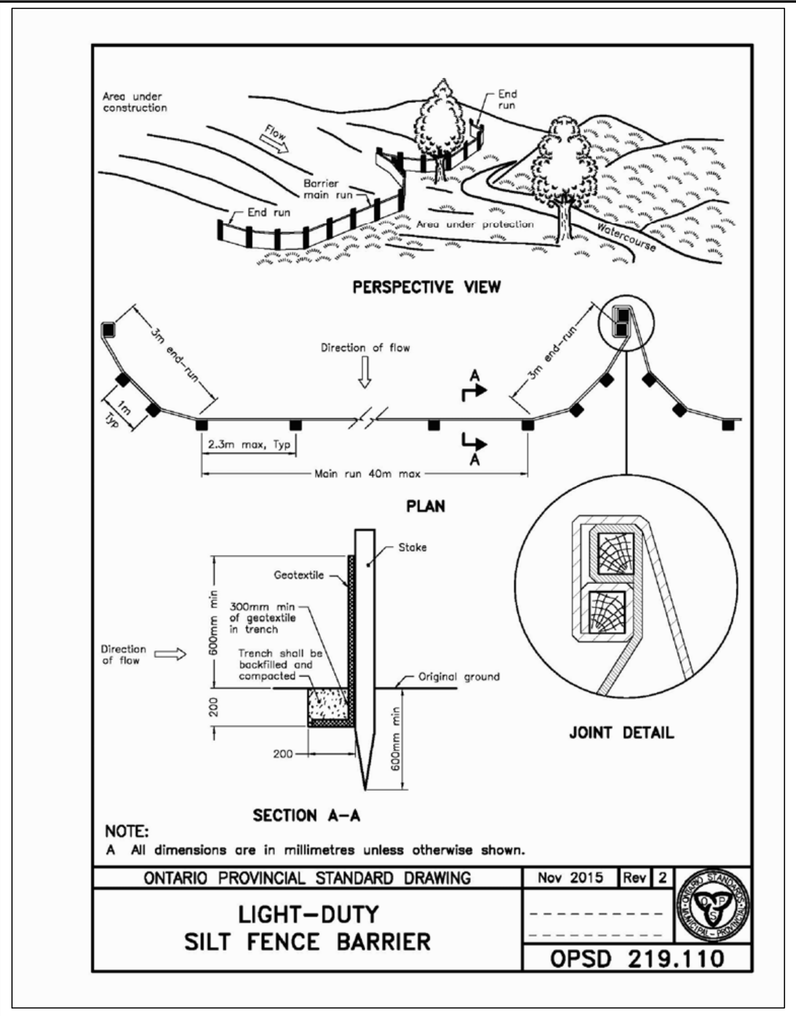
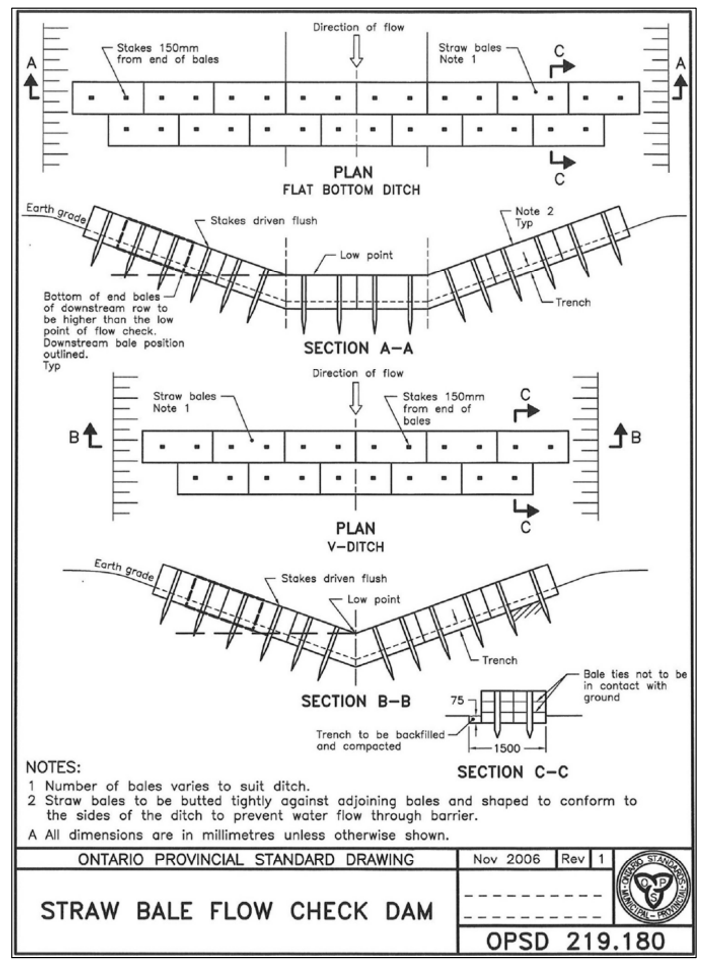
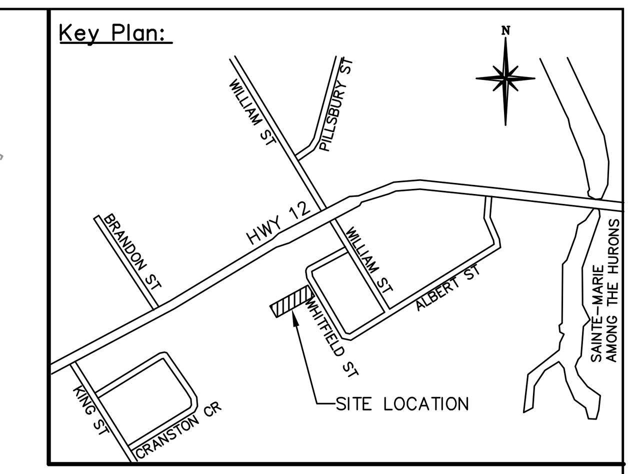
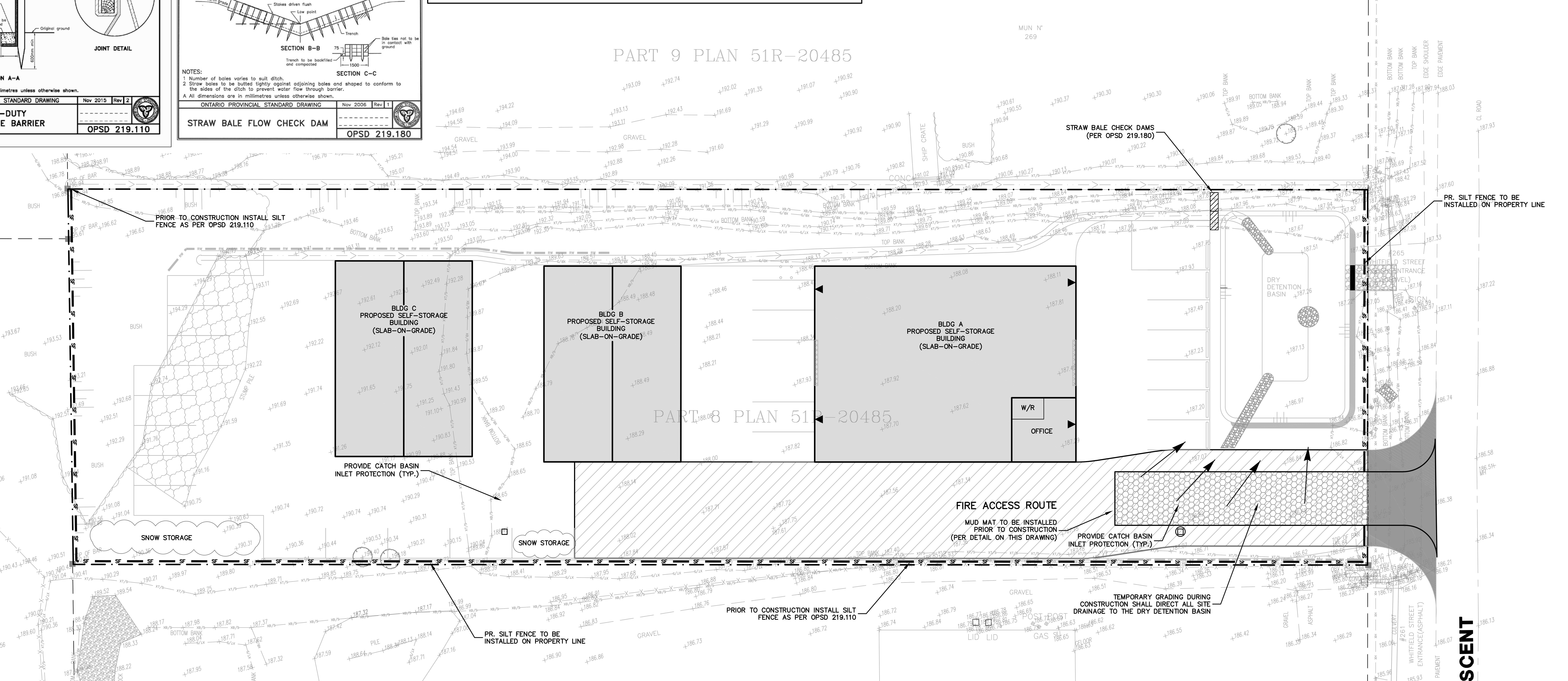




GENERAL NOTES:

1. EROSION CONTROL WORKS TO BE INSPECTED REGULARLY AFTER EVERY RAINFALL AND REPAIRED/REPLACED AS REQUIRED BY THE ENGINEER.
2. ALL DISTURBED AREAS TO BE RESTORED USING TOPSOIL AND SEED IMMEDIATELY UPON ESTABLISHING FINAL ELEVATIONS.
3. ALL AREAS WHICH REMAIN UNDISTURBED FOR MORE THAN 30 DAYS SHALL BE STABILIZED.
4. EROSION CONTROL WORKS TO BE MAINTAINED UNTIL THE SITE HAS STABILIZED AND REMOVAL IS DIRECTED BY THE ENGINEER.
5. SILT FENCE TO BE MAINTAINED ON THE DOWNSTREAM SIDE OF ALL STOCK PILES.
6. SILT FENCE IS TO BE CONSTRUCTED/INSTALLED AROUND PERIMETER OF THE SITE AT THE START OF CONSTRUCTION.
7. THE DEVELOPER AND DEVELOPER'S ENGINEER ARE RESPONSIBLE FOR COMPLETING ROUTINE INSPECTIONS OF THE SEDIMENT AND EROSION CONTROL STRUCTURES DURING THE CONSTRUCTION PHASE.
8. THE CONTRACTOR SHALL PROVIDE A MUD MAT AS PER THE DETAIL ON THIS DRAWING FOR EACH CONSTRUCTION ENTRANCE. THE TENDERED PRICE SHALL INCLUDE THE COST FOR ALL MUD MATS UTILIZED DURING CONSTRUCTION.
9. ALL MUD MATS SHALL BE MAINTAINED DURING THE CONSTRUCTION PERIOD AND REPLACED AS DIRECTED BY THE ENGINEER.





Legend:

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- EX SIB — EX STD IRON BAR
- EX IB — EX IRON BAR
- EX UP — EX UTILITY POLE
- EX BELL PED
- EX HYDRO TRANSFORMER
- EX WATER SERVICE
- EX FIRE HYD.
- EX ST NAME SIGN
- EX STOP SIGN
- EX ELEVATION
- EX FENCE
- EX U/G GASMAIN
- EX U/G BELL
- EX WATERMAIN & VALVE
- EX SAN SEWER & MH
- EX STM SEWER & MH

PROPOSED FEATURES (PR)

- PR STREET NAME SIGN
- PR STOP SIGN
- PR SILT FENCE
- PR STREET LIGHT
- PR WATER SERVICE
- PR SAN SERVICE
- PR STM SERVICE
- PR HYDRO TRANSFORMER
- PR WATERMAIN
- PR FIRE HYDRANT
- PR WATER VALVE
- PR SAN SEWER
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CAUTION
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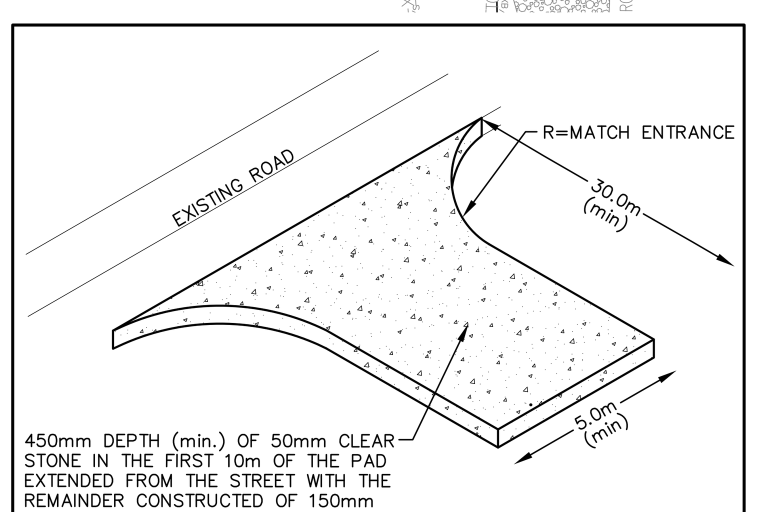
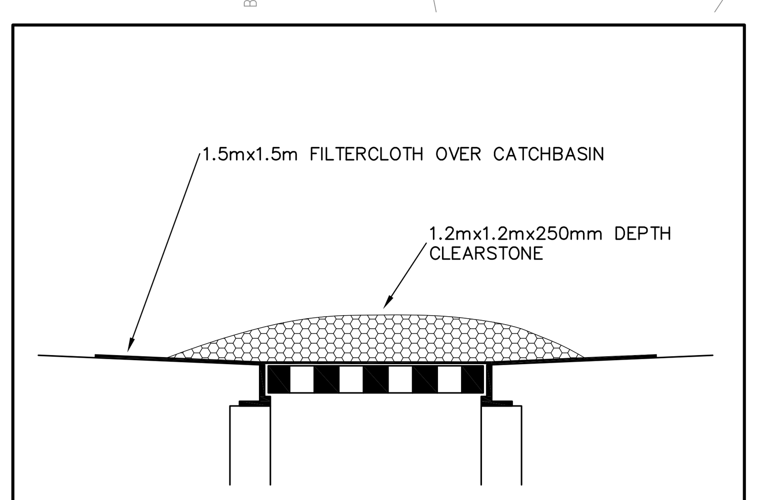


MUD MAT DETAIL
(CONSTRUCTION ACCESS)
N.T.S.



CATCHBASIN INLET PROTECTION
N.T.S.

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REGISTERED PROFESSIONAL ENGINEER
S. R. MORASH
July 8, 2019
PROVINCE OF ONTARIO

No.	Issue / Revision	Date
1	INTERNAL REVIEW	JUNE 21, 2019
2	FIRST SUBMISSION	JULY 8, 2019

265 WHITFIELD CRESCENT

EROSION & SEDIMENT CONTROL PLAN

Client:
Jason Redman
699 Aberdeen Blvd. Unit 902,
Midland ON,
L4M 1H2

WMI & Associates Limited
119 Collier Street
Barrie, Ontario
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Drawn By	Checked By	Drawing No.
AW	DW	ESC
Scale 1:200	Project No. 19-543	

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GENERAL

1. ALL MEASUREMENTS ARE IN METRES, PIPE SIZES IN MILLIMETERS, UNLESS OTHERWISE NOTED.
2. THE ONTARIO PROVINCIAL STANDARDS AND SPECIFICATIONS AND THE ONTARIO PROVINCIAL STANDARD DRAWINGS, MIDLAND PUC AND TOWN OF MIDLAND DEVELOPMENT DESIGN STANDARDS SHALL APPLY TO THIS CONTRACT.
3. ORDER OF PRECEDENCE OF STANDARD DRAWINGS IS FIRSTLY TOWN OF MIDLAND DEVELOPMENT DESIGN STANDARDS, SECONDLY MIDLAND PUC STANDARDS, AND THIRDLY ONTARIO PROVINCIAL STANDARD DRAWINGS (OPSD).
4. LOCATIONS OF EXISTING SERVICES ARE NOT GUARANTEED. THE CONTRACTOR SHALL CONFIRM EXISTING UTILITY LOCATIONS AND ELEVATIONS PRIOR TO CONSTRUCTION. THE CONTRACTOR IS REQUIRED TO NOTIFY THE VARIOUS UTILITY COMPANIES 48 HOURS PRIOR TO THE COMMENCEMENT OF ANY WORK.
5. A ROAD OCCUPANCY PERMIT IS REQUIRED FROM THE TOWN OF MIDLAND PRIOR TO THE COMMENCEMENT OF WORK WITHIN ANY TOWN RIGHT-OF-WAY.
6. NATIVE MATERIAL SUITABLE FOR BACKFILL SHALL BE COMPACTED TO 98% STANDARD PROCTOR MAXIMUM DRY DENSITY, UNLESS OTHERWISE NOTED. ENGINEERED FILL SHALL BE COMPACTED TO 100% STANDARD PROCTOR MAXIMUM DRY DENSITY.
7. GRANULAR MATERIAL AND BEDDING MATERIAL SHALL BE PLACED IN LAYERS 150mm IN DEPTH AND COMPACTED TO 100% STANDARD PROCTOR MAXIMUM DRY DENSITY OR AS DIRECTED BY THE SOILS CONSULTANT.
8. ALL DISTURBED AREAS WITHIN EXISTING MUNICIPAL RIGHT-OF-WAYS ARE TO BE REINSTATED TO THEIR ORIGINAL CONDITION OR BETTER AS DETERMINED BY THE TOWN OF MIDLAND (MIN 150mm TOPSOIL AND SOD).
9. ALL SILT CONTROL AND EROSION PROTECTION DEVICES ARE TO BE IN PLACE PRIOR TO THE COMMENCEMENT OF CONSTRUCTION AND SHALL REMAIN IN PLACE AND BE MAINTAINED BY THE CONTRACTOR UNTIL CONSTRUCTION IS COMPLETE, THE GRASS HAS ESTABLISHED GROWTH AND APPROVED BY THE ENGINEER.
10. UTILITY CROSSING, WHERE REQUIRED, SHALL BE SUPPORTED AS PER OPSD 1007.01 AND ANY EXISTING STRUCTURES SHALL BE PROPERLY SUPPORTED.
11. THE CONTRACTOR SHALL COORDINATE HIS WORK SUCH THAT HE DOES NOT INTERFERE WITH WORK BEING UNDERTAKEN BY A UTILITY COMPANY.
13. RE-INSTATEMENT OF ALL ROAD CUTS SHALL BE AS PER THE TOWN OF MIDLAND ENGINEERING STANDARDS. ROAD STRUCTURE TO BE AS FOLLOWS OR AS DIRECTED BY THE TOWN OF MIDLAND:
    - 40mm HL3
    - 60mm HL8
    - 150mm GRANULAR A
    - 450mm GRANULAR B
14. FOR DETAILS ON THE PROPOSED 2:1 SLOPES REFER TO THE SLOPE STABILITY INSPECTION LETTER PREPARED BY CAMBIUM (DATED JUNE 10, 2019)

SANITARY SERVICING

1. SANITARY SERVICE SHALL BE 150mmø PVC (SDR 28). IN ACCORDANCE WITH CSA AND AS PER 1006.010.
2. SEWERS SHALL BE CONSTRUCTED WITH BEDDING AS PER OPSD-802.010, (GRAN. 'A' EMBEDMENT MATERIAL) FOR FLEXIBLE PIPES UNLESS OTHERWISE ADVISED BY A GEOTECHNICAL ENGINEER.
3. SANITARY CLEANOUTS ARE TO BE AS PER OBC REQUIREMENTS.

WATER SERVICING

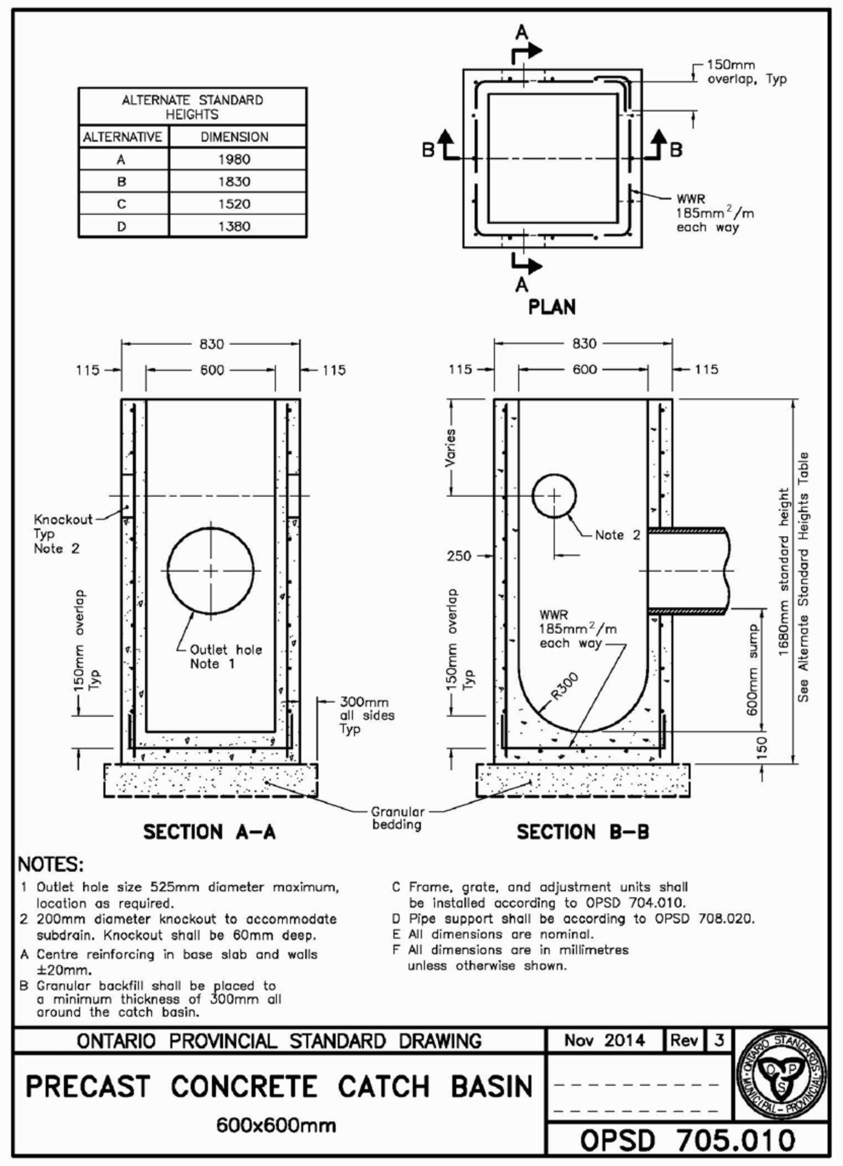
1. CONTRACTOR SHALL INFORM THE TOWN OF MIDLAND A MINIMUM OF 48 HOURS IN ADVANCE OF THEIR INTENTIONS TO WORK AND SHALL ONLY COMPLETE CONNECTIONS TO THE EXISTING WATERMAIN WHILE A TOWN OF MIDLAND WATER OPERATOR IS PRESENT.
2. WATER SERVICE LATERAL SHALL BE 19mmø MUNICIPEX.
3. HYDRANT WATERMAINS SHALL BE 150mmø PVC. (CLASS 150, DR-18) IN ACCORDANCE WITH AWWA C900 SPECIFICATION.
4. MECHANICAL JOINT FITTINGS MEETING AWWA SPECIFICATIONS C-907 AND CSA B.1.3.7.2 SHALL BE USED WHERE APPLICABLE. SHOULD DUCTILE IRON MECHANICAL JOINT FITTINGS BE EMPLOYED, THE CONTRACTOR SHALL INSTALL SACRIFICIAL CAPS ON EVERY BOLT. PVC JOINTS USING MECHANICAL JOINT FITTINGS ARE ARE TO BE SQUARE CUT, NOT BEVELLED.
5. WATERMAIN BEDDING SHALL CONFORM TO OPSD 802.010 (GRANULAR 'A' EMBEDMENT) FOR FLEXIBLE PIPE UNLESS OTHERWISE APPROVED BY THE TOWN OF MIDLAND.
6. ALL MATERIALS TO BE IN ACCORDANCE WITH THE TOWN OF MIDLAND WATER AND WASTEWATER APPROVED WATER MATERIALS LIST.
7. MINIMUM DEPTH OF COVER OVER WATER SERVICE TO BE 1.8 METRES.
8. MATERIAL SPECIFICATIONS ARE AS FOLLOWS:
    - SADDLES: SMITH-BLAIR 313 (OR APPROVED EQUIVALENT)
    - MAIN STOP: CAMBRIDGE BRASS 301NL-A7H7
    - CURB STOP: MUELLER CANADA B-25222N OR B-25218N
    - SERVICE BOXES: MUELLER CANADA A726/ A728 A800
    - FIRE HYDRANT: CANADA VALVE CENTURY/PREMIERE MODEL, AWWA 502
    - VALVES: MUELLER RESILIENT SEAT AWWA C509
    - VALVE BOXES: BIBBY
9. ALL TESTING AND COMMISSIONING TO BE AS PER THE TOWN OF MIDLAND ENGINEERING DEVELOPMENT DESIGN STANDARDS.

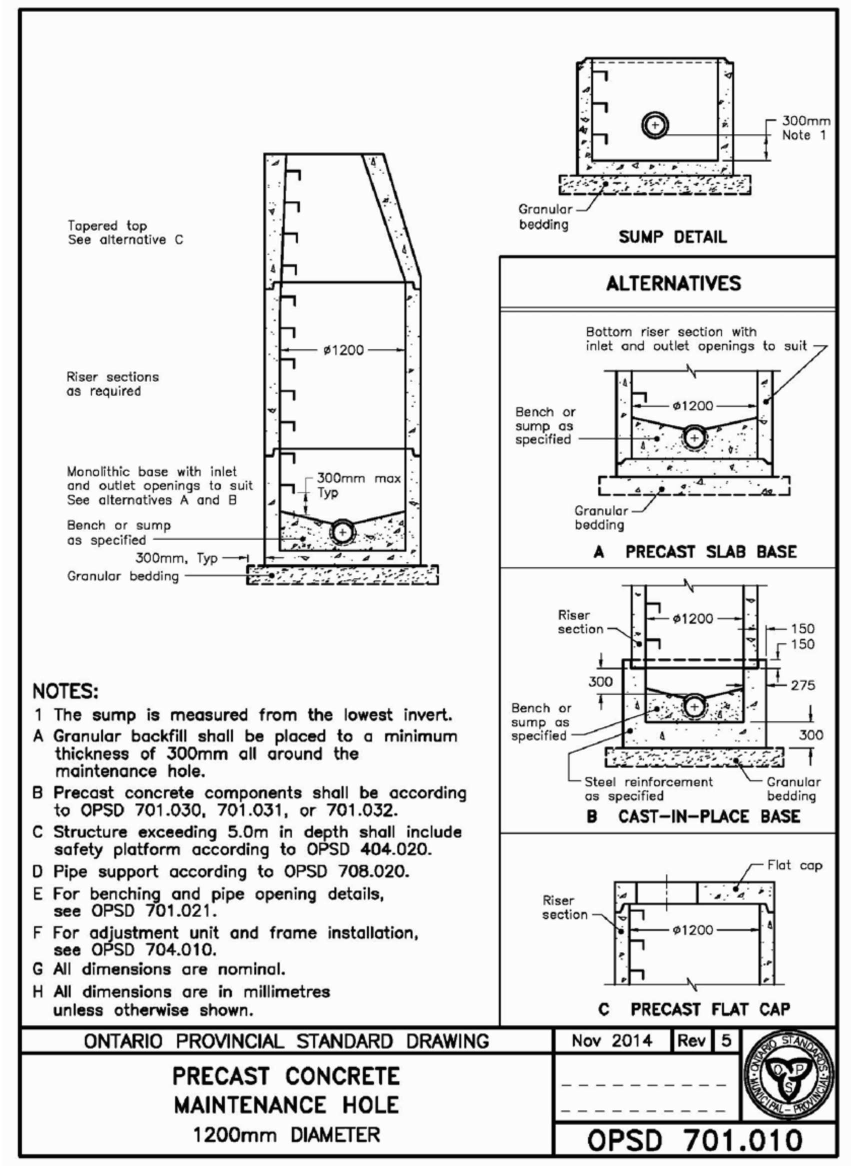
STORM SEWER:

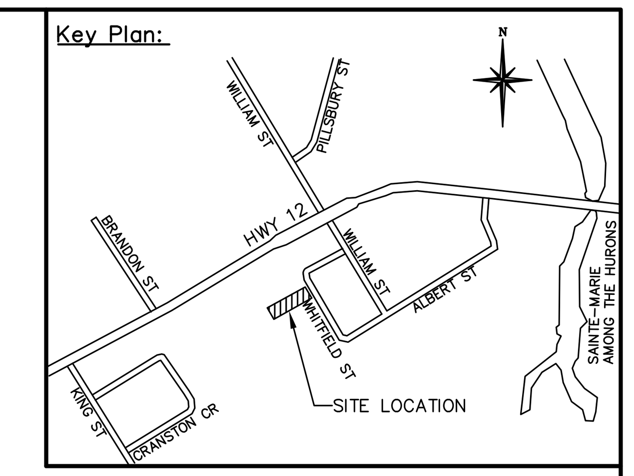
1. ALL SITE DRAINAGE POSSIBLE, INCLUDING ALL ROOF AND ASPHALT DRAINAGE, IS TO BE DIRECTED TO THE STORMWATER MANAGEMENT SYSTEM.
2. PIPE MATERIAL TO BE REINFORCED CONCRETE WITH A MINIMUM STRENGTH OF 50 N/m/mm CERTIFIED TO C.S.A. STANDARD A257.2 CLASS 50-D OR PVC CERTIFIED TO C.S.A. STANDARDS 182.2 AND 182.4.
3. STORM SEWER TO BE MINIMUM 300mm DIAMETER WITH JOINTS CONFORMING TO C.S.A. STANDARD A257.3.
4. MODULAR ADJUSTMENT UNITS FOR MANHOLES TO BE PROVIDED IN ACCORDANCE WITH OPSD 704.010. MAXIMUM THICKNESS OF ADJUSTMENTS UNITS IS 300mm.
5. STORM SEWER BEDDING AS PER OPSD 802.010 (FLEXIBLE PIPE) OR 802.030 (RIGID PIPE).
6. MANHOLES AND CATCHBASINS ARE TO BE IN ACCORDANCE WITH OPSD STANDARDS C/W 0.3m SUMPS.
7. STORM SEWER COVER LESS THAN 1.2m TO PIPE OBVERT WILL REQUIRE FROST PROTECTION (INSULATION, SEE DETAIL ON DRAWING DS1).
8. ALL STORM MANHOLES SHALL BE COMPLETE WITH FROST STRAPS AS PER OPSD 701.100.

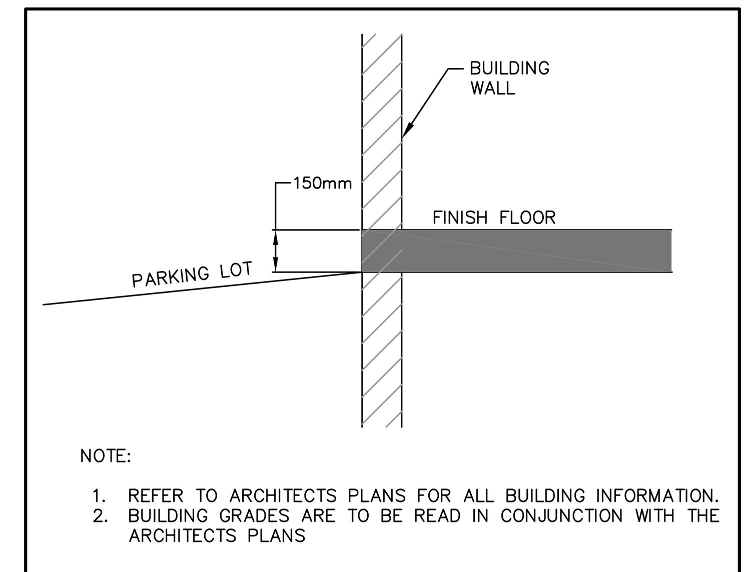
ACCESS ROUTES / PARKING LOT

1. ALL PARKING LOT CONSTRUCTION SHALL CONFORM TO THE TOWN OF MIDLAND ENGINEERING STANDARDS AND THE RECOMMENDATIONS OF THE GEOTECHNICAL REPORT PREPARED BY CAMBIUM INC. (DATED APRIL 1, 2019).
2. SUBGRADE TO BE COMPACTED TO A MINIMUM DRY DENSITY OF 98% OF THE MATERIAL'S STANDARD PROCTOR MAXIMUM DRY DENSITY (SPMDD).
3. GRANULAR 'A' BASE TO BE COMPACTED TO 100% OF MATERIAL'S STANDARD PROCTOR MAXIMUM DRY DENSITY (SPMDD).
4. PARKING LOT CONSTRUCTION TO BE COMPLETED UNDER THE SUPERVISION OF A GEOTECHNICAL ENGINEER.
5. THE ENTRANCE CULVERT SHALL BE INSTALLED WITH A HEADWALL AT BOTH ENDS. THE HEADWALL SHALL BE MASONRY LAID INTERLOCKING PRECAST UNITS AND SECURED IN PLACE. THE HEADWALL SHALL BE AS PER GARDENIA STEP TREAD BY OAKS CONCRETE PRODUCTS (OR APPROVED EQUIVALENT).
6. ALL ENTRANCE CULVERTS SHALL BE INSTALLED WITH A MINIMUM OF 300mm COVER.
7. PRE-CAST CURB SHALL BE AS PER OPSD 603.020.
8. THE ENTRANCE GATE SHALL BE 1.8m HIGH CHAIN LINK FENCE 4.3m WIDE, EQUIPPED WITH ELECTRONIC POWER CONTROLS.



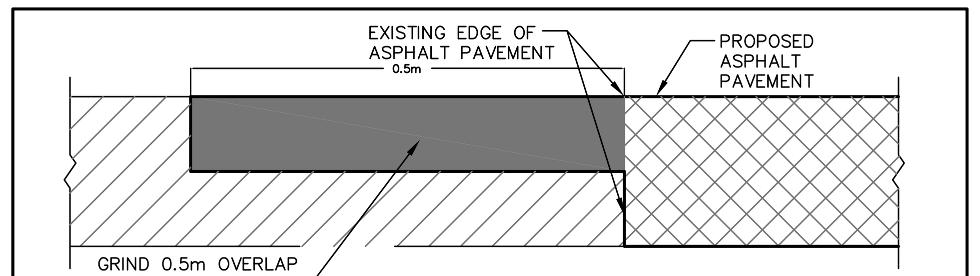






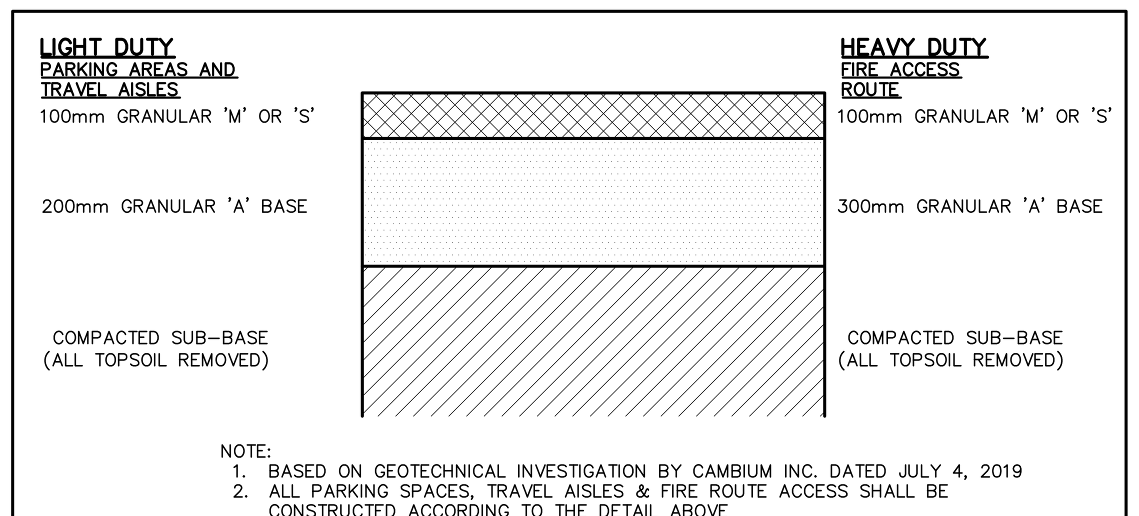
BUILDING WALL CROSS-SECTION

N.T.S.



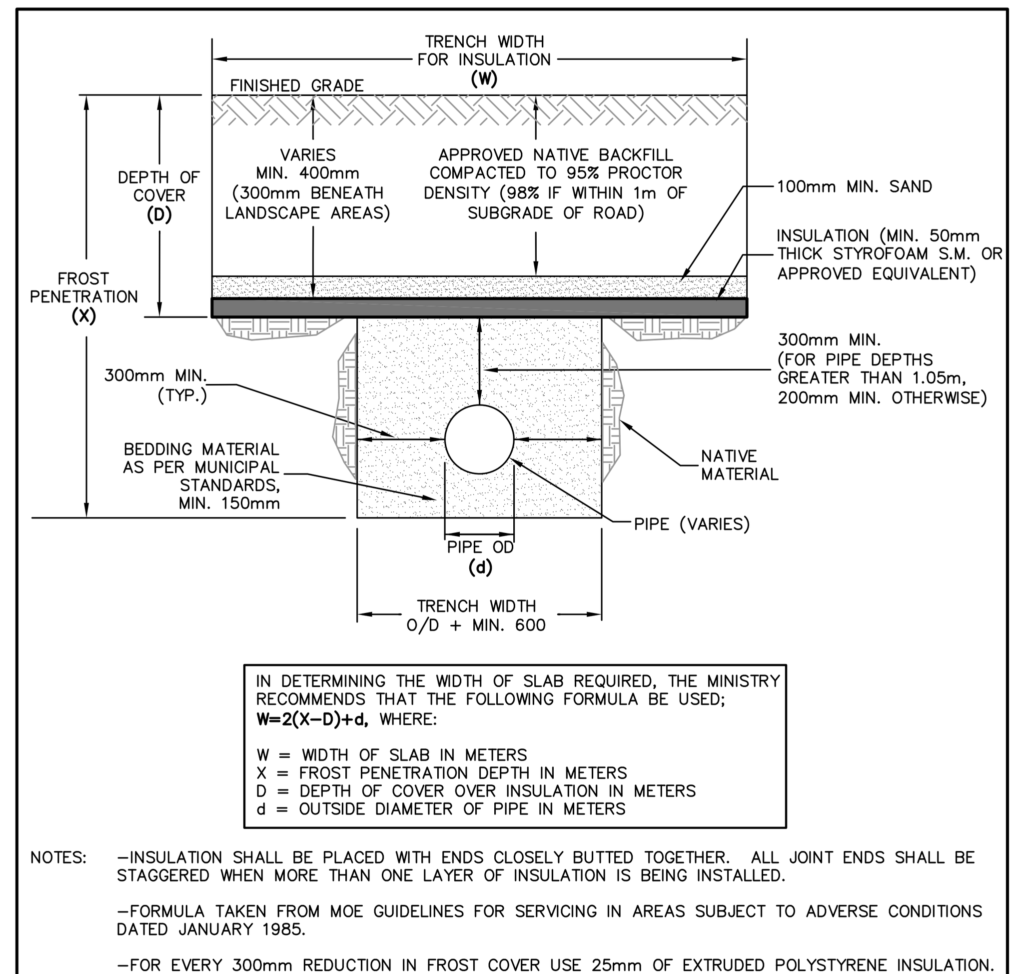
PAVEMENT LAP JOINT

N.T.S.



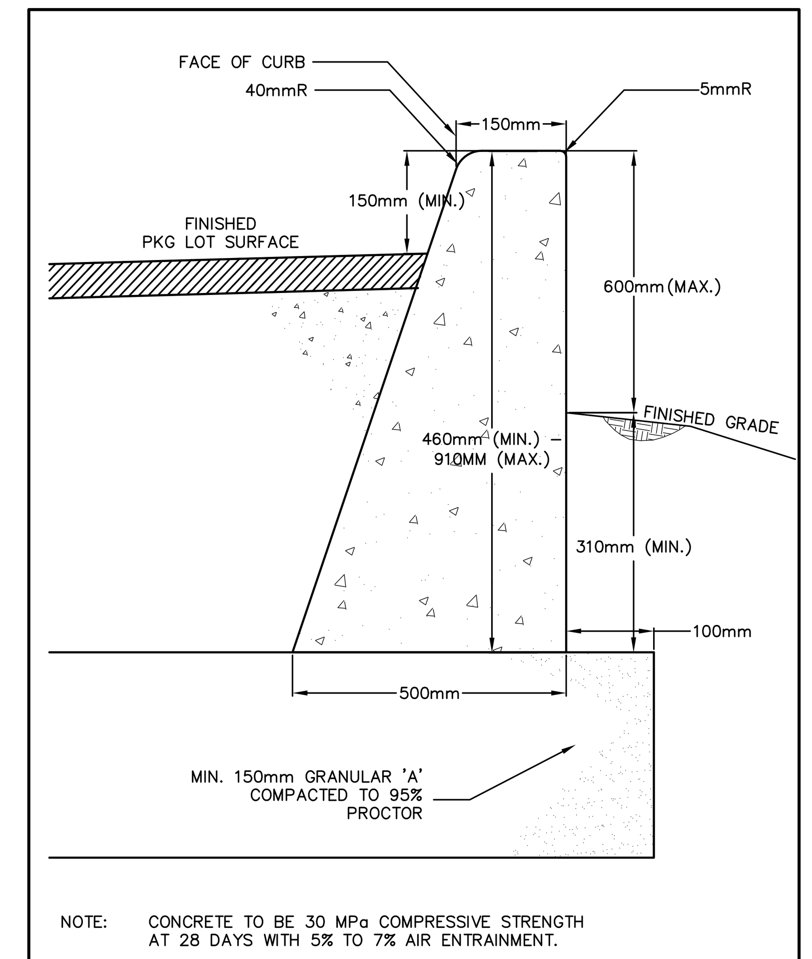
PAVEMENT CROSS-SECTIONS

N.T.S.



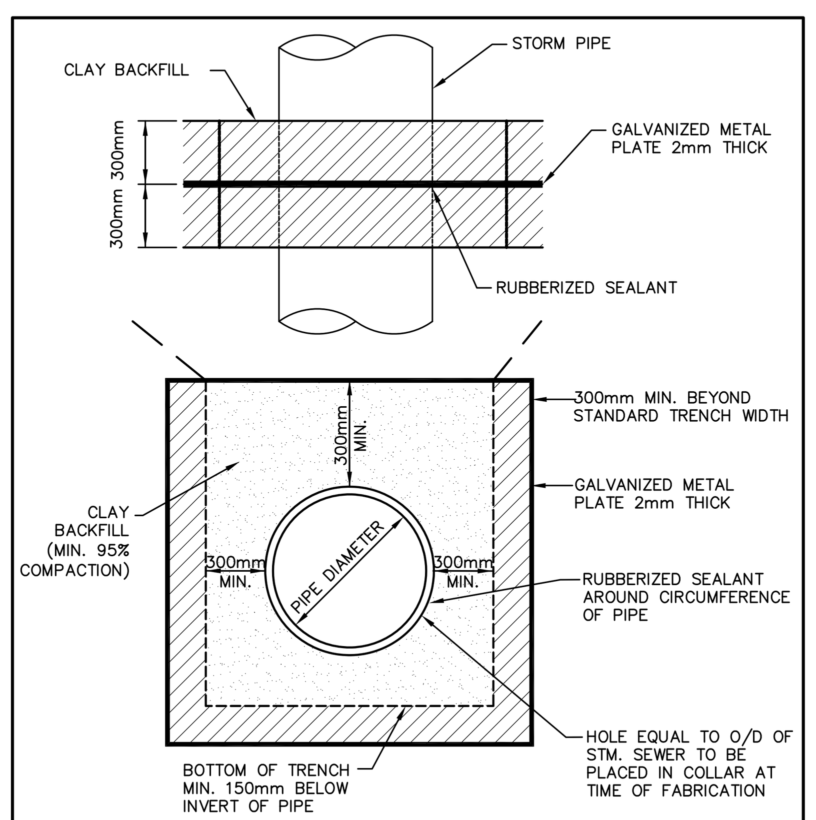
INSULATION FOR SHALLOW PIPES

N.T.S.



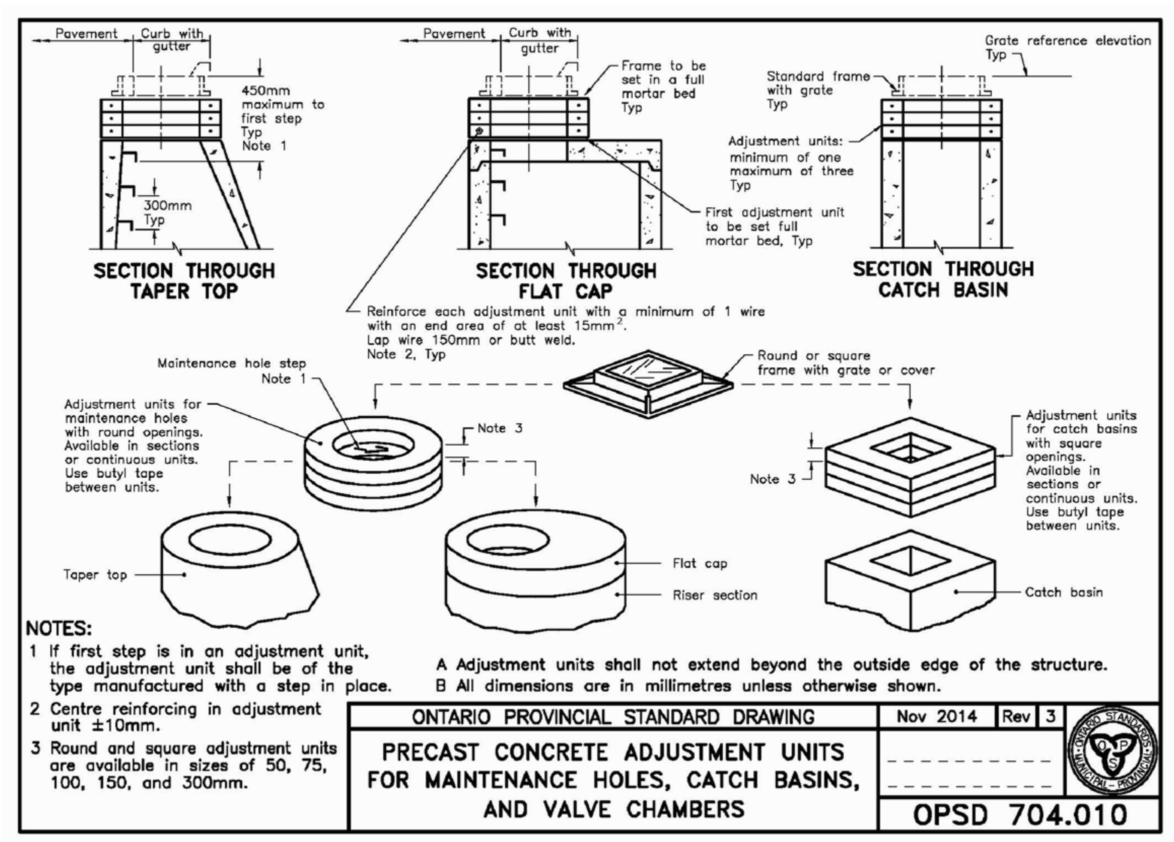
EXTENDED CONCRETE CURB DETAIL

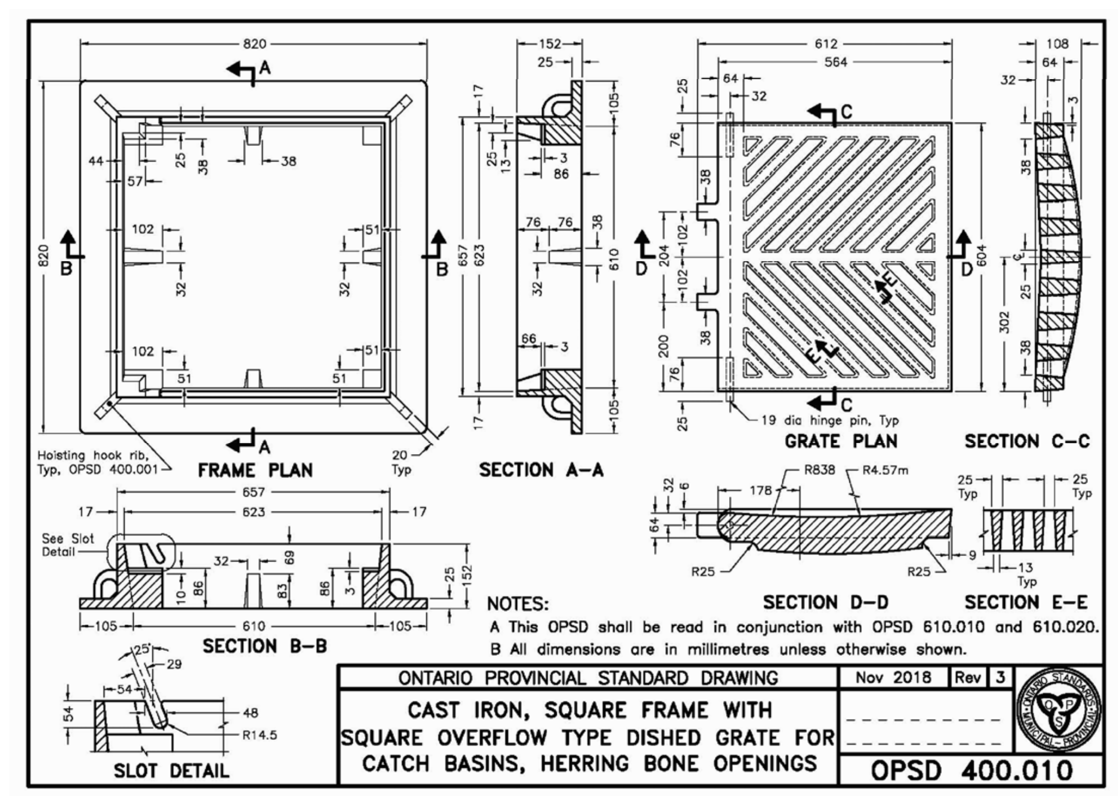
N.T.S.



ANTI-SEEPAGE COLLAR DETAIL

N.T.S.





CAUTION
CONTRACTOR TO DETERMINE LOCATION OF EXISTING UTILITIES PRIOR TO CONSTRUCTION.

Notes:

1. Unless noted otherwise, the measurements and distances shown on this drawing are shown in meters.
2. Do not scale drawings.
3. It is the contractor's responsibility to verify all dimensions, levels and datums on site and report any discrepancies or omissions to WMI & Associates Ltd. prior to construction.
4. This drawing is to be read and understood in conjunction with all other relevant documents applicable to this project.
5. This drawing is the exclusive property of WMI & Associates Ltd. and the reproduction of any part of this document without prior written consent is strictly prohibited.

Benchmark: 202.945m

TOP NUT OF EXISTING FIRE HYDRANT LOCATED ON THE NORTH SIDE OF HIGHWAY No.12, APPROXIMATELY 200m WEST OF THE INTERSECTION OF WILLIAM STREET AND HIGHWAY No.12.

REGISTERED PROFESSIONAL ENGINEER S. R. MORASH July 8, 2019 PROVINCE OF ONTARIO

No.	Issue / Revision	Date
1	INTERNAL REVIEW	JUNE 21, 2019
2	FIRST SUBMISSION	JULY 8, 2019

265 WHITFIELD CRESCENT

DETAIL SHEET 1

Client:
Jason Redman
699 Aberdeen Blvd. Unit 902, Midland ON, L4M 1H2

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

Drawn By	Checked By	Drawing No.
AW	DW	DS1
Scale 1:200	Project No. 19-543	

\\Wmi-server\wmi-server\Data\Projects\2019\19-543\CAD\WMI\Issue_No1\190705_19-543_baseplan.dwg, DS1, 1:25.4

APPENDIX B

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: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** AW

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	0.30	0.45	0.60
	10 - 30% grade	0.40	0.65	0.70
Pasture Land	0 - 5% grade	0.10	0.28	0.40
	5 - 10% grade	0.15	0.35	0.45
	10 - 30% grade	0.22	0.40	0.55
Woodlot or Cutover	0 - 5% grade	0.08	0.25	0.35
	5 - 10% grade	0.12	0.30	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)	0.40	0.50	0.60
Residential	Single Family	0.30	0.40	0.50
	Multiple (i.e. semi, townhouse, apartment, etc.)	0.50	0.60	0.70
Industrial	Light	0.55	0.65	0.75
	Heavy	0.65	0.75	0.85
Commercial		0.60	0.70	0.80
Unimproved Areas		0.10	0.20	0.30
Lawn	< 2% grade	0.05	0.11	0.17
	2 - 7% grade	0.10	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**

PRE-DEVELOPMENT CONDITION

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	0.03		
	10 - 30% grade			
Lakes and Wetlands				
Impervious Area	(i.e. buildings, roads, parking lot, etc.)			
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		0.39		
Lawn	< 2% grade			
	2 - 7% grade			
	> 7% grade			

Total Area (ha) = 0.42 **Runoff Coefficient, C =** **0.10**

POST-DEVELOPMENT CONDITION

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.)	0.25		
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	0.17		

Total Area (ha) = 0.42 **Runoff Coefficient, C =** **0.63**

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[1.0_C_CALCS.xlsx]C CALCS

WMI & Associates Limited
119 Collier Street, Barrie, Ontario L4M 1H5
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RATIONAL METHOD CALCULATIONS

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** AW

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (T_C / 60)^B \quad (mm/hr)$$

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

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

2-year	C_2 =	C
5-year	C_5 =	C
10-year	C_{10} =	C
25-year	C_{25} =	1.10 x C
50-year	C_{50} =	1.20 x C
100-year	C_{100} =	1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

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

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

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	0.42	15.0	0.10	0.001	0.006	0.009	0.010	0.013	0.016	0.018
POST	0.42	15.0	0.63	0.024	0.041	0.055	0.064	0.082	0.100	0.114

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[3.0_Rational_Method_Calcs(A,B)_Final.xlsx]Rational Method

WMI & Associates Limited
119 Collier Street, Barrie, Ontario L4M 1H5
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MODIFIED RATIONAL METHOD CALCULATIONS
2-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

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

2-year	C_2 =	C
5-year	C_5 =	C
10-year	C_{10} =	C
25-year	C_{25} =	1.10 x C
50-year	C_{50} =	1.20 x C
100-year	C_{100} =	1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[4.0_Modified_Rational_Method_Calcs(A,B).xlsx]100YR

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)
POST	2-year	0.42	0.63	0.63	15	5	0.0083

NOTES: - Due to its low magnitude and the constraint of using the MOE minimum recommended orifice size (0.075m), the 2-year post-development peak flow is attenuated to just above the 2-year pre-development target rate of 0.0060m3/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)
	15	55.6	0.041	36.78	7.47	29.31	
	20	45.5	0.033	40.11	8.72	31.40	
	25	38.9	0.029	42.90	9.96	32.94	
	30	34.3	0.025	45.32	11.21	34.11	
	35	30.8	0.023	47.47	12.45	35.02	
	40	28.0	0.021	49.42	13.70	35.72	
	45	25.8	0.019	51.20	14.94	36.26	
	50	24.0	0.018	52.85	16.19	36.66	
	55	22.4	0.016	54.39	17.43	36.96	
	60	21.1	0.016	55.83	18.68	37.16	
	65	20.0	0.015	57.19	19.92	37.27	
DESIGN STORM DURATION >>>	**70**	18.9	0.014	58.48	21.17	37.32	**37.32**
	75	18.1	0.013	59.71	22.41	37.30	
	80	17.3	0.013	60.88	23.66	37.23	
	85	16.5	0.012	62.00	24.90	37.10	
	90	15.9	0.012	63.08	26.15	36.93	
	95	15.3	0.011	64.11	27.39	36.72	
	100	14.8	0.011	65.11	28.64	36.48	
	105	14.3	0.010	66.07	29.88	36.19	
	110	13.8	0.010	67.01	31.13	35.88	
	115	13.4	0.010	67.91	32.37	35.54	
	120	13.0	0.010	68.78	33.62	35.17	
	125	12.6	0.009	69.63	34.86	34.77	
	130	12.3	0.009	70.46	36.11	34.36	
	135	12.0	0.009	71.27	37.35	33.92	
	140	11.7	0.009	72.05	38.60	33.45	
	145	11.4	0.008	72.81	39.84	32.97	
	150	11.1	0.008	73.56	41.09	32.48	
	155	10.9	0.008	74.29	42.33	31.96	
	160	10.6	0.008	75.00	43.58	31.43	
	165	10.4	0.008	75.70	44.82	30.88	

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MODIFIED RATIONAL METHOD CALCULATIONS
5-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

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

2-year	C_2 =	C
5-year	C_5 =	C
10-year	C_{10} =	C
25-year	C_{25} =	1.10 x C
50-year	C_{50} =	1.20 x C
100-year	C_{100} =	1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[4.0_Modified_Rational_Method_Calcs(A,B).xlsx]100YR

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)
POST	5-year	0.42	0.63	0.63	15	5	0.0095

NOTES: - Due to its low magnitude and the constraint of using the MOE minimum recommended orifice size (0.075m), the 2-year post-development peak flow is attenuated to just above the 2-year pre-development target rate of 0.0090m3/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)
	15	74.3	0.055	49.16	8.55	40.61	
	20	60.8	0.045	53.61	9.98	43.63	
	25	52.0	0.038	57.33	11.40	45.93	
	30	45.8	0.034	60.57	12.83	47.74	
	35	41.1	0.030	63.44	14.25	49.19	
	40	37.4	0.028	66.04	15.68	50.37	
	45	34.5	0.025	68.43	17.10	51.33	
	50	32.0	0.024	70.63	18.53	52.11	
	55	30.0	0.022	72.69	19.95	52.74	
	60	28.2	0.021	74.62	21.38	53.24	
	65	26.7	0.020	76.44	22.80	53.64	
	70	25.3	0.019	78.16	24.23	53.94	
	75	24.1	0.018	79.80	25.65	54.15	
	80	23.1	0.017	81.37	27.08	54.29	
	85	22.1	0.016	82.86	28.50	54.36	
DESIGN STORM DURATION >>>	**90**	21.2	0.016	84.30	29.93	54.38	**54.38**
	95	20.5	0.015	85.69	31.35	54.34	
	100	19.7	0.015	87.02	32.78	54.24	
	105	19.1	0.014	88.31	34.20	54.11	
	110	18.5	0.014	89.55	35.63	53.93	
	115	17.9	0.013	90.76	37.05	53.71	
	120	17.4	0.013	91.93	38.48	53.45	
	125	16.9	0.012	93.06	39.90	53.16	
	130	16.4	0.012	94.17	41.33	52.84	
	135	16.0	0.012	95.25	42.75	52.50	
	140	15.6	0.011	96.29	44.18	52.12	
	145	15.2	0.011	97.32	45.60	51.72	
	150	14.9	0.011	98.31	47.03	51.29	
	155	14.5	0.011	99.29	48.45	50.84	
	160	14.2	0.010	100.24	49.88	50.37	
	165	13.9	0.010	101.18	51.30	49.88	

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MODIFIED RATIONAL METHOD CALCULATIONS
10-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

<<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

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

2-year	C_2 =	C
5-year	C_5 =	C
10-year	C_{10} =	C
25-year	C_{25} =	1.10 x C
50-year	C_{50} =	1.20 x C
100-year	C_{100} =	1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[4.0_Modified_Rational_Method_Calcs(A,B).xlsx]100YR

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)
POST	10-year	0.42	0.63	0.63	15	5	0.0102

NOTES: - Due to its low magnitude and the constraint of using the MOE minimum recommended orifice size (0.075m), the 2-year post-development peak flow is attenuated to just above the 2-year pre-development target rate of 0.0100m3/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)
	15	86.4	0.064	57.18	9.18	48.00	
	20	70.7	0.052	62.35	10.71	51.64	
	25	60.5	0.044	66.68	12.24	54.44	
	30	53.2	0.039	70.45	13.77	56.68	
	35	47.8	0.035	73.79	15.30	58.49	
	40	43.5	0.032	76.82	16.83	59.99	
	45	40.1	0.029	79.59	18.36	61.23	
	50	37.3	0.027	82.15	19.89	62.26	
	55	34.9	0.026	84.55	21.42	63.13	
	60	32.8	0.024	86.79	22.95	63.84	
	65	31.0	0.023	88.91	24.48	64.43	
	70	29.4	0.022	90.91	26.01	64.90	
	75	28.1	0.021	92.82	27.54	65.28	
	80	26.8	0.020	94.64	29.07	65.57	
	85	25.7	0.019	96.38	30.60	65.78	
	90	24.7	0.018	98.05	32.13	65.92	
	95	23.8	0.017	99.66	33.66	66.00	
DESIGN STORM DURATION >>>	**100**	23.0	0.017	101.21	35.19	66.02	**66.02**
	105	22.2	0.016	102.71	36.72	65.99	
	110	21.5	0.016	104.16	38.25	65.91	
	115	20.8	0.015	105.56	39.78	65.78	
	120	20.2	0.015	106.92	41.31	65.61	
	125	19.6	0.014	108.25	42.84	65.41	
	130	19.1	0.014	109.53	44.37	65.16	
	135	18.6	0.014	110.78	45.90	64.88	
	140	18.1	0.013	112.00	47.43	64.57	
	145	17.7	0.013	113.19	48.96	64.23	
	150	17.3	0.013	114.35	50.49	63.86	
	155	16.9	0.012	115.49	52.02	63.47	
	160	16.5	0.012	116.60	53.55	63.05	
	165	16.2	0.012	117.68	55.08	62.60	

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MODIFIED RATIONAL METHOD CALCULATIONS
25-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

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

2-year C_2 = C
5-year C_5 = C
10-year C_{10} = C
25-year C_{25} = 1.10 x C
50-year C_{50} = 1.20 x C
100-year C_{100} = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

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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)
POST	25-year	0.42	0.63	0.69	15	5	0.011

NOTES: - The 25-year post-development peak flow is attenuated to the 25-year pre-development target rate of 0.013m3/s or less.

	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)
	15	101.7	0.082	74.02	10.17	63.85	
	20	83.2	0.067	80.72	11.87	68.85	
	25	71.2	0.058	86.32	13.56	72.76	
	30	62.7	0.051	91.19	15.26	75.94	
	35	56.3	0.045	95.52	16.95	78.57	
	40	51.2	0.041	99.44	18.65	80.80	
	45	47.2	0.038	103.03	20.34	82.69	
	50	43.8	0.035	106.35	22.04	84.31	
	55	41.0	0.033	109.44	23.73	85.71	
	60	38.6	0.031	112.35	25.43	86.92	
	65	36.5	0.030	115.09	27.12	87.97	
	70	34.7	0.028	117.68	28.82	88.87	
	75	33.0	0.027	120.15	30.51	89.64	
	80	31.6	0.026	122.51	32.21	90.31	
	85	30.3	0.024	124.77	33.90	90.87	
	90	29.1	0.024	126.93	35.60	91.34	
	95	28.0	0.023	129.02	37.29	91.73	
	100	27.0	0.022	131.02	38.99	92.04	
	105	26.1	0.021	132.96	40.68	92.28	
	110	25.3	0.020	134.84	42.38	92.46	
	115	24.5	0.020	136.65	44.07	92.58	
	120	23.8	0.019	138.41	45.77	92.65	
DESIGN STORM DURATION >>>	**125**	23.1	0.019	140.13	47.46	92.67	**92.67**
	130	22.5	0.018	141.79	49.16	92.63	
	135	21.9	0.018	143.41	50.85	92.56	
	140	21.3	0.017	144.99	52.55	92.44	
	145	20.8	0.017	146.53	54.24	92.29	
	150	20.3	0.016	148.03	55.94	92.10	
	155	19.9	0.016	149.50	57.63	91.87	
	160	19.4	0.016	150.93	59.33	91.61	
	165	19.0	0.015	152.34	61.02	91.32	

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MODIFIED RATIONAL METHOD CALCULATIONS
50-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

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

2-year C_2 = C
5-year C_5 = C
10-year C_{10} = C
25-year C_{25} = 1.10 x C
50-year C_{50} = 1.20 x C
100-year C_{100} = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2\text{-}100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

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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)
POST	50-year	0.42	0.63	0.76	15	5	0.012

NOTES: - The 50-year post-development peak flow is attenuated to the 50-year pre-development target rate of 0.016m3/s or less.

	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)
	15	113.1	0.100	89.74	10.98	78.76	
	20	92.5	0.082	97.86	12.81	85.05	
	25	79.1	0.070	104.66	14.64	90.02	
	30	69.6	0.061	110.57	16.47	94.10	
	35	62.5	0.055	115.82	18.30	97.52	
	40	57.0	0.050	120.57	20.13	100.44	
	45	52.5	0.046	124.92	21.96	102.96	
	50	48.7	0.043	128.94	23.79	105.15	
	55	45.6	0.040	132.69	25.62	107.07	
	60	42.9	0.038	136.22	27.45	108.77	
	65	40.6	0.036	139.54	29.28	110.26	
	70	38.5	0.034	142.69	31.11	111.58	
	75	36.7	0.032	145.68	32.94	112.74	
	80	35.1	0.031	148.54	34.77	113.77	
	85	33.6	0.030	151.27	36.60	114.67	
	90	32.3	0.028	153.90	38.43	115.47	
	95	31.1	0.027	156.42	40.26	116.16	
	100	30.0	0.026	158.86	42.09	116.77	
	105	29.0	0.026	161.21	43.92	117.29	
	110	28.1	0.025	163.48	45.75	117.73	
	115	27.2	0.024	165.68	47.58	118.10	
	120	26.4	0.023	167.82	49.41	118.41	
	125	25.7	0.023	169.89	51.24	118.65	
	130	25.0	0.022	171.91	53.07	118.84	
	135	24.3	0.021	173.87	54.90	118.97	
	140	23.7	0.021	175.79	56.73	119.06	
DESIGN STORM DURATION >>>	**145**	23.2	0.020	177.65	58.56	119.09	**119.09**
	150	22.6	0.020	179.48	60.39	119.09	
	155	22.1	0.019	181.26	62.22	119.04	
	160	21.6	0.019	183.00	64.05	118.95	
	165	21.2	0.019	184.70	65.88	118.82	

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MODIFIED RATIONAL METHOD CALCULATIONS
100-year Design Storm

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

[ ] <<< **Elements Requiring Input Information**

Rainfall Intensity-Duration-Frequency Coefficients from: http://www.mto.gov.on.ca/IDF_Curves/terms.shtml

2-year		5-year		10-year		25-year		50-year		100-year	
A =	21.1	A =	28.2	A =	32.8	A =	38.6	A =	42.9	A =	47.2
B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699	B =	-0.699

Rational Method Formula

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

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

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

2-year C_2 = C
5-year C_5 = C
10-year C_{10} = C
25-year C_{25} = 1.10 x C
50-year C_{50} = 1.20 x C
100-year C_{100} = 1.25 x C

For storms having a return period of more than 10 years, the Runoff Coefficient, C, will be increased as shown above up to a maximum coefficient of 0.95.

Rainfall Intensity Equation (2-100 year storm events)

$$I_{2-100} = A \times (t_d/60)^B \quad (mm/hr)$$

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

Runoff Volume

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

where, Q_{Runoff} = Runoff Peak Flow Rate, (m^3/sec)
t_d = Storm Duration, (sec)

Released Volume

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

where, $Q_{Released}$ = Max. Release Rate, (m^3/sec)
t_d = Storm Duration, (sec)
T_C = Time of Concentration, (sec)

Max. Storage Required

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

V_{Runoff} = Runoff Volume, (m^3)
$V_{Released}$ = Released Volume, (m^3)

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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)
POST	100-year	0.42	0.63	0.79	15	15	0.013

NOTES: - The 100-year post-development peak flow is attenuated to the 100-year pre-development target rate of 0.018m3/s or less.

	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)
	15	124.4	0.114	102.85	11.61	91.24	
	30	76.6	0.070	126.72	17.42	109.30	
	45	57.7	0.053	143.16	23.22	119.94	
	60	47.2	0.043	156.11	29.03	127.09	
	75	40.4	0.037	166.96	34.83	132.13	
	90	35.6	0.033	176.38	40.64	135.74	
	105	31.9	0.029	184.76	46.44	138.32	
	120	29.1	0.027	192.33	52.25	140.09	
	135	26.8	0.025	199.27	58.05	141.22	
	150	24.9	0.023	205.69	63.86	141.84	
DESIGN STORM DURATION >>>	**165**	23.3	0.021	211.68	69.66	142.02	**142.02**
	180	21.9	0.020	217.30	75.47	141.83	
	195	20.7	0.019	222.60	81.27	141.33	
	210	19.7	0.018	227.62	87.08	140.54	
	225	18.7	0.017	232.39	92.88	139.51	
	240	17.9	0.016	236.95	98.69	138.27	
	255	17.2	0.016	241.32	104.49	136.83	
	270	16.5	0.015	245.50	110.30	135.21	
	285	15.9	0.015	249.53	116.10	133.43	
	300	15.3	0.014	253.41	121.91	131.51	
	315	14.8	0.014	257.16	127.71	129.45	
	330	14.3	0.013	260.79	133.52	127.28	
	345	13.9	0.013	264.30	139.32	124.98	
	360	13.5	0.012	267.71	145.13	122.59	
	375	13.1	0.012	271.02	150.93	120.09	
	390	12.8	0.012	274.24	156.74	117.50	
	405	12.4	0.011	277.37	162.54	114.83	
	420	12.1	0.011	280.42	168.35	112.08	
	435	11.8	0.011	283.40	174.15	109.25	
	450	11.5	0.011	286.31	179.96	106.35	
	465	11.3	0.010	289.15	185.76	103.39	

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

Date: 07-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

<<< **Elements Requiring Input Information**

Unsubmerged Orifice (Weir Flow)

$Q = C_W L H^{3/2}$ (m^3/s)

where, Q = Flow through unsubmerged orifice (m^3/s)
C_W = Weir Coefficient
H = Head/Depth of water acting on weir measured from above the crest/invert of orifice (m)
L = Length of weir (m)
D = Diameter of Pipe/Orifice (m)
For circular vertical weir,
L = Wetted Perimeter
$L = D \times \cos^{-1}((D/2 - H)/(D/2))$
For circular horizontal weir,
L = Circumference
L = 3.14 x D

Submerged Orifice (Orifice Flow)

$Q = C_O A_O (2gH)^{1/2}$ (m^3/s)

where, Q = Flow through submerged orifice (m^3/s)
C_O = Orifice Discharge Coefficient
A_O = Cross-sectional area of orifice (m^2)
g = Gravitational acceleration ($9.81m^2/s$)
For circular vertical orifice,
H = Head/Depth of water acting on orifice measured from centroid of the opening (m)
For circular horizontal orifice,
H = Head/Depth of water acting on orifice measured from above the invert (m)

Unsubmerged Weir (Weir Flow)

Rectangular Broad- & Sharp-Crested Weirs
$Q = C_W L H^{3/2}$ (m^3/s)
Triangular Broad-Crested Weirs
$Q = 1.225H^{5/2}\tan(Theta/2)$ (m^3/s)
Triangular Sharp-Crested Weirs
$Q = 0.581(8/15)(2g)^{1/2}\tan(Theta/2)H^{5/2}$ (m^3/s)
Trapezoidal Broad- & Sharp-Crested Weirs
$Q_{TRAPEZOIDAL} = Q_{RECTANGULAR} + Q_{TRIANGULAR}$ (m^3/s)

where, Q = Flow through unsubmerged weir (m^3/s)
C_W = Weir Coefficient
(1.65 for Broad-Crested)
(1.80 for Sharp-Crested)
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 ($9.81m^2/s$)

Submerged Weir (Orifice Flow)

Submerged Sharp-Crested Weirs
$Q = C_O A_O (2gH)^{1/2}$ (m^3/s)

where, Q = Flow through submerged weir opening (m^3/s)
C_O = Orifice Discharge Coefficient
A_O = Cross-sectional area of opening (m^2)
g = Gravitational acceleration ($9.81m^2/s$)
H = Head/Depth of water acting on orifice measured from centroid of the opening (m)

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.

Starting Water Elevation, m = **186.20**
Incremental Depth, m = **0.02**

	Orifice 1	Orifice 2	Orifice 3	Weir 1	Weir 2	Weir 3	
				Trapezoidal			= Weir Type
Orifice Type =	Vertical			Sharp-Crested			
Orifice Invert Elev., m =	186.20			187.35			= Weir Crest Elev., m
Incremental Depth, m =	0.02	0.02	0.02	0.02	0.02	0.02	= Incremental Depth, m
Water Elev. @ Inflow, m =	186.20						= Weir Openings Crown Elev., m (if appl.)
Orifice Diameter, m =	0.075			1.50			= Weir Length, m
Centroid of Orifice, m =	186.238			1.80			= Weir Coefficient
Orifice Area, m^2 =	0.0044			3			= Side Slope (H:1)
Orifice Coefficient =	0.63			72			= Theta/2, Degrees
Weir Coefficient =	1.80						= Centroid of Orifice, m (if appl.)
							= Orifice Area, m^2 (if appl.)
							= Orifice Coefficient (if appl.)

Elevation (m)	Area 1 (m^2)	Area 2 (m^2)	Total Area (m^2)	Storage Volume (m^3)
186.20	51.07		51.1	**0.0**
187.50	237.00		237.0	**187.2**

^
Only increments of 0.01m are valid

NOTES: Due to the low magnitude of the pre-development target rates, the minimum orifice size (0.075m) recommended by the MOE was used to attenuate post-development peak flows to the pre-development targets for the 25-year, 50-year, and 100-year design storms. This results in the peak flows from the 2-year, 5-year, 10-year design storm events being controlled to rates slightly above pre-development levels. It is not feasible to control these respective storm events any further than proposed, moreover, it should be noted that the slight increase in these flows is negligible and should not impact existing downstream infrastructure.

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)	Notes
Base	**186.20**	0.0000						**0.0000**	**0.0**	
	186.68	0.0082						**0.0082**	**37.2**	**2-year storm (Q=0.0083m3/s, V=37.20m3 at 186.68m)**
	186.70	0.0084						**0.0084**	**39.4**	
	186.72	0.0086						**0.0086**	**41.6**	
	186.74	0.0087						**0.0087**	**44.0**	
	186.76	0.0089						**0.0089**	**46.4**	
	186.78	0.0091						**0.0091**	**48.8**	
	186.80	0.0092						**0.0092**	**51.3**	
	186.82	0.0094						**0.0094**	**53.9**	**5-year storm (Q=0.0095m3/s, V=54.38m3 at 186.83m)**
	186.84	0.0096						**0.0096**	**56.5**	
	186.86	0.0097						**0.0097**	**59.2**	
	186.88	0.0099						**0.0099**	**62.0**	
	186.90	0.0100						**0.0100**	**64.9**	
	186.92	0.0102						**0.0102**	**67.8**	**10-year storm (Q=0.0102m3/s, V=66.02m3 at 186.91m)**
	186.94	0.0103						**0.0103**	**70.8**	
	186.96	0.0105						**0.0105**	**73.8**	
	186.98	0.0106						**0.0106**	**77.0**	
	187.00	0.0108						**0.0108**	**80.2**	
	187.02	0.0109						**0.0109**	**83.5**	
	187.04	0.0110						**0.0110**	**86.8**	
	187.06	0.0112						**0.0112**	**90.3**	
	187.08	0.0113						**0.0113**	**93.8**	**25-year storm (Q=0.0113m3/s, V=92.67m3 at 187.09m)**
	187.10	0.0114						**0.0114**	**97.4**	
	187.12	0.0116						**0.0116**	**101.0**	
	187.14	0.0117						**0.0117**	**104.8**	
	187.16	0.0118						**0.0118**	**108.6**	
	187.18	0.0120						**0.0120**	**112.5**	
	187.20	0.0121						**0.0121**	**116.5**	
	187.22	0.0122						**0.0122**	**120.6**	**50-year storm (Q=0.0122m3/s, V=119.09m3 at 187.22m)**
	187.24	0.0123						**0.0123**	**124.8**	
	187.26	0.0125						**0.0125**	**129.1**	
	187.28	0.0126						**0.0126**	**133.4**	
	187.30	0.0127						**0.0127**	**137.8**	
Freeboard	187.32	0.0128						**0.0128**	**142.4**	**100-year storm (Q=0.0129m3/s, V=142.02m3 at 187.32m)**
	187.34	0.0129						**0.0129**	**147.0**	
	187.36	0.0131			0.003			**0.0158**	**151.7**	
	187.38	0.0132			0.015			**0.0278**	**156.5**	
	187.40	0.0133			0.032			**0.0458**	**161.4**	
	187.42	0.0134			0.055			**0.0687**	**166.3**	
	187.44	0.0135			0.083			**0.0964**	**171.4**	
	187.46	0.0136			0.115			**0.1287**	**176.6**	
	187.48	0.0137			0.152			**0.1654**	**181.9**	
Top	187.50	0.0139			0.193			**0.2066**	**187.2**	

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

MANNING'S PIPE EQUATION
STORM SEWER PIPE DESIGN

Date: 19-Jun-19 **Project No.:** 19-543

Project: 265 Whitfield Crescent **Prepared By:** BD

Mannings' Equation		
Pipe Diameter (mm) =	375	
Area =	0.110	m^2
Slope (%) =	0.50	
Manning 'n' =	0.013	
Nom'l Pipe Capacity, Q =	0.124	m^3/sec
Actual Pipe Capacity, Q =	**0.129**	m^3/sec

Box Culvert		
Height =	0.00	
Width =	0	
Q =	0.00	
Velocity		
	1.12	m/s
	1.17	m/s
R=	**0.09375**	
Sf=	**0.0050**	

NOTE: Pipe sized to convey the 100-year peak flow (0.114cu.m/s) from the site.

\\WMI-SERVER\wmi-server\Data\Projects\2019\19-543\Design\Storm\Issue_#1\[8.0_190702_Mannings_Pipe_Eqn.xlsx]Mannings_Pipe

APPENDIX C

GEOTECHNICAL INVESTIGATION / HYDROGEOLOGICAL EVALUATION

Ian D. Wilson Associates Ltd.
since 1974

Tel: 519.233.3500
Fax: 519.233.3501

P. O. Box 299
Clinton, Ontario
N0M 1L0

May 24, 2019

Wilson Associates
Consulting Hydrogeologists

Mr. David Walter, C.E.T.
WMI & Associates Limited
119 Collier Street
Barrie, Ontario
L4M 1H5

Dear Mr. Walter:

Re: Hydrogeological Study and Water Balance Analysis
265 Whitfield Crescent, Town of Midland

It is proposed to develop an existing 0.4247ha property at 265 Whitfield Crescent in the Town of Midland as self storage facility.

As requested by WMI & Associates, this report has been prepared to address the requirements of the June 2013 "Hydrogeological Assessment Submissions: Conservation Authority Guidelines for Development Applications" (the CA Guideline).

Provided for this study were the following documentation:

- Geotechnical Investigation Report, 1000 William Street & 265 Whitfield Crescent, Midland. Cambium Inc. (Cambium), April 1, 2019.

- Site Servicing & Grading Plan, WMI & Associates Limited, May 2019.

Copies of the above documentation are attached for reference.

LOCATION AND HYDROGEOLOGICAL SETTING

The subject lands at 265 Whitfield Crescent occupy a 0.4247ha, rectangularly-shaped parcel situated on the west side of Whitfield Crescent. The site is currently undeveloped, and mostly cleared. The site exhibits a moderate slope to the east, with an approximate relief of 5 to 6m.

No surface water bodies are mapped on the property. A small water feature is mapped (per Simcoe County website) nearby to the east (on 1000 William Street), possibly functioning as a perched groundwater feature atop low-permeability soils, but is not connected to a surface water body. Wetland associated with the Wye River is mapped about 50m to the south of the southwest corner of the property.

Lands surrounding the site to the north, east and south are mainly developed as commercial properties. Lands immediately to the west of the site are undeveloped.

The subject lands are located within the Simcoe Uplands physiographic region of southern Ontario, an area of northern Simcoe County characterized by till upland plains and steep-sided, flat floored valleys. According to the Ontario Geological Survey Map P.975 "Quaternary Geology of the Orr Lake (Western Half) - Nottawasaga Area (Eastern Half)", the native upper soils beneath the site are reported to consist of glaciolacustrine shallow water deposits of sand with minor fine gravel or ice-contact deposits of gravel and sand. According to the Cambium Report, site-specific test pits identified that the upper soils on the site consist of gravelly sand.

According to a historical water well record for a well drilled nearby to the northeast (MECP Well Record # 57-7708, attached), the overburden in the vicinity of the site is about 24 metres deep, and consists largely of sand with some intermediate-depth fine-grained deposits. The 2005 North Simcoe Municipal Groundwater Study (Cross-Section B) indicates that the overburden sands form regional Aquifers A2 and A3.

The bedrock beneath the site consists mainly of limestone and dolostone of the Simcoe Group.

Although the area is municipally serviced, municipal and historical water wells will have obtained potable groundwater from aquifers in the lower overburden. The bedrock beneath the site is not locally typically used as a source of potable groundwater due to the likelihood of obtaining lower yields of aesthetically-poorer quality groundwater.

According to the 2015 Severn Sound Source Protection Area Approved Assessment Report (the Severn Sound Report), the site is not located within a well head protection area (WHPA-A through WHPA-E). The Simcoe County Interactive Mapping Website indicates that the site is located within Well Head Protection Zone WHPA-Q2 of the Russell and Heritage municipal well fields (located >1km to the northwest and southwest), however the site is not mapped to be located within a significant groundwater recharge area or a highly vulnerable aquifer area.

WATERTABLE

Watertable conditions were observed by Cambium in open test pits, and are summarized in Table 5 of the Cambium report. To generally summarize the Cambium Table 5 data, no groundwater was encountered in the on-site test pits.

Locally, Figure 4.4.1 of the 2005 North Simcoe Municipal Groundwater Study (NSMGS) indicates that shallow groundwater will flow eastwards towards the Wye River system.

WATER BUDGET ANALYSIS

The following assumptions are made for this assessment:

- Based on the small site area and relatively consistent relief, the site is assumed to act as one catchment. The site is considered to exhibit a rolling topography (per the 1995 MECP definitions referenced by the CA guideline) and sandy soil conditions (native upper soils reported by Cambium and by Quaternary Geology mapping).

- According to calculations provided by WMI & Associates Limited, the 0.4247ha site currently exhibits a pervious area of 100% (0.4247ha) and an impervious area of 0% (0ha). The proposed development of the site will exhibit a pervious area of 35.2% (0.1495ha) and an impervious area of 64.8% (0.2752ha).

- The water surplus for the site is assumed to be 384mm/year, as identified for the Wye River subwatershed by the 2015 Severn Sound Report (precipitation 967mm/year, actual evapotranspiration 583mm/year). Normal precipitation for the area is 1040.6mm/year (1981-2010 precipitation normal for the closest Environment Canada weather station - Midland WPCP weather station). For this assessment, the 2015 Severn Sound Report precipitation rate of 967mm/year is assumed.

The following tables provide a water budget analysis following the general guidance of the April 2013 Conservation Authority Guidelines for Hydrogeological Assessments.

Table 1 - Water Budget - Undeveloped Conditions

Catchment Designation	Site	
	Undeveloped	Totals
Area (m²)	4247	4247
Pervious Area (m²)	4247	4247
Impervious Area (m²)	0	0
Impervious Factors (Per MECP Guidelines referenced by CA Guideline)		
Topography Infiltration Factor	Rolling 0.20	
Soil Infiltration Factor	Sand 0.4	
Land Cover Infiltration Factor	Cleared 0.1	
MOECC Infiltration Factor	0.7	
Actual Infiltration Factor	0.7	
Run-Off Coefficient	0.3	
Runoff from Impervious Surfaces*	0	
Inputs (per Unit Area)		
Precipitation (mm/year)	967	967
Run-On (mm/year)	0	0
Other Inputs (mm/year)	0	0
Total Inputs (mm/year)	967	967
Outputs (per Unit Area)		
Precipitation Surplus (mm/year)	384	384
Net Surplus (mm/year)	384	384
Evapotranspiration (mm/year)	583	583
Infiltration (mm/year)	269	269
Impervious Area Infiltration (mm/year)	0	0
Total Infiltration (mm/year)	269	269
Runoff Pervious Areas (mm/year)	115	115
Runoff Impervious Areas (mm/year)	0	0
Total Runoff (mm/year)	115	115
Total Outputs (mm/year)	967	967
Difference (Inputs - Outputs) (mm/year)	0	0

Inputs (Volume)		
Precipitation (m³/year)	4107	4107
Run-On (m³/year)	0	0
Other Inputs (m³/year)	0	0
Total Inputs (m³/year)	4107	4107
Outputs (Volume)		
Precipitation Surplus (m³/year)	1631	1631
Net Surplus (m³/year)	1631	1631
Evapotranspiration (m³/year)	2476	2476
Infiltration (m³/year)	1142	1142
Impervious Area Infiltration (m³/year)	0	0
Total Infiltration (m³/year)	1142	1142
Runoff Pervious Areas (m³/year)	488	488
Runoff Impervious Areas (m³/year)	0	0
Total Runoff (m³/year)	488	488
Total Outputs (m³/year)	4106	4106
Difference (Inputs - Outputs) (m³/year)	-1**	-1**

Note: ** Minor differences attributable to rounding.

Table 2 - Water Budget - Post-Development Conditions

Under Post-Development conditions, the proposed re-development of the site will exhibit a pervious area of 35.2% (0.1495ha) and an impervious area of 64.8% (0.2752ha).

Catchment Designation	Site		
	Pervious	Impervious	Totals
Area (m^2)	1495	2752	4247
Pervious Area (m^2)	1495	0	1495
Impervious Area (m^2)	0	2752	2752
Impervious Factors (Per MECP Guidelines referenced by CA Guideline)			
Topography Infiltration Factor	Rolling 0.20	Rolling 0.20	
Soil Infiltration Factor	Sand 0.4	Sand 0.4	
Land Cover Infiltration Factor	Cleared 0.1	Cleared 0.1	
MOECC Infiltration Factor	0.7	0.7	
Actual Infiltration Factor	0.7	0.7	
Run-Off Coefficient	0.3	1	
Runoff from Impervious Surfaces*	0	0.8	
Inputs (per Unit Area)			
Precipitation (mm/year)	967	967	967
Run-On (mm/year)	0	0	0
Other Inputs (mm/year)	0	0	0
Total Inputs (mm/year)	967	967	967
Outputs (per Unit Area)			
Precipitation Surplus (mm/year)	384	774	637
Net Surplus (mm/year)	384	774	637
Evapotranspiration (mm/year)	583	193	330
Infiltration (mm/year)	269	0	95
Impervious Area Infiltration (mm/year)	0	0	0
Total Infiltration (mm/year)	269	0	95
Runoff Pervious Areas (mm/year)	115	0	40
Runoff Impervious Areas (mm/year)	0	774	502
Total Runoff (mm/year)	115	774	542
Total Outputs (mm/year)	967	967	967
Difference (Inputs - Outputs) (mm/year)	0	0	0

Inputs (Volume)			
Precipitation (m³/year)	1446	2661	4107
Run-On (m³/year)	0	0	0
Other Inputs (m³/year)	0	0	0
Total Inputs (m³/year)	1446	2661	4107
Outputs (Volume)			
Precipitation Surplus (m³/year)	574	2130	2704
Net Surplus (m³/year)	574	2130	2704
Evapotranspiration (m³/year)	871	531	1402
Infiltration (m³/year)	402	0	402
Impervious Area Infiltration (m³/year)	0	0	0
Total Infiltration (m³/year)	402	0	402
Runoff Pervious Areas (m³/year)	172	0	172
Runoff Impervious Areas (m³/year)	0	2130	2130
Total Runoff (m³/year)	172	2130	2302
Total Outputs (m³/year)	1445	2661	4106
Difference (Inputs - Outputs) (m³/year)	-1**	0	-1**

Note: * Per guidelines, evaporation from impervious areas assumed to be 20% of precipitation.

** Minor differences attributable to rounding.

Table 3 - Water Budget - Post-Development Conditions with Mitigation

Based on the above assessment, approximately 740m^3/year (35%) of the runoff from the impervious areas of the site will need to be infiltrated on the site in order to maintain the overall rate of infiltration relative to pre-development conditions. The viability of infiltrating this volume of water is discussed below.

Catchment Designation	Site		
	Pervious	Impervious	Totals
Area (m^2)	1495	2752	4247
Pervious Area (m^2)	1495	0	1495
Impervious Area (m^2)	0	2752	2752
Impervious Factors (Per MECP Guidelines referenced by CA Guideline)			
Topography Infiltration Factor	Rolling 0.20	Rolling 0.20	
Soil Infiltration Factor	Sand 0.4	Sand 0.4	
Land Cover Infiltration Factor	Cleared 0.1	Cleared 0.1	
MOECC Infiltration Factor	0.7	0.7	
Actual Infiltration Factor	0.7	0.7	
Run-Off Coefficient	0.3	1	
Runoff from Impervious Surfaces*	0	0.8	
Inputs (per Unit Area)			
Precipitation (mm/year)	967	967	967
Run-On (mm/year)	0	0	0
Other Inputs (mm/year)	0	0	0
Total Inputs (mm/year)	967	967	967
Outputs (per Unit Area)			
Precipitation Surplus (mm/year)	384	774	637
Net Surplus (mm/year)	384	774	637
Evapotranspiration (mm/year)	583	193	330
Infiltration (mm/year)	269	0	95
Impervious Area Infiltration (mm/year)	0	269	174
Total Infiltration (mm/year)	269	0	269
Runoff Pervious Areas (mm/year)	115	0	40
Runoff Impervious Areas (mm/year)	0	505	327
Total Runoff (mm/year)	115	505	368
Total Outputs (mm/year)	967	967	967

Difference (Inputs - Outputs) (mm/year)	0	0	0
Inputs (Volume)			
Precipitation (m³/year)	1446	2661	4107
Run-On (m³/year)	0	0	0
Other Inputs (m³/year)	0	0	0
Total Inputs (m³/year)	1446	2661	4107
Outputs (Volume)			
Precipitation Surplus (m³/year)	574	2130	2704
Net Surplus (m³/year)	574	2130	2704
Evapotranspiration (m³/year)	871	531	1402
Infiltration (m³/year)	402	0	402
Impervious Area Infiltration (m³/year)	0	740	740
Total Infiltration (m³/year)	402	0	1142
Runoff Pervious Areas (m³/year)	172	0	172
Runoff Impervious Areas (m³/year)	0	1390	1390
Total Runoff (m³/year)	172	1390	1562
Total Outputs (m³/year)	1445	2661	4106
Difference (inputs - Outputs) (m³/year)	-1**	0	-1**

Note: * Per guidelines, evaporation from impervious areas assumed to be 20% of precipitation.

** Minor differences attributable to rounding.

Table 4 - Water Budget Summary

Characteristic	Site				
	Current	Post-Development	% Change (Current to Post)	Post Development with Mitigation	% Change (Current to Post with Mitigation)
Inputs (Volumes)					
Precipitation (m^3/year)	4107	4107	0	4107	0
Run-On (m^3/year)	0	0	0	0	0
Other Inputs (m^3/year)	0	0	0	0	0
Total Inputs (m^3/year)	4107	4107	0	4107	0
Outputs (Volumes)					
Precipitation Surplus (m^3/year)	1631	2704	66	2704	66
Net Surplus (m^3/year)	1631	2704	66	2704	66
Evapotranspiration (m^3/year)	2476	1402	-43	1402	-43
Infiltration (m^3/year)	1142	402	-65	402	-65
Impervious Area Infiltration (m^3/year)	0	0	0	740	35
Total Infiltration (m^3/year)	1142	402	-65	1142	0
Runoff Pervious Areas (m^3/year)	488	172	-65	172	-65
Runoff Impervious Areas (m^3/year)	0	2130	+2130 m^3/year	1390	+1390 m^3/year
Total Runoff (m^3/year)	488	2302	371	1562	220
Total Outputs (m^3/year)	4106	4106	0	4106	0

Mitigation assumes that 35% of runoff from the impervious areas of the site can be infiltrated on-site, or about 740m^3/year. It is assumed that most of this will be infiltrated into grass swales, infiltration galleries, or other equivalent Low Impact Development (LID) measures. According to the grain-size analyses for the upper overburden deposits provided in the Cambium report (for TP5 GS2, attached), the native soils (i.e. a gravelly sand) will exhibit a percolation rate (T-time) in the range of 20min/cm (per Cambium interpretation of Ontario Building Code guidelines for Unified Soil Classification Type "SP"), or about 0.72m/day. Conservatively assuming that the impervious area drainage of 740m^3/year is to be infiltrated over 30 days throughout the year, approximately 25m^3 of water needs to be infiltrated per day. Based on an infiltration rate of 0.72m/day, LID measures with a total site footprint of at least 35m^2 are required.

SUMMARY

1. The upper soils on the site consist of gravelly sand.

2. Based on the Cambium Inc. Test Pit data, no shallow groundwater was encountered.

3. The site is located within Well Head Protection Zone WHPA-Q2 of the Russell and Heritage municipal well fields (located >1km to the northwest and southwest), however the site is not mapped to be located within a significant groundwater recharge area or a highly vulnerable aquifer area.

4. Based on known site conditions (i.e. sandy soils, rolling relief, cleared cover), an MECP infiltration factor of 0.7 is indicated for the undeveloped site.

5. Water budget analysis indicates that the development proposal of the site will reduce overall infiltration by about 65% from pre-development conditions.

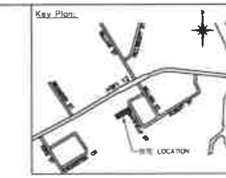
6. Due to the calculated loss in overall infiltration of the development proposal in comparison to pre-development conditions, infiltration enhancement measures must be adopted to infiltrate approximately 35% of runoff from impervious surfaces. It is assumed that most of this will be infiltrated into grass swales, infiltration galleries, or other equivalent Low Impact Development (LID) measures (see above for minimum LID areas). The infiltration measures need to be maintained in a low-sediment condition to avoid infiltration loss over time.

Should there be any questions regarding the above information and analysis, please feel free to contact this office.

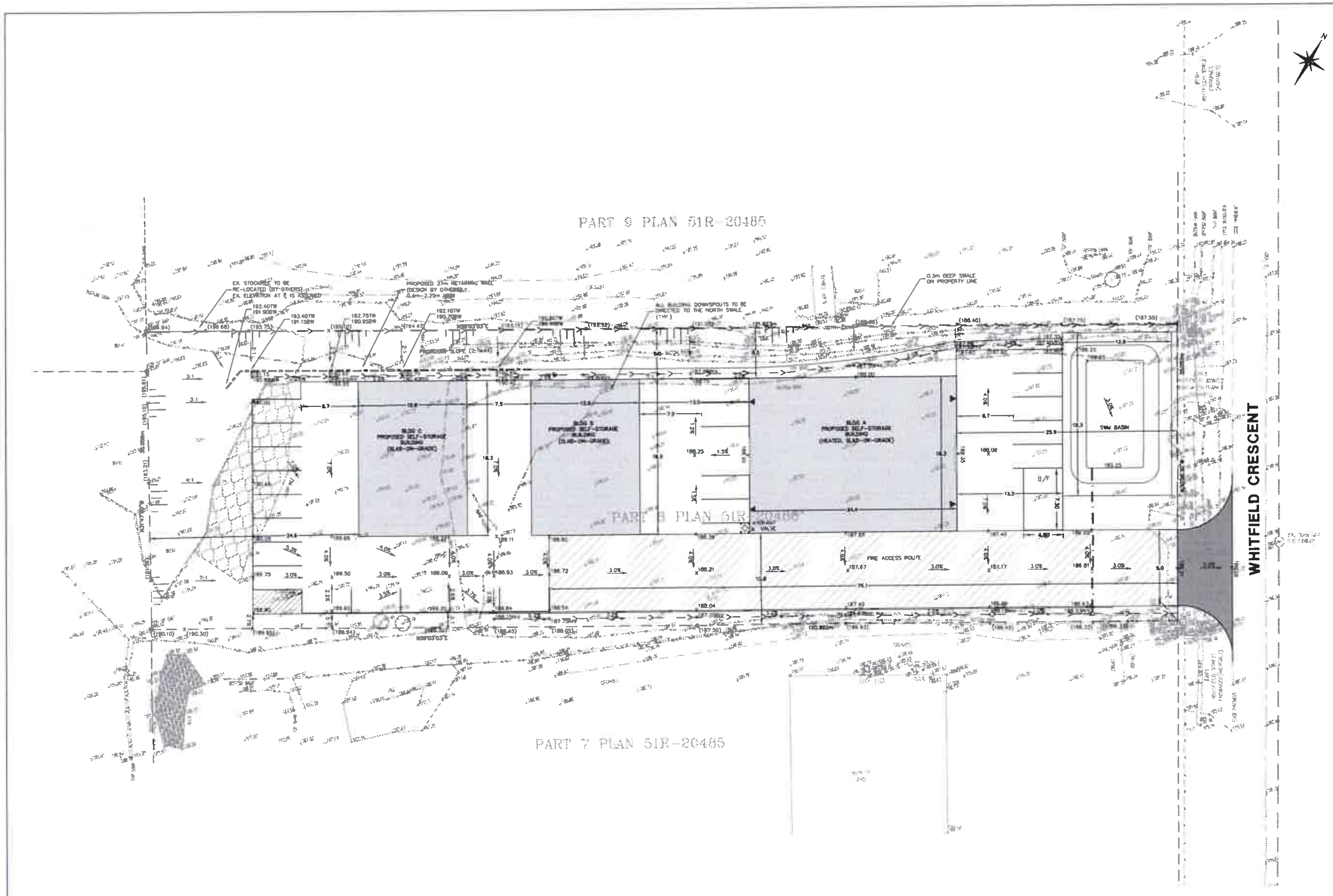
Yours sincerely.
IAN D. WILSON ASSOCIATES LIMITED

Geoffrey Rether, P.Geo.

PROFESSIONAL GEOSCIENTIST
GEOFFREY B. RETHER
PRACTISING MEMBER
0426
ONTARIO



SITE STATISTICS		
ZONING – (M1) INDUSTRIAL		
	REQUIRED	PROVIDED
MINIMUM LOT AREA	0.4 ha	0.42ha
MINIMUM FRONTAGE	30.0 m	35.05m
MAXIMUM COVERAGE	[illegible]	21.7%
MINIMUM SETBACKS		
FRONT	7.5m	27.8m
REAR	8.8m	21.7m
INTERNAL SIDE	6.0m	6.0m
BUILDING AREA		911sqm
PARKING	1 PER [illegible] (23)	22
B/F	1	1
TOTAL		23



Notes:

1. Unless noted otherwise, the measurements and distances shown on this drawing are shown in meters
2. Do not scale drawings.
3. It is the contractor's responsibility to verify all dimensions, levels and datums on site and report any discrepancies or omissions to WMI & Associates Ltd. prior to construction.
4. This drawing is to be read and understood in conjunction with all other relevant documents applicable to this project
5. This drawing is the exclusive property of WMI & Associates Ltd. and the reproduction of any part of this document without prior written consent is strictly prohibited

Benchmark: 202.945m

TOP NUT OF EXISTING FIRE HYDRANT LOCATED ON THE NORTH SIDE OF HIGHWAY No.12, APPROXIMATELY 200m WEST OF THE INTERSECTION OF WILLIAM STREET AND HIGHWAY No.12.

PRELIMINARY

No.	Issue / Revision	Date
1	OWNER REVIEW	MAY 14, 2019

265 WHITFIELD CRESCENT

SITE SERVICING & GRADING PLAN

Client:
Jason Redmon
699 Aberdeen Blvd. Unit 902,
Midland ON,
L4M 1H2

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

Drawn By	Checked By	Drawing No.
AB	DW	SSGR
Scale 1:200	Project No. 19-543	

Geotechnical Investigation Report 1000 William Street & 265 Whitfield Crescent, Midland, Ontario

Cambium Reference No.: 8679-001

April 01, 2019

Prepared for: Jason Redman

CAMBIUM

Cambium Inc.

74 Cedar Pointe Drive, Unit 1009
Barrie, Ontario, L4N 5R7

Telephone: (866) 217.7900
Facsimile: (705) 742.7907

cambium-inc.com

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1.0 INTRODUCTION

Cambium Inc. (Cambium) was retained by WMI & Associates on behalf of Jason Redman (Client) to complete a geotechnical investigation in support of the design and construction of a commercial storage development at 1000 William Street and an assessment of subsurface conditions at 265 Whitfield Crescent in Midland, Ontario (Site).

The William Street property is currently used as outdoor heavy equipment and construction materials storage, the lot is rectangular, relatively flat, and approximately 2.25 acres in size with fill noted across the center and eastern extents of the site, with the western extents appearing to have recently been stripped. The Whitfield Crescent property is currently vacant and undeveloped, the lot is rectangular, has rolling topography and is approximately 1 acre in size.

The proposed development at 1000 William Street consist of numerous 1-storey storage structures throughout the site, driving and parking areas, and storm water management features at the west and east ends of the site. At the time of investigation the development details of the 265 Whitfield Crescent site were understood to consist of a 1-storey office building, two 1-storey storage structures, driving and parking areas, outdoor storage areas, and a storm water management feature at the east end of the site. Following consultation with the Client, Cambium was directed that a test pit investigation was the Client's preferred method to sample and test the in-situ subsurface soils.

The geotechnical investigation was required to confirm the subsurface conditions at the Site in order to provide geotechnical design parameters as input into the design and construction of the proposed storage development. A Site Plan, including test pit locations, is included as Figure 1 of this report.

2.0 METHODOLOGY

2.1 TEST PIT INVESTIGATION

A test pit investigation was completed on February 27th, 2019, to assess subsurface conditions at the Site. A total of six (6) test pits, designated as TP101-19 through TP106-19, were advanced throughout each of the properties. All of the test pits were terminated at depths ranging from 1.8 m to 3.1 m below ground surface (mbgs). The test pit locations were selected and laid out in consultation with the Client. Test pits TP101-19 through TP104-19 were advanced throughout the William Street property, generally adjacent to proposed structures. Test pits TP105-19 and TP106-19 were advanced at the eastern and western ends of the Whitfield Crescent property to classify the native soils present at the site.

The test pit elevations and locations were surveyed by DEMTech Services. The test pit UTM's where surveyed by Cambium with a handheld Garmin etrex 20x and are provided in Table 4 and on the test pit logs, elevations are provided in Table 3 and on the test pit logs. Test pit locations are shown on Figure 1.

Test pits were advanced using a track mounted CAT 312 hydraulic excavator, equipped with a frost ripper and toothed bucket, provided by the client and supervised by a Cambium technician. Dynamic probe penetration tests (DPT), consisting of measuring the number of blows required to advance a 19 mm diameter steel rod into the subgrade soils a distance of 150 mm using an 8 kg hammer falling 750 mm, were attempted in each test pit to determine the in-situ density and bearing capacity of the subgrade soils.

The encountered soil units were logged in the field using visual and tactile methods, and samples were placed in labelled plastic bags for transport, future reference, possible laboratory testing, and storage.

Open test pits were checked for groundwater and general stability prior to backfilling. The test pits were backfilled with the excavated material, compacted with the bucket of the excavator, and the property was reinstated to as close to pre-existing conditions as possible.

Test pit logs are provided in Appendix A. Site soil and groundwater conditions are described and geotechnical recommendations are discussed in the following sections of this report.

2.2 PHYSICAL LABORATORY TESTING

Physical laboratory testing, including four (4) sieve and hydrometer analyses (LS-702, 705), was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters. Natural moisture content testing (LS-701) was completed on all retrieved soil samples. Results are presented in Appendix B and are discussed in Section 3.0.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site consist predominantly of topsoil or fill soils overlying clayey silt or till soils predominantly grading from a sandy silt to silt matrix. These soils were encountered throughout the test pits to the termination depths ranging from 1.5 mbgs to 3.1 mbgs. A layer of fill soil consisting of either sandy soils or clayey silt soils was noted at the surface of each of the test pit locations within the William Street property, the fill soils generally extended to depths between 0.8 mbgs and 1.5 mbgs. It should be noted that organic soils were encountered below the fill soils in test pits TP103-19 and TP104-19. All the test pits were terminated in native soils, and bedrock was not encountered within the excavation depths.

The test pit locations are shown on Figure 1 and the individual soil units are described in detail below with test pit logs provided in Appendix A. A summary of the depth of imported fill and topsoil is provided in Table 1 as an overview, with further descriptions provided below.

Table 1 Summary of Depths of Fill and Topsoil Across Site

Test Pit	Depth of Imported Fill (mbgs)	Depth of Organics (mbgs)	Description of Organics
TP101-19	0 – 1.5	-	-
TP102-19	0 – 1.5	-	-
TP103-19	0 – 0.8	0.8 – 1.1	Topsoil
TP104-19	0 – 0.9	0.9 – 1.2	Topsoil
TP105-19	-	0 – 0.6	Topsoil
TP106-19	-	0 – 0.3	Topsoil

3.1 TOPSOIL

A layer of black to brown topsoil between 300 mm and 600 mm in thickness was encountered at the surface of test pits TP105-19 and TP106-19 advanced at 265 Whitfield Crescent. The topsoil was frozen at the time of the investigation and loose in relative density. Black topsoil with some rootlets and organics was also noted beneath the fill soils in TP103-19 and TP104-19; in both test pits the topsoil was observed to be approximately 300 mm thick.

3.2 FILL SOILS

A layer of fill soils was observed at the surface of test pits TP101-19 through TP104-19 on the William Street property, and was generally brown sand with some gravel and silt, trace clay and occasional cobble, the exception being TP104-19 where the fill was predominately brown clayey silt, trace sand and likely reworked native soils. The fill extended to depths between 0.8 mbgs and 1.5 mbgs, and is summarized in Table 1. Based on visual inspection and observations during excavations the soils were noted as loose to compact in relative density with a natural moisture content ranging between 4% and 13%.

Laboratory particle size distribution analyses were completed for two (2) samples of the fill soils, taken from the test pits and depths provided in Table 2 in order to identify the varying textures encountered throughout the fill material. The testing results are provided in Appendix B and are summarized in Table 2 based on the Unified Soils Classification System (USCS).

Table 2 Particle Size Distribution – Fill Soils

TP	Depth (mbgs)	Description	% Gravel	% Sand	% Silt	% Clay
TP102-19	1.5	Sand some Silt some Gravel trace Clay	14	66	17	3
TP103-19	0.3	Sand some Gravel some Silt trace Clay	16	66	14	4

3.3 NATIVE SOILS

Beneath the fill soils discussed above, the native soils consisted glaciofluvial ice-contact deposits generally consisting of till material with varying amounts of silt and sand throughout the test pit locations, which extended to the termination depths ranging from 1.8 mbgs to 3.1 mbgs.

The texture of the native soils varied at each property. At 1000 William Street the native soils encountered was predominantly brown clayey silt, with trace sand. The DPT penetration resistances indicated a firm to very stiff consistency. Based on laboratory testing, the natural moisture content ranged between 16% and 38%. All of the test pits located in this property were terminated in the native clayey silt soils.

At 265 Whitfield Crescent, the native soils were predominately brown silty gravelly sand with trace clay inferred as a till material. Based on the DPT penetration resistances this material had a compact to very dense relative density with natural moisture content between 5% and 6%. Both test pits TP105-19 and TP106-19 were terminated in the native silty gravelly sand.

Laboratory particle size distribution analyses were completed for two (2) samples of the native soils, taken from the test pits and depths provided in Table 3 in order to identify the varying textures encountered throughout the overburden material. The testing results are provided in Appendix B and are summarized in Table 3 based on the USCS.

Table 3 Particle Size Distribution – Native Soils

TP	Depth (mbgs)	Description	% Gravel	% Sand	% Silt	% Clay
TP101-19	2.1	Silt and Clay trace Sand	0	5	54	41
TP105-19	1.8	Gravelly Silty Sand trace Clay	26	39	28	7

3.4 BEDROCK

Bedrock was not encountered within the investigation depths. Each of the test pits were terminated at depths ranging from 1.8 mbgs to 3.1 mbgs generally in native soils, the exception being TP102-19 which was terminated in fill soils at 1.5 mbgs. The elevation of each test pit and their respective termination depths are identified in Table 4 below.

Table 4 Test Pit Termination Depth – Elevations

Test Pit ID	Test Pit Elevation (mASL)	Test Pit Termination Depth (mbgs)	Test Pit Termination Elevation (mASL)
TP101-19	187.31	2.4	184.91
TP102-19	186.51	2.1	184.41
TP103-19	186.42	3.1	183.32
TP104-19	187.12	3.1	184.02
TP105-19	**	1.8	**
TP106-19	**	1.8	**

***Test pits not surveyed by DEMTech*

3.5 GROUNDWATER

Groundwater (free water) was noted in test pits TP101-19, TP102-19 and TP103-19. The observed groundwater elevation and caving (sloughing) depths are summarised in Table 5. Given the presence of predominately granular fill overlying low permeable clayey silt along the central and western extents of 1000 William Street, it is possible that observed groundwater may be perched seepage in this area.

The moisture content of the soils generally ranged from 3% to 43%. It should be noted that soil moisture and groundwater levels at the Site may fluctuate seasonally and in response to climatic events.

Table 5 Ground Water and Caving Observations

Test Pit ID	Test Pit Elevation (mASL)	Depth to Groundwater (mbgs)	Ground Water Elevation (mASL)	Caving Depth (mbgs)
TP101-19	187.31	1.2	186.11	0.9
TP102-19	186.51	1.3	185.21	1.2
TP103-19	186.42	1.5	184.92	-
TP104-19	187.12	-	-	-
TP105-19	**	-	-	-
TP106-19	**	-	-	-

***Test pits not surveyed by DEMTech*

3.6 INFILTRATION TESTING

In order to help determine the infiltration rates, four (4) particle size distribution tests (hydrometer analyses) were completed on samples as described in Section 3.2. In order to determine the rate at which water will be absorbed into the soil ("T" time), the soil was classified according to the USCS and the T Time was interpolated based on the USCS gradation charts for the two particle size distribution tests (hydrometer analyses) described in Section 3.2 and 3.3 of this report. The hydraulic conductivity was calculated based on the Puckett equation. The results are summarised in Tables 6, 7 and 8 and the T time is included on the grain size distribution charts in Appendix B.

Table 6 Infiltration Results – Fill Soils

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP102-19	1.8	10 mins/cm	Silty Sand (SM)	2.4×10^{-5} m/s
TP103-19	0.3	9 mins/cm	Silty Sand (SM)	2.0×10^{-5} m/s

Table 7 Infiltration Results – Native Soils (1000 William Street)

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP101-19	2.1	> 50 mins/cm	Silt (ML)	1.3×10^{-8} m/s

Table 8 Infiltration Results – Native Soils (265 Whitfield Crescent)

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP105-19	1.8	20 mins/cm	Silt (ML)	1.1×10^{-5} m/s

Based on these test results we believe a percolation time of 10 mins/cm is appropriate for the gravelly sand fill soils, 20 mins/cm for the gravelly silty sand at 265 Whitfield Crescent and > 50 mins/cm for the silt soils at 1000 William Street.

4.0 GEOTECHNICAL CONSIDERATIONS

The following recommendations are based on test pit information and are intended to assist designers. Recommendations should not be construed as providing instructions to contractors, who should form their own opinions about site conditions. It is possible that subsurface conditions beyond the test pit locations may vary from those observed. If significant variations are found before or during construction, Cambium should be contacted so that we can reassess our findings, if necessary.

4.1 SITE PREPARATION

The existing fill material and any organic materials encountered should be excavated and removed from beneath any structures which will be occupied (i.e., offices, maintenance buildings, residential, etc.); additionally this material should be excavated and removed to a minimum distance of 3 m around the proposed occupied building footprint. The fill material may potentially be left in place beneath the single storey storage units and driving areas, however an additional test pitting program is recommended to confirm that the site was stripped prior to the placement of existing fill and/or delineate the extent of the organics at 1000 William Street, as organics and topsoil were noted in TP103-19 and TP104-19. The fill material includes, but is not limited to the fill identified in this report. Any topsoil and materials with significant quantities of organics and deleterious materials (i.e., construction debris, asphalt etc.) are not appropriate for use as fill below storage units and driving areas.

The exposed subgrade should be proof-rolled and inspected by a qualified geotechnical engineer prior to placement of granular fill or foundations. Any loose/soft soils identified at the time of proof-rolling that are unable to uniformly be compacted should be sub-excavated and removed. The excavations created through the removal of these materials should be backfilled with approved engineered fill consistent with the recommendations provided below. Additionally the test pit locations summarized below in Table 9 should be excavated to the termination depths provided in Table 4 and reinstated with approved engineered fill should they be situated beneath any load bearing structural elements (i.e., footings).

The near surface sand and silt soils can be very unstable if they are wet or saturated. Such conditions are common in the spring and late fall. Under these conditions, temporary use of granular fill, and possible reinforcing geotextiles, may be required to prevent severe rutting on construction access routes.

Table 9 Test Pit UTM Coordinates

Test Pit ID	UTM Zone	UTM Northing	UTM Easting
TP101-19***	17 T	590548	4953893
TP102-19***	17 T	590557	4953975
TP103-19***	17 T	590696	4953893
TP104-19***	17 T	590557	4953975
TP105-19	17 T	590408	4953928
TP106-19	17 T	590359	4953882

****Test pit locations also provided in DEMTech Topographic Survey*

4.2 FROST PENETRATION

Based on climate data and design charts, the maximum frost penetration depth below the surface at the site is estimated at 1.6 mbgs.

If strip and spread foundations are to be used, exterior footings for the proposed structures should be situated at or below this depth for frost penetration or should be adequately insulated.

It is assumed that the pavement structure thickness will be less than 1.6 m, so grading and drainage are important for good pavement performance and life expectancy. Any services should be located below this depth or be appropriately insulated.

4.3 EXCAVATIONS AND BACKFILL

All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). The generally loose to compact fill and native soils may be classified as Type 3 soils above the groundwater table in accordance with OHSA. Type 3 soils may be excavated with side slopes no steeper than 1H:1V. Below the groundwater table the soils may be classified as Type 4 soils and may be excavated with unsupported side slopes no steeper than 3H:1V.

4.4 DEWATERING

Groundwater was encountered in three (3) of the six (6) test pits at TP101-19, TP102-19 and TP103-19 at depths ranging from 1.2 mbgs to 1.5 mbgs, given the presence of predominately granular fill overlying low permeable clayey silt in this area, it is possible that observed groundwater may be perched seepage. Seepage may occur across the Site if high groundwater conditions are present during construction due to seasonal fluctuations. If groundwater seepage is encountered it should be manageable with filtered sumps and pumps and depending on size of excavation, a Permit to Take Water (PTTW) from the Ministry of the Environment and Climate Change (MOECC) will likely not be required. It is noted that the elevation of the groundwater table will vary due to seasonal conditions and in response to heavy precipitation events. In order to minimize predictable water issues and costs, it is recommended that excavation and in-ground construction be performed in drier seasons.

4.5 BACKFILL AND COMPACTION

Excavated topsoil from the Site is not appropriate for use as fill below grading and parking areas. Excavated sand soils not containing organics, may be appropriate for use as fill below grading and parking areas, provided that the actual or adjusted moisture content at the time of construction is within a range that permits compaction to required densities, and that the material is only used below frost penetration depth of 1.6 m below proposed grade. Some moisture content adjustments may be required depending upon seasonal conditions. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality.

Any engineered fill below foundations should be placed in lifts appropriate to the type of compaction equipment used, and be compacted to a minimum of 100% of standard Proctor maximum dry density (SPMDD), as confirmed by nuclear densometer testing. If native soils from the site are not used as engineered fill, imported material for engineered fill should consist of clean, non-organic soils, free of chemical contamination or deleterious material. The moisture content of the engineered fill will need to be close enough to optimum at the time of placement to allow for adequate compaction. Consideration could be given to using a material meeting the specifications of OPSS 1010 Granular B or an approved equivalent. Foundation wall and any buried utility backfill material should consist of free-draining imported granular material. Most of the native site soils are too fine-grained to provide proper drainage, and as such this should be accomplished using well graded Granular B Type 1 material complying with OPSS 1010.

The backfill material, if any, in the upper 300 mm below the pavement subgrade elevation should be compacted to 100 percent of SPMDD in all areas.

4.6 FOUNDATION DESIGN

We understand that the proposed development at 1000 William Street consists of multiple one-storey self-storage units, all with which will be constructed without basements. At the time of investigation, the proposed development plans for 265 Whitfield Crescent consists three (3) one-storey structures which includes one office/maintenance building and two self-storage units, all with which will be constructed without basements. Assuming that the site is prepared as outlined above, the native sub-soils are competent to support all structures on either conventional strip and spread footings or frost protected reinforced raft foundations.

4.6.1 STRIP AND SPREAD FOOTINGS

Assuming any new exterior footings will be placed a minimum of 1.6 m below final adjacent grade for frost protection, these footings can be founded on compact clayey silt or till soils at depth. Any required grade raises to the footing elevations can be accomplished with engineered fill, using an OPSS 1010 SSM or Granular 'B' Type I granular material in 200 mm lifts and compacted to a minimum of 100% of Standard Proctor Maximum Dry Density (SSPMD) as specified above. New footings situated at a minimum depth of 1.6 m below the final adjacent grade, founded in undisturbed compact native clayey silt or till may be designed for an allowable bearing capacity of 100 kPa at serviceability limit state (SLS) and 145 kPa at ultimate limit state (ULS) in all areas.

4.6.2 FROST PROTECTED REINFORCED RAFT FOUNDATION

In addition to the strip and spread footings recommendations above, the storage units may be constructed on frost protected reinforced raft foundations found on either native soils or potentially compact fill soils overlying native inorganic clayey silt subject to the approval by Cambium. Storage units constructed on raft foundations, founded in approved compact fill soils may be designed for an allowable bearing capacity of 50 kPa at SLS and 70 kPa at ULS in all areas. It is noted that topsoil and organics was noted between the fill and inorganic soils in test pits TP103-19 and TP104-19, as such further test pits are recommend prior to construction in order to delineate the underlying topsoil extents. Raft foundations may also be suitable for the proposed office/maintenance building, however given that it would be classified as an occupied structure, it will need to be found on either native soils or approved engineered fill placed and compacted on inorganic soils per Section 4.5.

The quality of the subgrade should be inspected by Cambium during construction, prior to constructing the footings, to confirm bearing capacity estimates and suitability of fill. Settlement potential at the above-noted SLS loadings is less than 25 mm and differential settlement should be less than 10 mm.

4.7 FLOOR SLABS

To create a stable working surface, to distribute loadings, and for drainage purposes, an allowance should be made to provide at least 200 mm of OPSS 1010 Granular A compacted to 98% of SPMDD beneath all floor slabs.

4.8 SUBDRAINAGE

Perimeter subdrains will not be required for structures built on reinforced, raft foundations. Given the investigation was limited to termination depths varying between 1.5 and 3.1 mbgs, if the groundwater table is encountered during excavation for strip footings, geotextile wrapped subdrains set in a trench of clear stone and connected to a sump or other frost-free positive outlet would be recommended around the perimeter of the building foundations.

4.9 BURIED UTILITIES

Trench excavations above the groundwater table should generally consider Type 3 soil conditions, which require side slopes no steeper than 1H:1V, otherwise shoring would be required. Any excavations below the water table should generally consider Type 4 soil conditions which require side slopes of 3H:1V or flatter. Bedding and cover material for any services should consist of OPSS 1010-3 Granular A or B Type II, placed in accordance with pertinent Ontario Provincial Standard Drawings (OPSD 802.013). The bedding and cover material shall be placed in maximum 200 mm thick lifts and should be compacted to at least 98 percent of SPMDD. The cover material shall be a minimum of 300 mm over the top of the pipe and compacted to 98 percent of SPMDD, taking care not to damage the utility pipes during compaction.

4.10 PAVEMENT DESIGN

The performance of the pavement is dependent upon proper drainage and subgrade preparation. All topsoil and organic materials should be removed down to native material and backfilled with approved engineered fill or native material, compacted to 98 percent SPMDD. The subgrade should be proof rolled and inspected by a Geotechnical Engineer. Any areas where boulders, rutting, or appreciable deflection is noted should be subexcavated and replaced with suitable fill. The fill should be compacted to at least 98 percent SPMDD.

From discussions with the client, it is understood that the preference is to have gravel surfaced driving and parking areas throughout the Whitfield Crescent and William Street properties. The recommended pavement structure should satisfy applicable standards for parking and driving areas and should, as a minimum, consist of the pavement layers identified in Table 10.

Table 10 Recommended Minimum Pavement Structure

Pavement Layer	
Granular Surface	100 mm OPSS 1010 Granular M or Granular S
Granular Base	300 mm OPSS 1010 Granular A

Material and thickness substitutions must be approved by the Design Engineer.

The thickness of the base layer could be increased at the discretion of the Engineer, to accommodate site conditions at the time of construction, including soft or weak subgrade soil replacement.

Compaction of the subgrade should be verified by the Engineer prior to placing the granular fill. Granular layers should be placed in 200 mm maximum loose lifts and compacted to at least 98% of SPMDD (ASTM D698) standard. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing.

Drainage features such as subdrains beneath the pavement structure, connecting to the storm sewer or an alternate frost-free outlet, or other drainage alternatives left to the discretion of the designer are recommended to extend the lifespan of the pavement structure.

The final granular surface should be sloped at a minimum of 2 percent to shed runoff, and regular maintenance of the granular surface should be performed to ensure it remains free of surficial deformations.

4.11 DESIGN REVIEW AND INSPECTIONS

Cambium should be retained to complete testing and inspections during construction operations to examine and approve subgrade conditions, placement and compaction of fill materials, granular base courses, and asphaltic concrete.

We should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed. It is important that onsite geotechnical supervision be provided at this site for excavation and backfill procedures, deleterious soil removal, subgrade inspections and compaction testing.

5.0 CLOSING

We trust that the information contained in this report meets your current requirements. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at (705) 719-0700 ext. 405.

Respectfully submitted,

CAMBIUM INC.

LICENSED PROFESSIONAL ENGINEER
R. L. GETHIN
100196175
APRIL 1, 2019
PROVINCE OF ONTARIO

Rob Gethin, P.Eng.
Senior Project Manager

RLG/jb

P:\8600 to 8699\8679-001 Jason Redman - Geotechnical Investigation - #1000 William Street, Midland, ON\Deliverables\REPORT - Geotechnical\Final\2019-04-01 RPT 1000 William & 265 Whitfield Geotech.docx

Appended Figures

GEOTECHNICAL INVESTIGATION

JASON REDMAN
1000 William Street and
265 Whitfield Crescent
Midland, Ontario

LEGEND

- Testpit Locations
- Subject Property(approx.)

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2017 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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TEST PIT LOCATION PLAN

Project No.: 8679-001	Date: March 2019
	Rev.:
Scale: 1:2,000	Projection: NAD 1983 UTM Zone 17N
Created by: SH	Checked by: RG
Figure: 1	

O:\GIS\project_MXDs\8600 to 8699\8679-001 Jason Redman - Geotechnical Investigation - #1000 William Street, Midland ON\2019-03-04 FIG 1 Test Pit Location Plan.mxd

Appendix A
Test Pit Logs

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

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Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP101-19	0 - 1.5	GS1		Brown sand, some gravel, some silt, trace clay, occasional cobble, frozen to 0.6 mbgs, moist, saturated at 1.2 mbgs, loose to compact, FILL	0.61 - 0.76	4
					0.76 - 0.91	13
17T, 590548, 4953893	1.5 - 2.4	GS2		Dark brown to grey clayey silt, trace sand, wet, firm to stiff	0.91 - 1.10	20
				Caving (sloughing) of test pit walls at 0.9 mbgs and seepage noted at 1.2 mbgs	1.10 - 1.22	13
				Test pit terminated at 2.4 mbgs	1.22 - 1.37	8
					1.37 - 1.52	8
				GSA GS2 (2.1 mbgs): 0% Gravel, 5% Sand, 54% Silt, 41% Clay	1.52 - 1.67	5
					1.67 - 1.83	5
					1.52 - 1.67	2
					1.67 - 1.83	3
					1.83 1.98	7
					1.98 2.13	9
					2.13 2.29	12
					2.29 2.44	15
					2.44 2.59	19
					2.59 2.74	21

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP102-19	0 - 1.5	GS1/GS2		Brown sand, some gravel, some silt, trace clay, occasional cobble, frozen to 0.9 mbgs, moist, saturated at 1.35, loose to compact, FILL		
17T, 590557, 4953975	1.5			Grey clayey silt, trace sand, wet, firm to stiff		
				Caving (sloughing) of test pit walls at 1.2 mbgs and seepage noted at 1.3 mbgs		
				Test pit terminated at 1.5 mbgs due to unstable excavation		
				GSA GS2 (1.5 mbgs): 14% Gravel, 66% Sand, 17% Silt, 3% Clay		

[1]: metres below ground surface

[2]: Dynamic Penetration Test

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP103-19	0 - 0.8	GS1		Brown silty sand, some gravel, trace clay, occasional cobble, frozen, compact, FILL	1.52 - 1.67	5
	0.8 - 1.1	GS2		Black sandy silty topsoil, some rootlets and organics, frozen	1.67 - 1.83	5
17T, 590696, 4953893	1.1 - 3.1	GS3/GS4		Brown clayey silt, trace sand, moist to wet, firm to stiff	1.83 1.98	5
				Test pit open upon completion, seepage noted at 1.5 mbgs	1.98 2.13	6
				Test pit terminated at 3.1 mbgs	2.13 2.29	7
					2.29 2.44	6
				GSA GS1 (0.3 mbgs): 16% Gravel, 66% Sand, 15% Silt, 3% Clay	2.44 2.59	6
					2.59 2.74	6

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP104-19	0 - 0.9	GS1		Brown clayey silt, trace sand, frozen to 0.91 mbgs, firm, FILL	1.22 - 1.37	2
	0.9 -1.2	GS2		Black sandy silty topsoil, some rootlets and organics, moist, loose	1.37 - 1.52	8
17T, 590557, 4953975	1.2 - 3.1	GS3/GS4		Brown clayey silt, trace sand, moist, firm to stiff	1.52 - 1.67	7
				Test pit open and dry upon completion	1.67 - 1.83	8
				Test pit terminated at 3.05 mbgs	1.83 - 1.98	7
					1.98 - 2.13	18
					2.13 - 2.29	30
					2.29 - 2.44	15

[1]: metres below ground surface

[2]: Dynamic Penetration Test

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

CAMBIUM

Test Pit ID	Depth (mbgl[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP105-19 17T, 590408, 4953928	0 - 0.6 0.6 -1.8	 GS1/GS2		Black sandy silty topsoil, some rootlets and organics, frozen to 0.6 mbgs Brown silty gravelly sand, some cobbles, trace clay, moist, dense to very dense Grey at 1.8 mbgs Test pit open and dry upon completion Test pit terminated at 1.8 mbgs due to refusal on very dense gravel GSA GS2 (1.8 mbgs) : 26% Gravel, 39% Sand, 28% Silt, 7% Clay	1.22 - 1.37 1.37 - 1.52 1.52 - 1.67	2 30 30 = 125mm

Test Pit ID	Depth (mbgl[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP106-19 17T, 590359, 4953882	0 - 0.3 0.3 - 1.8	 GS1/GS2		Black sandy silty topsoil, some rootlets and organics, frozen to 0.3 mbgs Brown silty gravelly sand, some cobbles, trace clay, moist, dense to very dense Grey at 1.8 mbgs Test pit open and dry upon completion Test pit terminated at 1.8 mbgs due to refusal on very dense gravel	1.22 - 1.37 1.37 - 1.52 1.52 - 1.67 1.67 - 1.83 1.83 - 1.98 1.98 - 2.13	13 15 17 24 24 30 = 125mm

[1]: metres below ground surface

[2]: Dynamic Penetration Test

Appendix B
Physical Laboratory Testing Results

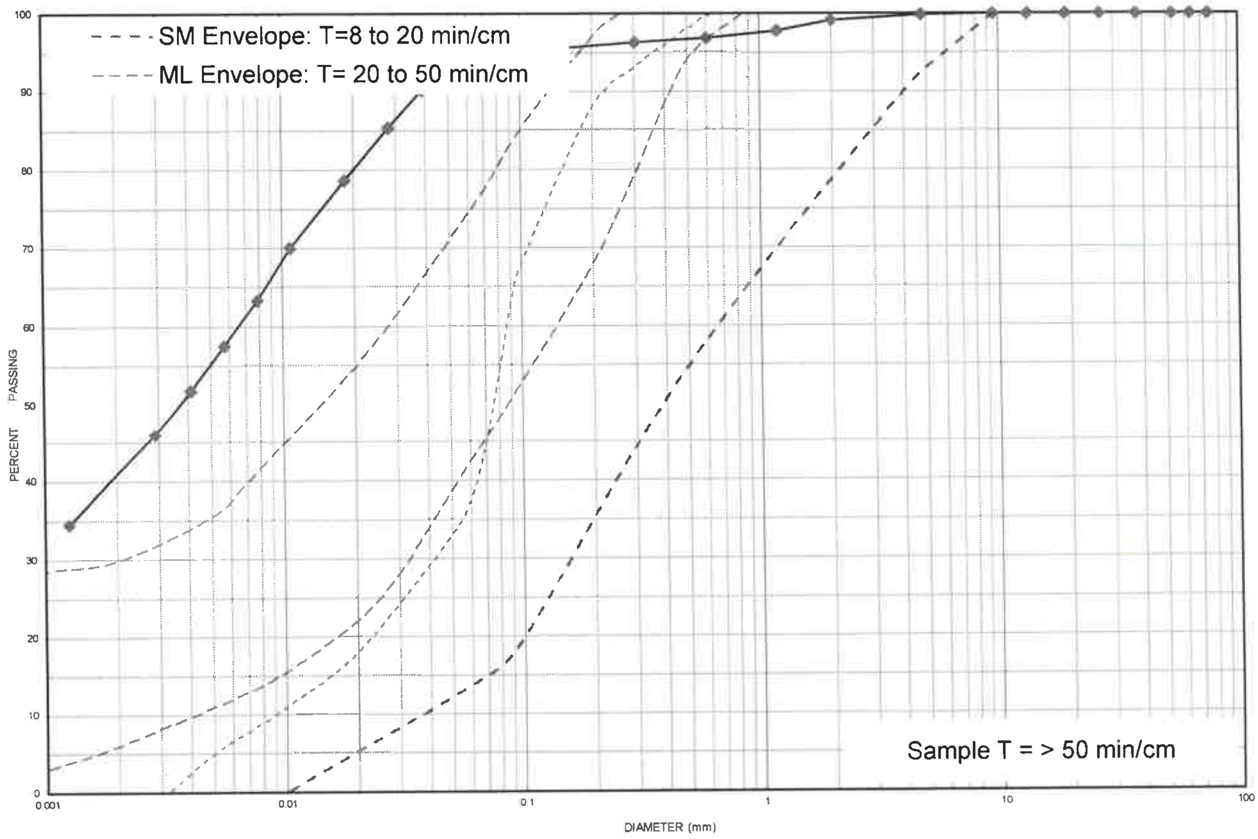
CAMBIUM

CCIL

Grain Size Distribution Chart

Project Number: *8679-001* **Client:** *Jason Redman*
Project Name: *1000 William Street, Midland, ON*
Sample Date: *February 27, 2019* **Sampled By:** *Alex Griffin - Cambium Inc.*
Hole No.: *TP 1 GS 2* **Depth:** *2.1 m* **Lab Sample No:** *S-19-0123*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 1	GS 2	2.1 m	0	5	95		42.6

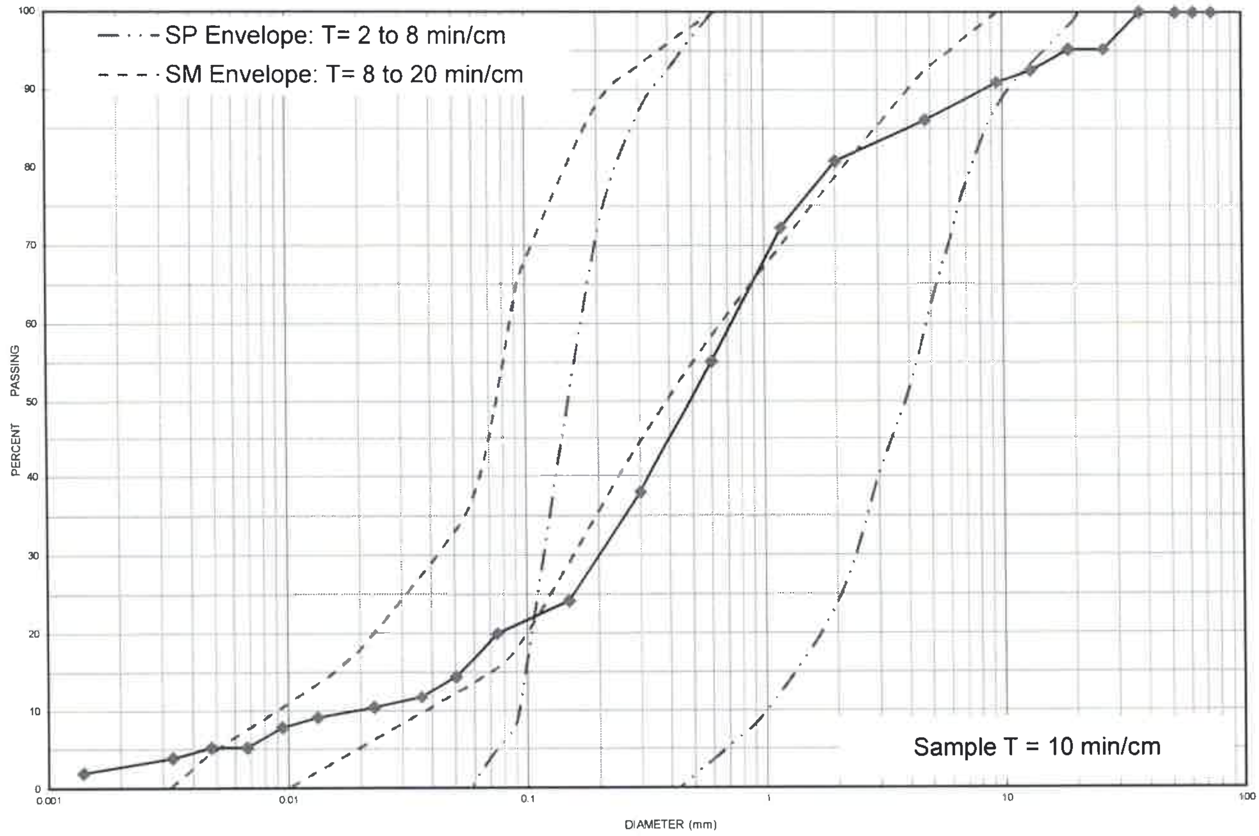
Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silt and Clay trace Sand	ML-CL	0.0066	-	-	-	-

Issued By: [signature] (Senior Project Manager) *Date Issued:* *March 15, 2019*

Grain Size Distribution Chart

Project Number: 8679-001 **Client:** Jason Redman
Project Name: 1000 William Street, Midland, ON
Sample Date: February 27, 2019 **Sampled By:** Alex Griffin - Cambium Inc.
Hole No.: TP 2 GS 2 **Depth:** 1.5 m **Lab Sample No:** S-19-0121

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 2	GS 2	1.5 m	14	66	20		11.5

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Silt some Gravel trace Clay	SW	0.720	0.200	0.019	37.89	2.92

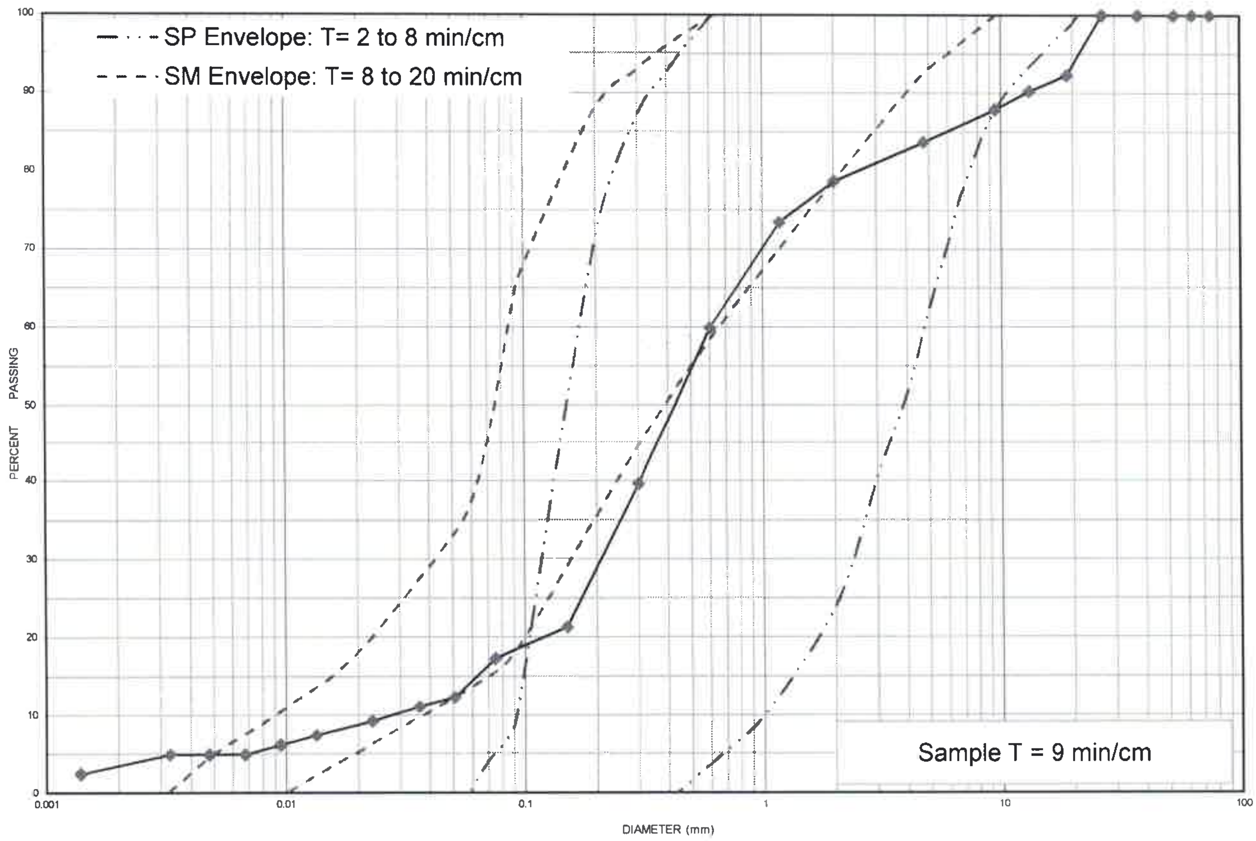
Issued By: (Senior Project Manager) *Date Issued:* March 15, 2019

Cambium Inc. (Laboratory)
866.217.7900 | cambium-inc.com
701 The Queensway | Units 5-6 | Peterborough | ON | K9J 7J6

Grain Size Distribution Chart

Project Number: 8679-001 **Client:** Jason Redman
Project Name: 1000 William Street, Midland, ON
Sample Date: February 27, 2019 **Sampled By:** Alex Griffin - Cambium Inc.
Hole No.: TP 3 GS 1 **Depth:** 0.3 m **Lab Sample No:** S-19-0122

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 3	GS 1	0.3 m	16	66	18		8.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Gravel some Silt trace Clay	SW	0.600	0.220	0.027	22.22	2.99

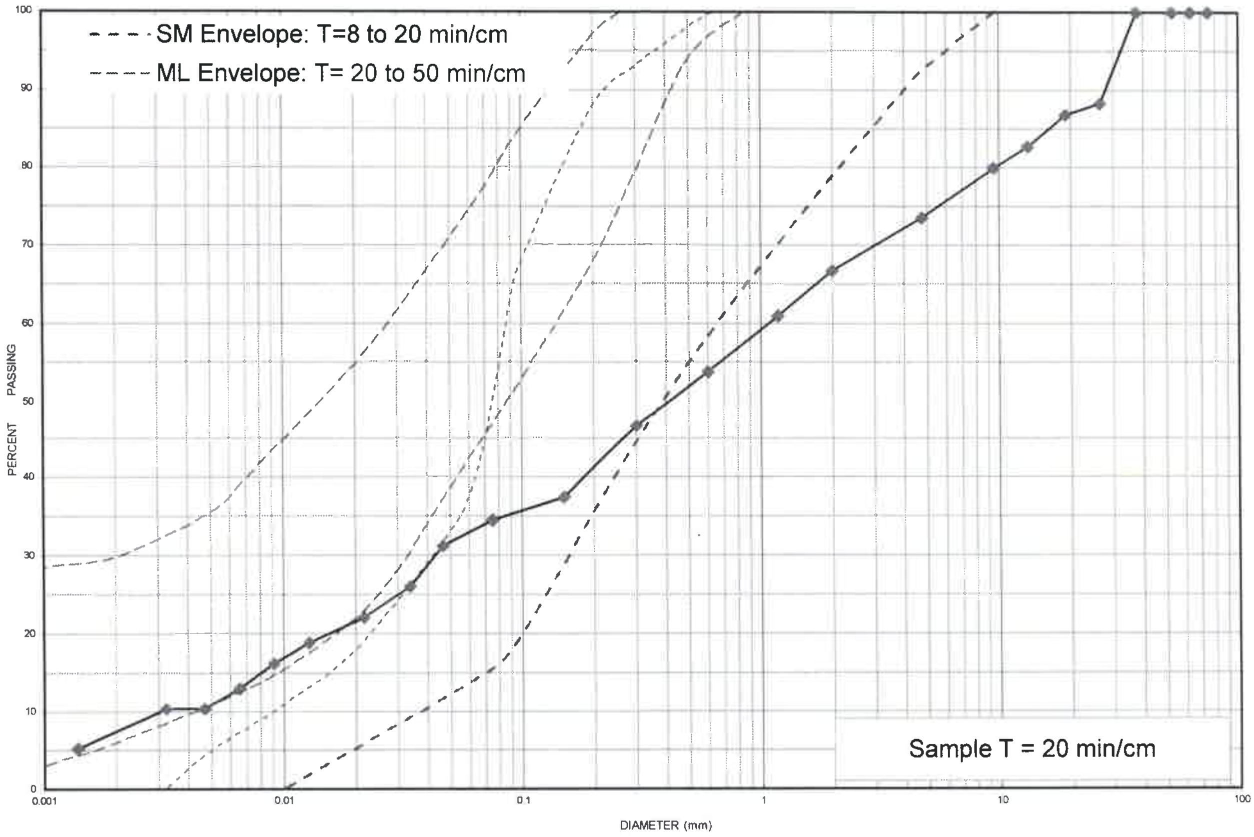
Issued By: (Senior Project Manager) *Date Issued:* March 15, 2019

CAMBIUM

Grain Size Distribution Chart

Project Number:	*8679-001*	**Client:**	*Jason Redman*		
Project Name:	*1000 William Street, Midland, ON*				
Sample Date:	*February 27, 2019*	**Sampled By:**	*Alex Griffin - Cambium Inc.*		
Hole No.:	*TP 5 GS 2*	**Depth:**	*1.8 m*	**Lab Sample No:**	*S-19-0123*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 5	GS 2	1.8 m	26	39	35		5.1

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Gravelly Silty Sand trace Clay	SP	1.100	0.044	0.003	366.67	0.59

Issued By: [signature] (Senior Project Manager) *Date Issued:* *March 15, 2019*

31D/12W

The Ontario Water Resources Commission Act

WATER WELL RECORD

Water management in Ontario

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

11 | 5707708 | MUNICIP 57013 | CON CON 03

COUNTY OR DISTRICT	TOWNSHIP, BOROUGH, CITY, TOWN, VILLAGE	CON., BLOCK, TRACT, SURVEY, ETC.	LOT
Simcoe	Tay (to Midland)	III	016

[redacted] Midland, Ont.

DATE COMPLETED: DAY 11 MO. 08 YR. 70

[redacted]53880 | RC 4 | ELEVATION 0600 | RC 5 | BASIN CODE 23

LOG OF OVERBURDEN AND BEDROCK MATERIALS (SEE INSTRUCTIONS)

GENERAL COLOUR	MOST COMMON MATERIAL	OTHER MATERIALS	GENERAL DESCRIPTION	DEPTH - FEET FROM	TO
dark	loam		Top Soil	0	1
yellow	sand			1	14
grey	sand			14	30
grey	clay		very soft	30	42
light yel.	sand	silt	fine	42	57
grey	sand		fine to medium	57	60
yellow	sand	silt, gravel		60	79
brown	limestone		shale	79	100
blue + brown	limestone		soft	100	136
grey red	granite		soft	136	139

APL

31 | 0001 02 | 0014509 | 0030209 | 0042205 | 0057509906 | 0060208 |
32 | 0079509906 | 1000415 | 1364315 | 0139731 |

41 WATER RECORD

WATER FOUND AT - FEET	KIND OF WATER
0081	☒ FRESH ☐ SALTY ☐ SULPHUR ☐ MINERAL
0090	☒ FRESH ☐ SALTY ☐ SULPHUR ☐ MINERAL
	☐ FRESH ☐ SALTY ☐ SULPHUR ☐ MINERAL
	☐ FRESH ☐ SALTY ☐ SULPHUR ☐ MINERAL
	☐ FRESH ☐ SALTY ☐ SULPHUR ☐ MINERAL

51 CASING & OPEN HOLE RECORD

INSIDE DIAM. INCHES	MATERIAL	WALL THICKNESS INCHES	DEPTH - FEET FROM	TO
6 (06)	☒ STEEL ☐ GALVANIZED ☐ CONCRETE ☐ OPEN HOLE	.188	0	81 (0081)
	☐ STEEL ☐ GALVANIZED ☐ CONCRETE ☒ OPEN HOLE		81	139 (0139)
	☐ STEEL ☐ GALVANIZED ☐ CONCRETE ☐ OPEN HOLE			

SCREEN: SIZE(S) OF OPENING (SLOT NO.) | DIAMETER INCHES | LENGTH FEET | MATERIAL AND TYPE | DEPTH TO TOP OF SCREEN FEET

61 PLUGGING & SEALING RECORD

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

71 PUMPING TEST

PUMPING TEST METHOD: ☒ PUMP ☐ BAILER | PUMPING RATE 0108 GPM | DURATION OF PUMPING 16 HOURS 00 MINS.

STATIC LEVEL	WATER LEVEL END OF PUMPING	15 MINUTES	30 MINUTES	45 MINUTES	60 MINUTES
016 (15')	045 FEET	018 (17') FEET	016 FEET	016 (15') FEET	016 (15') FEET

WATER LEVELS DURING: ☐ PUMPING ☒ RECOVERY

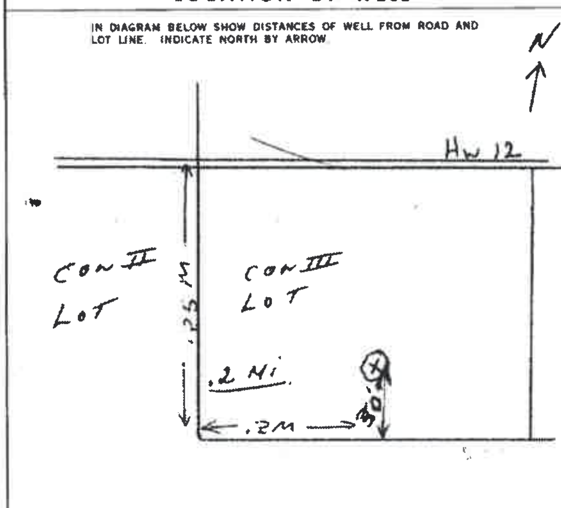
IF FLOWING, GIVE RATE ___ GPM | PUMP INTAKE SET AT ___ FEET | WATER AT END OF TEST: ☒ CLEAR ☐ CLOUDY

RECOMMENDED PUMP TYPE: ☐ SHALLOW ☒ DEEP | RECOMMENDED PUMP SETTING ___ FEET | RECOMMENDED PUMPING RATE ___ GPM

003.7 GPM/FT. SPECIFIC CAPACITY

LOCATION OF WELL

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



DRILLERS REMARKS:

FINAL STATUS OF WELL

☐ WATER SUPPLY ☐ OBSERVATION WELL ☒ TEST HOLE ☐ RECHARGE WELL ☐ ABANDONED, INSUFFICIENT SUPPLY ☐ ABANDONED, POOR QUALITY ☐ UNFINISHED

WATER USE 09

☐ DOMESTIC ☐ STOCK ☐ IRRIGATION ☐ INDUSTRIAL ☐ OTHER ☐ COMMERCIAL ☐ MUNICIPAL ☐ PUBLIC SUPPLY ☐ COOLING OR AIR CONDITIONING ☒ NOT USED

METHOD OF DRILLING

☒ CABLE TOOL ☐ ROTARY (CONVENTIONAL) ☐ ROTARY (REVERSE) ☐ ROTARY (AIR) ☐ AIR PERCUSSION ☐ BORING ☐ DIAMOND ☐ JETTING ☐ DRIVING

CONTRACTOR

NAME OF WELL CONTRACTOR: H. HAMMERS | LICENCE NUMBER 2514

ADDRESS: RR#3 Barrie, Ont.

NAME OF DRILLER OR BORER: A. Hammers | LICENCE NUMBER 2513

SIGNATURE OF CONTRACTOR: Henry Hammers | SUBMISSION DATE DAY ___ MO. ___ YR. ___

OFFICE USE ONLY

DATA SOURCE 1 | CONTRACTOR 2514 | DATE RECEIVED 111270

DATE OF INSPECTION | INSPECTOR P/L

REMARKS: CSS SS

OWRC COPY

Geotechnical Investigation Report 1000 William Street & 265 Whitfield Crescent, Midland, Ontario

Cambium Reference No.: 8679-001

July 4, 2019

Prepared for: Jason Redman

CAMBIUM

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Barrie, Ontario, L4N 5R7

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Facsimile: (705) 742.7907

cambium-inc.com

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1.0 INTRODUCTION

Cambium Inc. (Cambium) was retained by WMI & Associates on behalf of Jason Redman (Client) to complete a geotechnical investigation in support of the design and construction of a commercial storage development at 1000 William Street and an assessment of subsurface conditions at 265 Whitfield Crescent in Midland, Ontario (Site).

The William Street property is currently used as outdoor heavy equipment and construction materials storage, the lot is rectangular, relatively flat, and approximately 2.25 acres in size with fill noted across the center and eastern extents of the site, with the western extents appearing to have recently been stripped. The Whitfield Crescent property is currently vacant and undeveloped, the lot is rectangular, has rolling topography and is approximately 1 acre in size.

The proposed development at 1000 William Street consist of numerous 1-storey storage structures throughout the site, driving and parking areas, and storm water management features at the west and east ends of the site. At the time of investigation the development details of the 265 Whitfield Crescent site were understood to consist of a 1-storey office building, two 1-storey storage structures, driving and parking areas, outdoor storage areas, and a storm water management feature at the east end of the site. Following consultation with the Client, Cambium was directed that a test pit investigation was the Client's preferred method to sample and test the in-situ subsurface soils.

The geotechnical investigation was required to confirm the subsurface conditions at the Site in order to provide geotechnical design parameters as input into the design and construction of the proposed storage development. A Site Plan, including test pit locations, is included as Figure 1 of this report.

2.0 METHODOLOGY

2.1 TEST PIT INVESTIGATION

A test pit investigation was completed on February 27th, 2019, to assess subsurface conditions at the Site. A total of six (6) test pits, designated as TP101-19 through TP106-19, were advanced throughout each of the properties. All of the test pits were terminated at depths ranging from 1.8 m to 3.1 m below ground surface (mbgs). The test pit locations were selected and laid out in consultation with the Client. Test pits TP101-19 through TP104-19 were advanced throughout the William Street property, generally adjacent to proposed structures. Test pits TP105-19 and TP106-19 were advanced at the eastern and western ends of the Whitfield Crescent property to classify the native soils present at the site.

The test pit elevations and locations were surveyed by DEMTech Services. The test pit UTM's where surveyed by Cambium with a handheld Garmin etrex 20x and are provided in Table 4 and on the test pit logs, elevations are provided in Table 3 and on the test pit logs. Test pit locations are shown on Figure 1.

Test pits were advanced using a track mounted CAT 312 hydraulic excavator, equipped with a frost ripper and toothed bucket, provided by the client and supervised by a Cambium technician. Dynamic probe penetration tests (DPT), consisting of measuring the number of blows required to advance a 19 mm diameter steel rod into the subgrade soils a distance of 150 mm using an 8 kg hammer falling 750 mm, were attempted in each test pit to determine the in-situ density and bearing capacity of the subgrade soils.

The encountered soil units were logged in the field using visual and tactile methods, and samples were placed in labelled plastic bags for transport, future reference, possible laboratory testing, and storage.

Open test pits were checked for groundwater and general stability prior to backfilling. The test pits were backfilled with the excavated material, compacted with the bucket of the excavator, and the property was reinstated to as close to pre-existing conditions as possible.

Test pit logs are provided in Appendix A. Site soil and groundwater conditions are described and geotechnical recommendations are discussed in the following sections of this report.

2.2 PHYSICAL LABORATORY TESTING

Physical laboratory testing, including four (4) sieve and hydrometer analyses (LS-702, 705), was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters. Natural moisture content testing (LS-701) was completed on all retrieved soil samples. Results are presented in Appendix B and are discussed in Section 3.0.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site consist predominantly of topsoil or fill soils overlying clayey silt or till soils predominantly grading from a sandy silt to silt matrix. These soils were encountered throughout the test pits to the termination depths ranging from 1.5 mbgs to 3.1 mbgs. A layer of fill soil consisting of either sandy soils or clayey silt soils was noted at the surface of each of the test pit locations within the William Street property, the fill soils generally extended to depths between 0.8 mbgs and 1.5 mbgs. It should be noted that organic soils were encountered below the fill soils in test pits TP103-19 and TP104-19. All the test pits were terminated in native soils, and bedrock was not encountered within the excavation depths.

The test pit locations are shown on Figure 1 and the individual soil units are described in detail below with test pit logs provided in Appendix A. A summary of the depth of imported fill and topsoil is provided in Table 1 as an overview, with further descriptions provided below.

Table 1 Summary of Depths of Fill and Topsoil Across Site

Test Pit	Depth of Imported Fill (mbgs)	Depth of Organics (mbgs)	Description of Organics
TP101-19	0 – 1.5	-	-
TP102-19	0 – 1.5	-	-
TP103-19	0 – 0.8	0.8 – 1.1	Topsoil
TP104-19	0 – 0.9	0.9 – 1.2	Topsoil
TP105-19	-	0 – 0.6	Topsoil
TP106-19	-	0 – 0.3	Topsoil

3.1 TOPSOIL

A layer of black to brown topsoil between 300 mm and 600 mm in thickness was encountered at the surface of test pits TP105-19 and TP106-19 advanced at 265 Whitfield Crescent. The topsoil was frozen at the time of the investigation and loose in relative density. Black topsoil with some rootlets and organics was also noted beneath the fill soils in TP103-19 and TP104-19; in both test pits the topsoil was observed to be approximately 300 mm thick.

3.2 FILL SOILS

A layer of fill soils was observed at the surface of test pits TP101-19 through TP104-19 on the William Street property, and was generally brown sand with some gravel and silt, trace clay and occasional cobble, the exception being TP104-19 where the fill was predominately brown clayey silt, trace sand and likely reworked native soils. The fill extended to depths between 0.8 mbgs and 1.5 mbgs, and is summarized in Table 1. Based on visual inspection and observations during excavations the soils were noted as loose to compact in relative density with a natural moisture content ranging between 4% and 13%.

Laboratory particle size distribution analyses were completed for two (2) samples of the fill soils, taken from the test pits and depths provided in Table 2 in order to identify the varying textures encountered throughout the fill material. The testing results are provided in Appendix B and are summarized in Table 2 based on the Unified Soils Classification System (USCS).

Table 2 Particle Size Distribution – Fill Soils

TP	Depth (mbgs)	Description	% Gravel	% Sand	% Silt	% Clay
TP102-19	1.5	Sand some Silt some Gravel trace Clay	14	66	17	3
TP103-19	0.3	Sand some Gravel some Silt trace Clay	16	66	14	4

3.3 NATIVE SOILS

Beneath the fill soils discussed above, the native soils consisted glaciofluvial ice-contact deposits generally consisting of till material with varying amounts of silt and sand throughout the test pit locations, which extended to the termination depths ranging from 1.8 mbgs to 3.1 mbgs.

The texture of the native soils varied at each property. At 1000 William Street the native soils encountered was predominantly brown clayey silt, with trace sand. The DPT penetration resistances indicated a firm to very stiff consistency. Based on laboratory testing, the natural moisture content ranged between 16% and 38%. All of the test pits located in this property were terminated in the native clayey silt soils.

At 265 Whitfield Crescent, the native soils were predominately brown silty gravelly sand with trace clay inferred as a till material. Based on the DPT penetration resistances this material had a compact to very dense relative density with natural moisture content between 5% and 6%. Both test pits TP105-19 and TP106-19 were terminated in the native silty gravelly sand.

Laboratory particle size distribution analyses were completed for two (2) samples of the native soils, taken from the test pits and depths provided in Table 3 in order to identify the varying textures encountered throughout the overburden material. The testing results are provided in Appendix B and are summarized in Table 3 based on the USCS.

Table 3 Particle Size Distribution – Native Soils

TP	Depth (mbgs)	Description	% Gravel	% Sand	% Silt	% Clay
TP101-19	2.1	Silt and Clay trace Sand	0	5	54	41
TP105-19	1.8	Gravelly Silty Sand trace Clay	26	39	28	7

3.4 BEDROCK

Bedrock was not encountered within the investigation depths. Each of the test pits were terminated at depths ranging from 1.8 mbgs to 3.1 mbgs generally in native soils, the exception being TP102-19 which was terminated in fill soils at 1.5 mbgs. The elevation of each test pit and their respective termination depths are identified in Table 4 below.

Table 4 Test Pit Termination Depth – Elevations

Test Pit ID	Test Pit Elevation (mASL)	Test Pit Termination Depth (mbgs)	Test Pit Termination Elevation (mASL)
TP101-19	187.31	2.4	184.91
TP102-19	186.51	2.1	184.41
TP103-19	186.42	3.1	183.32
TP104-19	187.12	3.1	184.02
TP105-19	**	1.8	**
TP106-19	**	1.8	**

***Test pits not surveyed by DEMTech*

3.5 GROUNDWATER

Groundwater (free water) was noted in test pits TP101-19, TP102-19 and TP103-19. The observed groundwater elevation and caving (sloughing) depths are summarised in Table 5. Given the presence of predominately granular fill overlying low permeable clayey silt along the central and western extents of 1000 William Street, it is possible that observed groundwater may be perched seepage in this area.

The moisture content of the soils generally ranged from 3% to 43%. It should be noted that soil moisture and groundwater levels at the Site may fluctuate seasonally and in response to climatic events.

Table 5 Ground Water and Caving Observations

Test Pit ID	Test Pit Elevation (mASL)	Depth to Groundwater (mbgs)	Ground Water Elevation (mASL)	Caving Depth (mbgs)
TP101-19	187.31	1.2	186.11	0.9
TP102-19	186.51	1.3	185.21	1.2
TP103-19	186.42	1.5	184.92	-
TP104-19	187.12	-	-	-
TP105-19	**	-	-	-
TP106-19	**	-	-	-

***Test pits not surveyed by DEMTech*

3.6 INFILTRATION TESTING

In order to help determine the infiltration rates, four (4) particle size distribution tests (hydrometer analyses) were completed on samples as described in Section 3.2. In order to determine the rate at which water will be absorbed into the soil ("T" time), the soil was classified according to the USCS and the T Time was interpolated based on the USCS gradation charts for the two particle size distribution tests (hydrometer analyses) described in Section 3.2 and 3.3 of this report. The hydraulic conductivity was calculated based on the Puckett equation. The results are summarised in Tables 6, 7 and 8 and the T time is included on the grain size distribution charts in Appendix B.

Table 6 Infiltration Results – Fill Soils

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP102-19	1.8	10 mins/cm	Silty Sand (SM)	$2.4x10^{-5}$ m/s
TP103-19	0.3	9 mins/cm	Silty Sand (SM)	$2.0x10^{-5}$ m/s

Table 7 Infiltration Results – Native Soils (1000 William Street)

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP101-19	2.1	> 50 mins/cm	Silt (ML)	$1.3x10^{-8}$ m/s

Table 8 Infiltration Results – Native Soils (265 Whitfield Crescent)

Test ID	Sample Depth (mbgs)	Percolation Time (T-time)	USCS Soil Type	Hydraulic Conductivity (K)
TP105-19	1.8	20 mins/cm	Silt (ML)	$1.1x10^{-5}$ m/s

Based on these test results we believe a percolation time of 10 mins/cm is appropriate for the gravelly sand fill soils, 20 mins/cm for the gravelly silty sand at 265 Whitfield Crescent and > 50 mins/cm for the silt soils at 1000 William Street.

4.0 GEOTECHNICAL CONSIDERATIONS

The following recommendations are based on test pit information and are intended to assist designers. Recommendations should not be construed as providing instructions to contractors, who should form their own opinions about site conditions. It is possible that subsurface conditions beyond the test pit locations may vary from those observed. If significant variations are found before or during construction, Cambium should be contacted so that we can reassess our findings, if necessary.

4.1 SITE PREPARATION

The existing fill material and any organic materials encountered should be excavated and removed from beneath any structures which will be occupied (i.e., offices, maintenance buildings, residential, etc.); additionally this material should be excavated and removed to a minimum distance of 3 m around the proposed occupied building footprint. The fill material may potentially be left in place beneath the single storey storage units and driving areas, however an additional test pitting program is recommended to confirm that the site was stripped prior to the placement of existing fill and/or delineate the extent of the organics at 1000 William Street, as organics and topsoil were noted in TP103-19 and TP104-19. The fill material includes, but is not limited to the fill identified in this report. Any topsoil and materials with significant quantities of organics and deleterious materials (i.e., construction debris, asphalt etc.) are not appropriate for use as fill below storage units and driving areas.

The exposed subgrade should be proof-rolled and inspected by a qualified geotechnical engineer prior to placement of granular fill or foundations. Any loose/soft soils identified at the time of proof-rolling that are unable to uniformly be compacted should be sub-excavated and removed. The excavations created through the removal of these materials should be backfilled with approved engineered fill consistent with the recommendations provided below. Additionally the test pit locations summarized below in Table 9 should be excavated to the termination depths provided in Table 4 and reinstated with approved engineered fill should they be situated beneath any load bearing structural elements (i.e., footings).

The near surface sand and silt soils can be very unstable if they are wet or saturated. Such conditions are common in the spring and late fall. Under these conditions, temporary use of granular fill, and possible reinforcing geotextiles, may be required to prevent severe rutting on construction access routes.

Table 9 Test Pit UTM Coordinates

Test Pit ID	UTM Zone	UTM Northing	UTM Easting
TP101-19***	17 T	590548	4953893
TP102-19***	17 T	590557	4953975
TP103-19***	17 T	590696	4953893
TP104-19***	17 T	590557	4953975
TP105-19	17 T	590408	4953928
TP106-19	17 T	590359	4953882

****Test pit locations also provided in DEMTech Topographic Survey*

4.2 FROST PENETRATION

Based on climate data and design charts, the maximum frost penetration depth below the surface at the site is estimated at 1.6 mbgs.

If strip and spread foundations are to be used, exterior footings for the proposed structures should be situated at or below this depth for frost penetration or should be adequately insulated.

It is assumed that the pavement structure thickness will be less than 1.6 m, so grading and drainage are important for good pavement performance and life expectancy. Any services should be located below this depth or be appropriately insulated.

4.3 EXCAVATIONS AND BACKFILL

All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). The generally loose to compact fill and native soils may be classified as Type 3 soils above the groundwater table in accordance with OHSA. Type 3 soils may be excavated with side slopes no steeper than 1H:1V. Below the groundwater table the soils may be classified as Type 4 soils and may be excavated with unsupported side slopes no steeper than 3H:1V.

4.4 DEWATERING

Groundwater was encountered in three (3) of the six (6) test pits at TP101-19, TP102-19 and TP103-19 at depths ranging from 1.2 mbgs to 1.5 mbgs, given the presence of predominately granular fill overlying low permeable clayey silt in this area, it is possible that observed groundwater may be perched seepage. Seepage may occur across the Site if high groundwater conditions are present during construction due to seasonal fluctuations. If groundwater seepage is encountered it should be manageable with filtered sumps and pumps and depending on size of excavation, a Permit to Take Water (PTTW) from the Ministry of the Environment and Climate Change (MOECC) will likely not be required. It is noted that the elevation of the groundwater table will vary due to seasonal conditions and in response to heavy precipitation events. In order to minimize predictable water issues and costs, it is recommended that excavation and in-ground construction be performed in drier seasons.

4.5 BACKFILL AND COMPACTION

Excavated topsoil from the Site is not appropriate for use as fill below grading and parking areas. Excavated sand soils not containing organics, may be appropriate for use as fill below grading and parking areas, provided that the actual or adjusted moisture content at the time of construction is within a range that permits compaction to required densities, and that the material is only used below frost penetration depth of 1.6 m below proposed grade. Some moisture content adjustments may be required depending upon seasonal conditions. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality.

Any engineered fill below foundations should be placed in lifts appropriate to the type of compaction equipment used, and be compacted to a minimum of 100% of standard Proctor maximum dry density (SPMDD), as confirmed by nuclear densometer testing. If native soils from the site are not used as engineered fill, imported material for engineered fill should consist of clean, non-organic soils, free of chemical contamination or deleterious material. The moisture content of the engineered fill will need to be close enough to optimum at the time of placement to allow for adequate compaction. Consideration could be given to using a material meeting the specifications of OPSS 1010 Granular B or an approved equivalent. Foundation wall and any buried utility backfill material should consist of free-draining imported granular material. Most of the native site soils are too fine-grained to provide proper drainage, and as such this should be accomplished using well graded Granular B Type 1 material complying with OPSS 1010.

The backfill material, if any, in the upper 300 mm below the pavement subgrade elevation should be compacted to 100 percent of SPMDD in all areas.

4.6 FOUNDATION DESIGN

We understand that the proposed development at 1000 William Street consists of multiple one-storey self-storage units, all with which will be constructed without basements. At the time of investigation, the proposed development plans for 265 Whitfield Crescent consists three (3) one-storey structures which includes one office/maintenance building and two self-storage units, all with which will be constructed without basements. Assuming that the site is prepared as outlined above, the native sub-soils are competent to support all structures on either conventional strip and spread footings or frost protected reinforced raft foundations.

4.6.1 STRIP AND SPREAD FOOTINGS

Assuming any new exterior footings will be placed a minimum of 1.6 m below final adjacent grade for frost protection, these footings can be founded on compact clayey silt or till soils at depth. Any required grade raises to the footing elevations can be accomplished with engineered fill, using an OPSS 1010 SSM or Granular 'B' Type I granular material in 200 mm lifts and compacted to a minimum of 100% of Standard Proctor Maximum Dry Density (SSPMD) as specified above. New footings situated at a minimum depth of 1.6 m below the final adjacent grade, founded in undisturbed compact native clayey silt or till may be designed for an allowable bearing capacity of 100 kPa at serviceability limit state (SLS) and 145 kPa at ultimate limit state (ULS) in all areas.

4.6.2 FROST PROTECTED REINFORCED RAFT FOUNDATION

In addition to the strip and spread footings recommendations above, the storage units may be constructed on frost protected reinforced raft foundations found on either native soils or potentially compact fill soils overlying native inorganic clayey silt subject to the approval by Cambium. Storage units constructed on raft foundations, founded in approved compact fill soils may be designed for an allowable bearing capacity of 50 kPa at SLS and 70 kPa at ULS in all areas. It is noted that topsoil and organics was noted between the fill and inorganic soils in test pits TP103-19 and TP104-19, as such further test pits are recommend prior to construction in order to delineate the underlying topsoil extents. Raft foundations may also be suitable for the proposed office/maintenance building, however given that it would be classified as an occupied structure, it will need to be found on either native soils or approved engineered fill placed and compacted on inorganic soils per Section 4.5.

The quality of the subgrade should be inspected by Cambium during construction, prior to constructing the footings, to confirm bearing capacity estimates and suitability of fill. Settlement potential at the above-noted SLS loadings is less than 25 mm and differential settlement should be less than 10 mm.

4.7 FLOOR SLABS

To create a stable working surface, to distribute loadings, and for drainage purposes, an allowance should be made to provide at least 200 mm of OPSS 1010 Granular A compacted to 98% of SPMDD beneath all floor slabs.

4.8 SUBDRAINAGE

Perimeter subdrains will not be required for structures built on reinforced, raft foundations. Given the investigation was limited to termination depths varying between 1.5 and 3.1 mbgs, if the groundwater table is encountered during excavation for strip footings, geotextile wrapped subdrains set in a trench of clear stone and connected to a sump or other frost-free positive outlet would be recommended around the perimeter of the building foundations.

4.9 BURIED UTILITIES

Trench excavations above the groundwater table should generally consider Type 3 soil conditions, which require side slopes no steeper than 1H:1V, otherwise shoring would be required. Any excavations below the water table should generally consider Type 4 soil conditions which require side slopes of 3H:1V or flatter. Bedding and cover material for any services should consist of OPSS 1010-3 Granular A or B Type II, placed in accordance with pertinent Ontario Provincial Standard Drawings (OPSD 802.013). The bedding and cover material shall be placed in maximum 200 mm thick lifts and should be compacted to at least 98 percent of SPMDD. The cover material shall be a minimum of 300 mm over the top of the pipe and compacted to 98 percent of SPMDD, taking care not to damage the utility pipes during compaction.

4.10 PAVEMENT DESIGN

The performance of the pavement is dependent upon proper drainage and subgrade preparation. All topsoil and organic materials should be removed down to native material and backfilled with approved engineered fill or native material, compacted to 98 percent SPMDD. The subgrade should be proof rolled and inspected by a Geotechnical Engineer. Any areas where boulders, rutting, or appreciable deflection is noted should be subexcavated and replaced with suitable fill. The fill should be compacted to at least 98 percent SPMDD.

From discussions with the client, it is understood that the preference is to have gravel surfaced driving and parking areas throughout the Whitfield Crescent and William Street properties. The recommended pavement structure should meet the Ministry standards for parking and driving areas and should, as a minimum, consist of the pavement layers identified in Table 10. The light duty pavement structure is intended for parking areas while the heavy duty pavement structure is appropriate for fire access routes. If the recommended minimum pavement structure in Table 10 is different from that specified by the Town of Midland, it is up to the discretion of the Design Engineer to decide which pavement structure to use.

Table 10 Recommended Minimum Pavement Structure

Pavement Layer	Light Duty	Heavy Duty
Granular Surface	100 mm OPSS 1010 Granular M or Granular S	100 mm OPSS 1010 Granular M or Granular S
Granular Base	200 mm OPSS 1010 Granular A	300 mm OPSS 1010 Granular A

Material and thickness substitutions must be approved by the Design Engineer. The thickness of the base layer could be increased at the discretion of the Engineer, to accommodate site conditions at the time of construction, including soft or weak subgrade soil replacement.

Compaction of the subgrade should be verified by the Engineer prior to placing the granular fill. Granular layers should be placed in 200 mm maximum loose lifts and compacted to at least 98% of SPMDD (ASTM D698) standard. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing.

Drainage features such as subdrains beneath the pavement structure, connecting to the storm sewer or an alternate frost-free outlet, or other drainage alternatives left to the discretion of the designer are recommended to extend the lifespan of the pavement structure.

The final granular surface should be sloped at a minimum of 2 percent to shed runoff, and regular maintenance of the granular surface should be performed to ensure it remains free of surficial deformations.

4.11 DESIGN REVIEW AND INSPECTIONS

Cambium should be retained to complete testing and inspections during construction operations to examine and approve subgrade conditions, placement and compaction of fill materials, granular base courses, and asphaltic concrete.

We should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed. It is important that onsite geotechnical supervision be provided at this site for excavation and backfill procedures, deleterious soil removal, subgrade inspections and compaction testing.

5.0 CLOSING

We trust that the information contained in this report meets your current requirements. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at (705) 719-0700.

Respectfully submitted,

CAMBIUM INC.

LICENSED PROFESSIONAL ENGINEER
R. L. GETHIN
100196175
JULY 4, 2019
PROVINCE OF ONTARIO

Rob Gethin, P.Eng.
Senior Project Manager

RLG/jb

\\camfile\Projects\8600 to 8699\8679-001 Jason Redman - Geotechnical Investigation - #1000 William Street, Midland, ON\Deliverables\REPORT - Geotechnical\Final\2019-07-04 RPT 1000 William & 265 Whitfield Geotech.docx

Appended Figures

GEOTECHNICAL INVESTIGATION

JASON REDMAN
1000 William Street and
265 Whitfield Crescent
Midland, Ontario

LEGEND

Testpit Locations

Subject Property(approx.)

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2017 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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TEST PIT LOCATION PLAN

Project No.: 8679-001	Date: March 2019 Rev.:
Scale: 1:2,000	Projection: NAD 1983 UTM Zone 17N

Created by: SH	Checked by: RG	Figure: 1

O:\GIS\project_MXDs\8600 to 8699\8679-001 Jason Redman - Geotechnical Investigation - #1000 William Street, Midland, ON\2019-03-04 FIG 1 Test Pit Location Plan.mxd

Appendix A
Test Pit Logs

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

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Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP101-19	0 - 1.5	GS1		Brown sand, some gravel, some silt, trace clay, occasional cobble, frozen to 0.6 mbgs, moist, saturated at 1.2 mbgs, loose to compact, FILL	0.61 - 0.76	4
					0.76 - 0.91	13
17T, 590548, 4953893	1.5 - 2.4	GS2		Dark brown to grey clayey silt, trace sand, wet, firm to stiff	0.91 - 1.10	20
				Caving (sloughing) of test pit walls at 0.9 mbgs and seepage noted at 1.2 mbgs	1.10 - 1.22	13
				Test pit terminated at 2.4 mbgs	1.22 - 1.37	8
					1.37 - 1.52	8
				GSA GS2 (2.1 mbgs): 0% Gravel, 5% Sand, 54% Silt, 41% Clay	1.52 - 1.67	5
					1.67 - 1.83	5
					1.52 - 1.67	2
					1.67 - 1.83	3
					1.83 1.98	7
					1.98 2.13	9
					2.13 2.29	12
					2.29 2.44	15
					2.44 2.59	19
					2.59 2.74	21

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP102-19	0 - 1.5	GS1/GS2		Brown sand, some gravel, some silt, trace clay, occasional cobble, frozen to 0.9 mbgs, moist, saturated at 1.35, loose to compact, FILL		
17T, 590557, 4953975	1.5			Grey clayey silt, trace sand, wet, firm to stiff		
				Caving (sloughing) of test pit walls at 1.2 mbgs and seepage noted at 1.3 mbgs		
				Test pit terminated at 1.5 mbgs due to unstable excavation		
				GSA GS2 (1.5 mbgs): 14% Gravel, 66% Sand, 17% Silt, 3% Clay		

[1]: metres below ground surface

[2]: Dynamic Penetration Test

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP103-19	0 - 0.8	GS1		Brown silty sand, some gravel, trace clay, occasional cobble, frozen, compact, FILL	1.52 - 1.67	5
	0.8 - 1.1	GS2		Black sandy silty topsoil, some rootlets and organics, frozen	1.67 - 1.83	5
17T, 590696,	1.1 - 3.1	GS3/GS4		Brown clayey silt, trace sand, moist to wet, firm to stiff	1.83 1.98	5
4953893				Test pit open upon completion, seepage noted at 1.5 mbgs	1.98 2.13	6
				Test pit terminated at 3.1 mbgs	2.13 2.29	7
					2.29 2.44	6
				GSA GS1 (0.3 mbgs): 16% Gravel, 66% Sand, 15% Silt, 3% Clay	2.44 2.59	6
					2.59 2.74	6

Test Pit ID	Depth (mbgs[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP104-19	0 - 0.9	GS1		Brown clayey silt, trace sand, frozen to 0.91 mbgs, firm, FILL	1.22 - 1.37	2
	0.9 -1.2	GS2		Black sandy silty topsoil, some rootlets and organics, moist, loose	1.37 - 1.52	8
17T, 590557,	1.2 - 3.1	GS3/GS4		Brown clayey silt, trace sand, moist, firm to stiff	1.52 - 1.67	7
4953975				Test pit open and dry upon completion	1.67 - 1.83	8
				Test pit terminated at 3.05 mbgs	1.83 - 1.98	7
					1.98 - 2.13	18
					2.13 - 2.29	30
					2.29 - 2.44	15

[1]: metres below ground surface

[2]: Dynamic Penetration Test

TABLE 1: TEST PIT LOGS

Geotechnical Investigation: 1000 William Street & 265 Whitfield Crescent, Midland, ON

Technician: A. Griffin

Cambium Reference No. 8679-001

Completed February 28th, 2019

Test Pit ID	Depth (mbgl[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP105-19 17T, 590408, 4953928	0 - 0.6 0.6 -1.8	 GS1/GS2		Black sandy silty topsoil, some rootlets and organics, frozen to 0.6 mbgs Brown silty gravelly sand, some cobbles, trace clay, moist, dense to very dense Grey at 1.8 mbgs Test pit open and dry upon completion Test pit terminated at 1.8 mbgs due to refusal on very dense gravel GSA GS2 (1.8 mbgs) : 26% Gravel, 39% Sand, 28% Silt, 7% Clay	1.22 - 1.37 1.37 - 1.52 1.52 - 1.67	2 30 30 = 125mm

Test Pit ID	Depth (mbgl[1])	Soil Sample	Moisture Content (%)	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP106-19 17T, 590359, 4953882	0 - 0.3 0.3 - 1.8	 GS1/GS2		Black sandy silty topsoil, some rootlets and organics, frozen to 0.3 mbgs Brown silty gravelly sand, some cobbles, trace clay, moist, dense to very dense Grey at 1.8 mbgs Test pit open and dry upon completion Test pit terminated at 1.8 mbgs due to refusal on very dense gravel	1.22 - 1.37 1.37 - 1.52 1.52 - 1.67 1.67 - 1.83 1.83 - 1.98 1.98 - 2.13	13 15 17 24 24 30 = 125mm

[1]: metres below ground surface

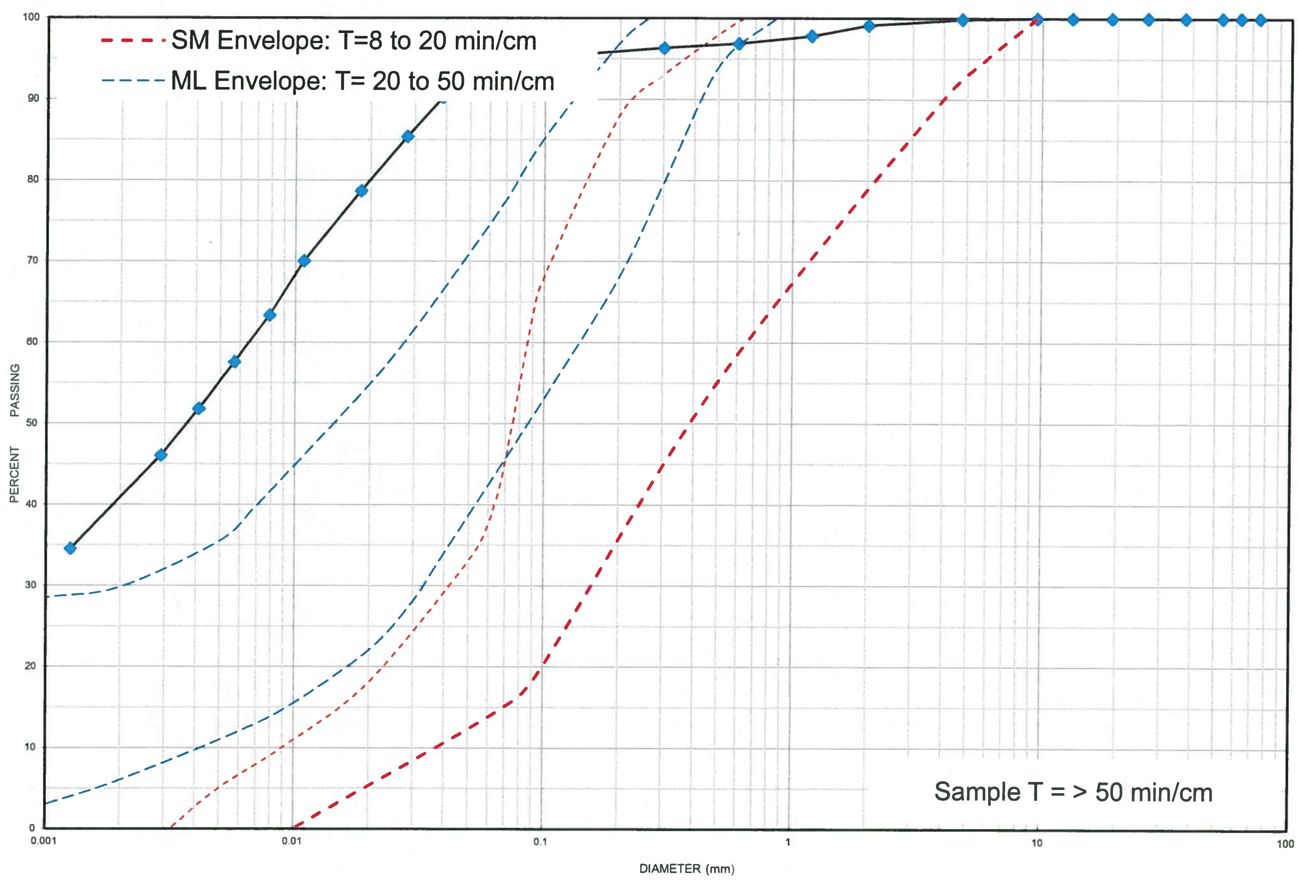
[2]: Dynamic Penetration Test

Appendix B
Physical Laboratory Testing Results

Grain Size Distribution Chart

Project Number:	*8679-001*	**Client:**	*Jason Redman*
Project Name:	*1000 William Street, Midland, ON*		
Sample Date:	*February 27, 2019*	**Sampled By:**	*Alex Griffin - Cambium Inc.*
Hole No.:	*TP 1 GS 2*	**Depth:**	*2.1 m*
Lab Sample No:	*S-19-0123*		

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 1	GS 2	2.1 m	0	5	95		42.6

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silt and Clay trace Sand	ML-CL	0.0066	-	-	-	-

Issued By: (Senior Project Manager)

Date Issued: *March 15, 2019*

Cambium Inc. (Laboratory)
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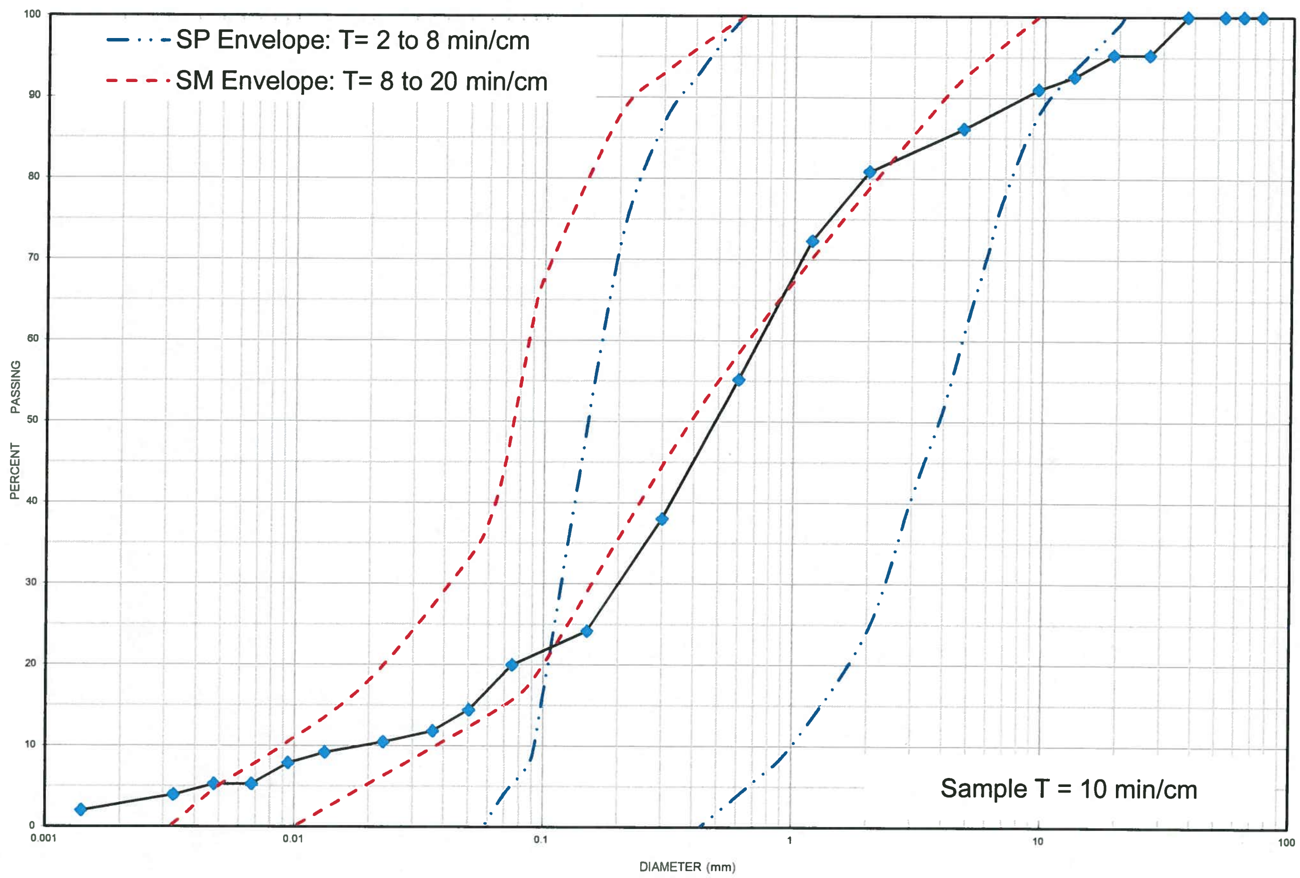
701 The Queensway | Units 5-6 | Peterborough | ON | K9J 7J6

Grain Size Distribution Chart

Project Number:	*8679-001*	**Client:**	*Jason Redman*		
Project Name:	*1000 William Street, Midland, ON*				
Sample Date:	*February 27, 2019*	**Sampled By:**	*Alex Griffin - Cambium Inc.*		
Hole No.:	*TP 2 GS 2*	**Depth:**	*1.5 m*	**Lab Sample No:**	*S-19-0121*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 2	GS 2	1.5 m	14	66	20		11.5

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Silt some Gravel trace Clay	SW	0.720	0.200	0.019	37.89	2.92

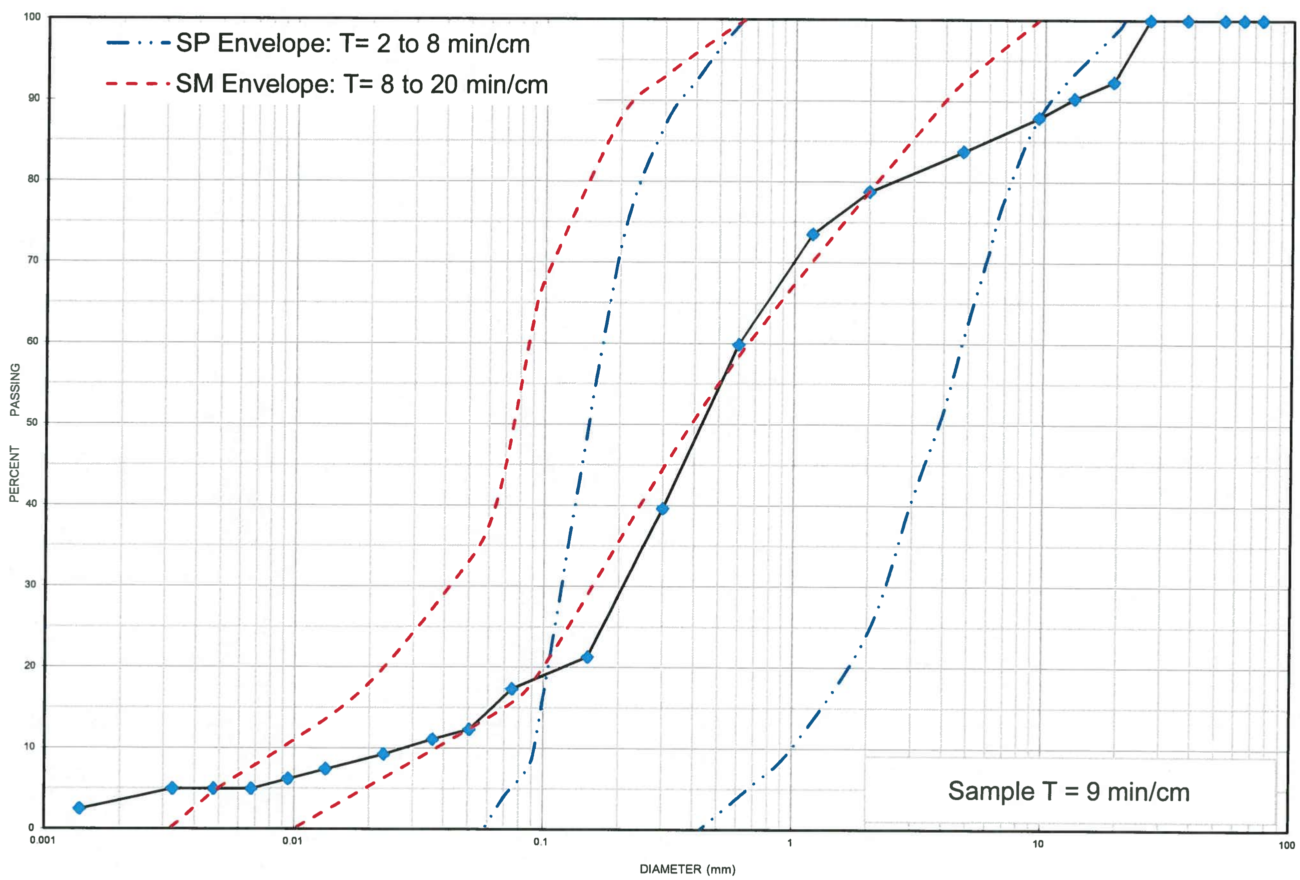
Issued By: (Senior Project Manager) *Date Issued:* *March 15, 2019*

Grain Size Distribution Chart

CAMBIUM

Project Number:	*8679-001*	**Client:**	*Jason Redman*		
Project Name:	*1000 William Street, Midland, ON*				
Sample Date:	*February 27, 2019*	**Sampled By:**	*Alex Griffin - Cambium Inc.*		
Hole No.:	*TP 3 GS 1*	**Depth:**	*0.3 m*	**Lab Sample No:**	*S-19-0122*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 3	GS 1	0.3 m	16	66	18		8.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand some Gravel some Silt trace Clay	SW	0.600	0.220	0.027	22.22	2.99

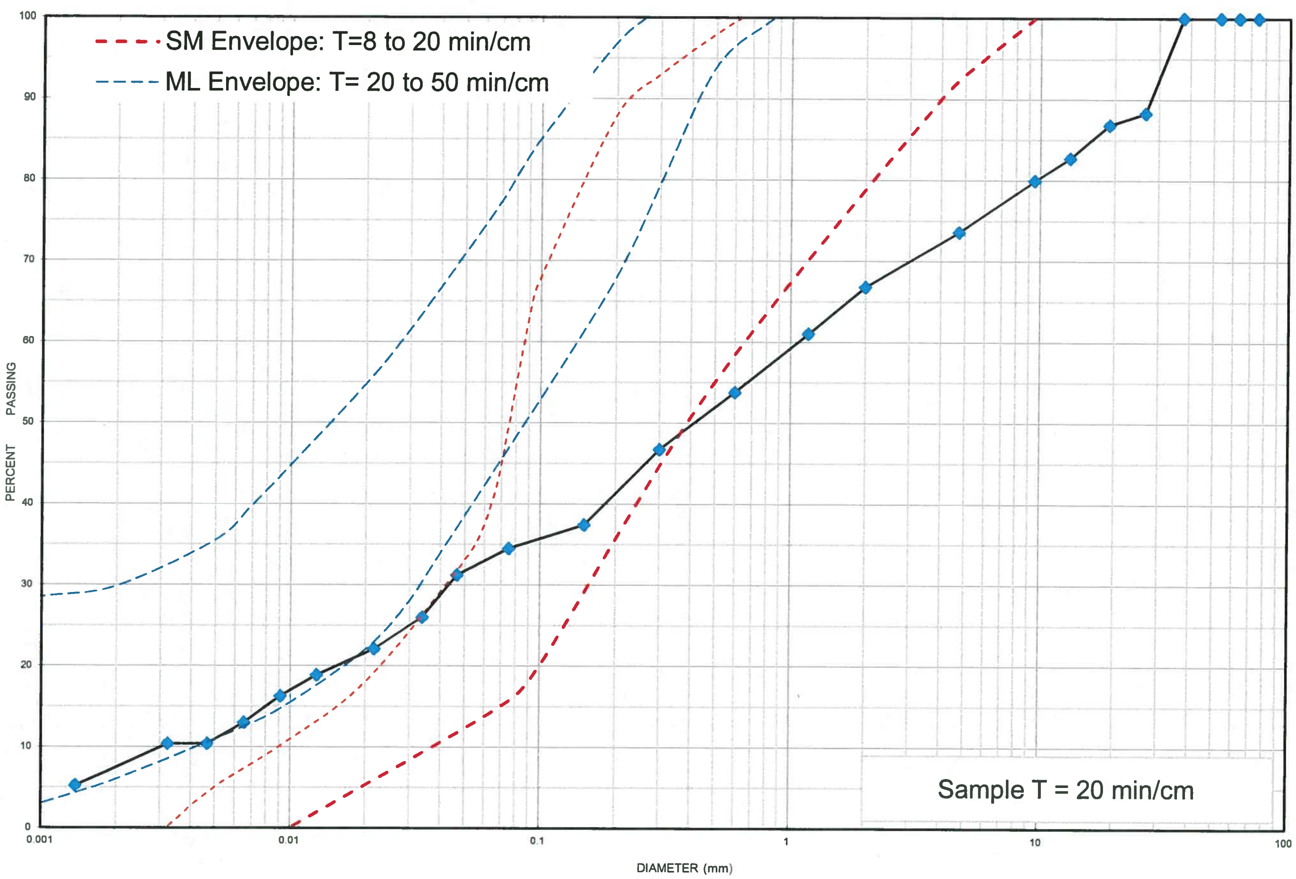
Issued By: [signature] (Senior Project Manager)

Date Issued: *March 15, 2019*

Grain Size Distribution Chart

Project Number:	*8679-001*	**Client:**	*Jason Redman*		
Project Name:	*1000 William Street, Midland, ON*				
Sample Date:	*February 27, 2019*	**Sampled By:**	*Alex Griffin - Cambium Inc.*		
Hole No.:	*TP 5 GS 2*	**Depth:**	*1.8 m*	**Lab Sample No:**	*S-19-0123*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 5	GS 2	1.8 m	26	39	35		5.1

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Gravelly Silty Sand trace Clay	SP	1.100	0.044	0.003	366.67	0.59

Issued By: [signature] (Senior Project Manager)

Date Issued: *March 15, 2019*