FUNCTIONAL SERVICING & PRELIMINARY STORMWATER MANAGEMENT REPORT

6939 KING STREET

TOWN OF CALEDON
REGION OF PEEL

PREPARED FOR:

SWAMINARAYAN MANDIR VASNA SANSTHA (SMVS)

PREPARED BY:

C.F. CROZIER & ASSOCIATES INC.
2800 HIGH POINT DRIVE, SUITE 100
MILTON, ON L9T 6P4

DECEMBER 2020

CFCA FILE NO. 1990-5787

CROZIER
CONSULTING ENGINEERS

Revision Number	Date	Comments
Rev.0	December 2020	Issued for First Submission

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1.0 Introduction

C.F. Crozier & Associates Inc. (Crozier) was retained by Swaminarayan Mandir Vasna Sanstha (SMVS) c/o Weston Consulting to prepare a Functional Servicing and Preliminary Stormwater Management Report to support the Official Plan Amendment (OPA) and Zoning By-Law Amendment (ZBA) for the proposed place of worship development located at 6939 King Street, in the Town of Caledon (the Site).

This report will demonstrate that the proposed site can be developed in accordance with the Town of Caledon and Peel Region guidelines from a functional servicing and preliminary stormwater management perspective.

The reports and design standards referenced during the preparation of this report include:

- Ontario Building Code (2012)
- The Town of Caledon Development Standards Manual (2019)
- Toronto and Region Conservation Authorities (TRCA) Stormwater Management Criteria Version 1.0 (August 2012)

2.0 Site Description

The subject property covers approximately 6.06 ha and currently comprises of a single-family dwelling located on the north-west corner of the Site with the remaining land comprised of an agricultural field. The Site is located in a rural residential and agricultural neighbourhood and is bounded by King Street to the north, Centreville Creek Road to the east, agricultural lands to the south, and a residential lot to the west.

The south west corner of the Site is located within an area regulated by the Toronto and Region Conservation Area (TRCA).

The elements envisioned for this development include demolishing the existing dwelling in order to construct a place of worship, complete with worship areas, dining hall, gym/activity hall as well as several offices and kitchens. The proposed building will be accompanied with above ground parking lot with a landscaped area in the front. The proposed building will be privately serviced with an on-site sewage system, well and stormwater management feature.

The pertinent background information for the Site have been reviewed, including:

- Site Plan (Battaglia Architect Inc, November 11, 2020)
- Topographic Survey (P & C Surveying Inc, December 6, 2019)
- Geotechnical Investigation (Terraprobe Inc., October 20, 2020)
- Ministry of Environment, Conservation and Parks (MECP) Well Record (December 11, 1985)

3.0 Sanitary Servicing

3.1 Existing Sanitary Servicing

The subject property is in a rural area and does not have municipal sanitary services available. Currently the Town of Caledon does not have plans to provide sanitary servicing in this area.

The existing dwelling on the Site is assumed to be serviced by an on-site sewage system. The house is to be demolished and the existing sewage system is to be decommissioned by a licenced contractor.

3.2 Soil Conditions

Terraprobe Inc. was retained by the Client to complete a geotechnical investigation for the proposed development at 6939 King Street. The report was utilized to establish a percolation rate for this sewage system design. The report includes ten boreholes that were advanced across the property. The borehole relevant to this design (Borehole BH 1) along with a borehole location plan can be found in Appendix A to this report. The soil encountered in the vicinity of the proposed leaching bed (BH 1) comprises of a 1.20 m thick layer of surficial fill overlying an extensive deposit of silt and clay, that extended beyond a termination depth of 8.10 m. Groundwater was observed at an elevation of 1.0 m below grade.

Referring to Appendix A, the percolation time of the predominant native silt and clay deposit encountered by Terraprobe throughout the property and in the vicinity of proposed leaching bed is generally classified as 'ML-CL' under the Unified Soil Classification System (USCS) with a percolation rate of T =20 to 50 min/cm. A percolation rate of T = 50 min/cm is chosen for the purpose of this assessment.

3.3 Design Sewage Flow

The total daily design sanitary sewage flow for the subject property was calculated in accordance with Part 8 of the Ontario Building Code. A review of the received architectural plans indicate that the proposed building will have multiple uses, resulting in different occupancy rates used when determining the peak sewage flow. Table 1 summarizes the expected maximum day sewage flow for the proposed building based on assumed occupancies. Detailed calculations are provided in Appendix A.

Table 1: Peak Sewage Design Flow

Use	**Design Parameter**	**Design Flow per OBC**	**Peak Flow**(L/day)
Place of Worship	970 seats	8 L/day per seat; with kitchen facilities	7,760
Office Area	85 m^2 of floor area	75 L/day per 9.3 m^3 of floor area	683
Cafeteria	168 seats	12 L/day per meal; assume 2 meals per seat	8,064
Activity Hall with Kitchen	300 seats	36 L/day per seat	10,800
Mandir (Activity Hall)	100 seats	8 L/day per seat; no kitchen facilities	800
		Total	**28,107**

The total maximum day sewage flow for the proposed building is estimated at 28,107 L/day. A conservative flow of 30, 000 L/day will be used for design purposes. Note as this flow exceeds 10,000 L/day the property is subject to the Ontario Water Resources Act and will require an Environmental Compliance Approval (ECA) issued by the Ministry of Environment, Conservation and Parks (MECP).

3.4 Proposed Sanitary Servicing

Municipal sanitary sewage services are not available at the subject property. Therefore, the building will be serviced with a privately owned onsite sewage system with subsurface disposal. A Class 4 sewage system consisting of a grease interceptor for kitchen wastewater, a primary settling tank (septic tank), a Level IV treatment unit meeting the CAN/BNQ 3680-600 standard discharging to a Type A Dispersal Bed is proposed. Additional denitrification equipment may also be required subject to MECP review and approval.

Sanitary sewage from the building will flow to the primary settling tanks, where settleable solids are removed and stored. Wastewater from the kitchen(s) will first be directed to an external grease trap in order to remove fats, oil and grease, after which it will flow by gravity to the septic tanks, prior to reaching the advanced treatment unit. The advanced treatment unit will be designed to meet appropriate treatment standards as required by MECP. Minimum effluent objectives as required by 8.6.2.2 of the BOC are expected. Additional treatment for nitrate nitrogen may also be required. The sewage treatment system will be rated to treat a maximum day sewage flow of 30,000 L/day. The treated effluent will be directed to a Type A leaching bed designed in accordance with Section 8.7.7 of the 2012 OBC for final treatment and disposal.

The leaching bed is comprised of a 600 m^2 area of clear stone overlying a 3,750 m^2 area of imported sand. The proposed leaching bed is located on the southeast corner of the property. The detailed onsite sewage system calculations are presented in Appendix B. The Site Grading Plan (DWG C701) and the Site Servicing Plan (DWG C103) illustrate the location of the proposed onsite system to service the development. The internal sanitary plumbing within the building will be designed by the mechanical engineer in accordance with the Ontario Building Code (OBC).

The proposed sewage system described in this report and on the accompanying drawings is a functional level design. As noted above an ECA will be required from the MECP. Therefore, the detailed design of the sewage system will be completed in the future to apply for the ECA.

4.0 Water Servicing

4.1 Existing Water Servicing

There is no watermain infrastructure available to service the Site. An on-site well located in the front yard of the existing dwelling is currently being used to service the Site.

4.2 Design Water Demand

The water demand for the proposed development was calculated, referencing the maximum daily sewage flows as noted above and the appropriate Region of Peel peaking factors. Table 2 summarizes the anticipated water demand and Appendix C contains the detailed water demand calculations.

Table 2: Estimated Design Water Demand

Average Day Demand (L/min)	Maximum Day Demand (L/min)	Peak Hour Demand (L/min)
29.76	41.67	89.29

The design daily sewage flow for the proposed building is 30,000 L/day and using peaking factors of 1.4 and 3.0 for the maximum day and peak hour, respectively were used. The maximum day and peak hour demand of the proposed building is calculated to be 25.0 L/min and 53.57 L/min respectively.

4.3 Fire Flow Demand

The Office of the Fire Marshal Fire Protection Water Supply Guideline for Part 3 in the Ontario Building Code was used to calculate the fire flow. In consultation with the architect, it is understood that the proposed building is classified as a Group A, Division 3 with combustible construction. Referencing the OBC, the spatial coefficients and approximation of the building volumes in cubic meters, it was determined that a storage volume of 314,175 L is required for fire flow services. Table 3 summarizes the estimated fire flow demand and duration to meet fire protection for the proposed building. Appendix B contains the fire flow demand calculations.

Table 3: Estimated Fire Flow

Method	Fire Flow Volume (L)	Fire Flow Demand (L/s)	Duration (hr)
(Part 3 of the OBC)	314,175 L	150	2.00

Please note that the fire flow value is a conservative estimate for comparison purposes only. The architect and mechanical engineer will confirm the fire requirements at the detailed design stage.

4.4 Proposed Water Servicing

It is recommended that the existing well be decommissioned by a licenced well contractor and the Site be serviced by a new drilled well. The well must have a watertight casing to a minimum depth of 6.0 m and located a minimum distance of 15 m to any of the septic system components. The proposed supply well will need to be tested to determine if it can meet the anticipated water demand for the Site. If the proposed well cannot meet the anticipated water demand, then a domestic drinking water cistern will be required to provide sufficient water during peak times. The sizing and design of the water cistern will take place at the detailed design stage. A preliminary location of the proposed well is shown on the Preliminary Servicing Plan.

Fire protection cisterns are proposed to provide the fire protection volume calculated for the property. Three fire protection cisterns will be required to meet the required volume and they will be connected in series. A dry hydrant will be located on the fire route of the building to provide coverage for the proposed building. The fire protection cisterns and the dry hydrant are located in front of the proposed building on the east side. Refer to the Preliminary Site Servicing Plan for details.

5.0 Drainage Conditions

5.1 Existing Drainage

The majority of the Site is comprised of agricultural field with the exception of the northwest quadrant that comprises of a dwelling and driveway that are to be removed. A review of topographic survey indicates that surface runoff on the property drains via sheet flow to the south west corner of the Site and outlets into a tributary of the Humber River.

For the purpose of analyzing the runoff from the Site it is assumed that the Site currently consists of 2 catchments (Catchment C101 and C102). C101 comprises of the north portion of the Site and drains to Catchment C102 which ultimately leads to the tributary at the southwest corner of the property. Please refer to Figure 1 enclosed with this report illustrating the pre-development drainage patterns.

5.2 Proposed Drainage

The proposed development consists of a place of worship accompanied with a parking lot, drive aisles, and landscaped area.

The proposed development will be situated in the north half of the property leaving the southern portion pervious. Stormwater runoff generated from the building and surrounding impervious area is to be collected by a series of catchbasins into the on-site storm sewer network and directed to a SWM Facility at the southwest corner of the property. The SWM facility ultimately outlets to a tributary of the Humber River at the southwest corner of the Site.

The Preliminary Site Servicing and Site Grading Plans illustrate the proposed drainage patterns of the Site, the location and design of the storm sewer, SWM Facility, and all connections.

In the Post Development scenario, catchment C201 will comprise of the proposed development and Catchment 202 will remain as undeveloped, pervious area. Please refer to Figure 2 which illustrates the post-development impervious areas and drainage patterns for the Site. The composite runoff coefficient was calculated by using a runoff coefficient of 0.25 for pervious areas and 0.90 for impervious areas. Table 4 provides a comparison of the pre- and post-development land use areas and composite runoff coefficients.

Table 4: Land Area Comparison

Conditions	Catchment ID (Ha)	Total Area (Ha)	Impervious Area (Ha)	Runoff Coefficient
Pre-Development	C101	3.12	0.00	0.25
	C102	2.97	0.00	0.25
Post-Development	C201	2.83	2.38	0.84
	C202	3.18	0.00	0.25

Under the proposed development plan, the existing major and minor drainage patterns will be generally preserved, and the Site will continue to drain in a general north to south and east to west direction towards the Humber River tributary, as shown on the Site Grading Plan (Drawing C701). Refer to Figures 1 and 2 which highlight the pre- and post-development drainage catchments.

6.0 Stormwater Management

Stormwater management design criteria must comply with the policies and standards of:

- Town of Caledon
- Toronto and Region Conservation Authority (TRCA)
- Ministry of Environment, Conservation and Parks (MECP)

A summary of the stormwater management criteria controls is as follows:

Quantity Control

The Site is located within the watershed for the Humber River and outlets to a tributary to the Humber River. Therefore, the stormwater flow control is dictated by the Humber River Unit Flow Rates as defined within the TRCA Stormwater Management criteria.

Quality Control

Enhanced Level 80% TSS removal.

Water Balance

Retain the first 5 mm of runoff from the Site.

6.1 Stormwater Quantity Control

The Site is located within the Humber River Watershed Sub-basin 36 and is required to control peak flows to the TRCA unit flow rates as summarized in table 5 below.

Table 5: Humber River Watershed Unit Flow for Equation F Sub-Basin 36

Storm	Unit Flow Equation Q = Unit Flow (L/s/ha) A = Area (ha)	Area (ha)	Target Release Rate (L/s)
2-year	Q = 9.506 - 0.719 x ln(A)	2.83	25
5-year	Q = 14.652 - 1.136 x ln(A)		38
10-year	Q = 17.957 - 1.373 x ln(A)		46
25-year	Q = 22.639 - 1.741 x ln(A)		58
50-year	Q = 26.566 - 2.082 x ln(A)		68
100-year	Q = 29.912 - 2.316 x ln(A)		77

Using the Town of Caledon's intensity-duration-frequency (IDF) data, the Rational Method was used to determine the pre-development and post-development uncontrolled peak flow rates for site stormwater runoff. The IDF parameters and associated intensities are included within Appendix B.

Since the post-development uncontrolled peak runoff rates exceed the Humber River Sub-basin 36 target flows, quantity controls are required on site.

The proposed stormwater quantity controls consist of a SWM Facility with a controlled outlet, located at the southwest corner of the Site. Stormwater runoff will enter the SWM Facility via an enhanced grassed swale and flows from the SWM facility will be restricted by two orifice controls in order to meet the target flows. Based on preliminary sizing calculations, two 125 mm in diameter orifice controls are proposed to be installed at two different elevations, in order to provide control for smaller and larger storm events, respectively. Modified Rational Method calculations were prepared and determined the maximum required storage volumes to be 1,966 m^3 during the 100-year storm event. Preliminary grading for the SWM facility provides a total 3,119 m^3 storage volume at a maximum 1.50 m depth. Refer to Appendix C for preliminary stormwater calculations. The Preliminary Servicing and Grading Plan illustrates the location of the SWM Facility and enhanced swale. A summary of site flows and required storage volumes has been provided in Table 6 below.

Table 6: Pre- and Post-Development Flow Rates and Required Storage Volumes

Storm	Target Flow Rate (L/s)	Pre-Development Uncontrolled Flow Rate (L/s)	Post-Development Uncontrolled Flow Rate (L/s)	Post-Development Controlled Flow Rate (L/s)	Storage Volume Required (m^3)	Storage Volume Provided (m^3)
2-year	25	192	558	24.0	630	3,119
5-year	38	246	714	30.4	936	
10-year	46	300	873	34.5	1122	
25-year	58	350	1018	49.5	1336	
50-year	68	395	1146	56.5	1503	
100-year	77	440	1279	62.5	1693	

6.2 Stormwater Quality Control

Stormwater quality control for the Site will be provided by the enhanced grassed swale with an underlying sand filter. The enhanced swale is approximately 125 m long, with a 1.0 m bottom width and will provide the quality treatment prior to entering the SWM Facility. The sand filter details will be determined at detailed design stage.

6.3 Water Balance

Based on the 2.48 ha impervious development area, a 125 m^3 volume is required to meet the water balance criteria (2.48 ha x 0.005 m = 125 m^3). A storage volume of 142 m^3 will be provided below the pond in a 0.15m layer of clear stone which will infiltrate. This storage volume is necessary to comply with the water balance criteria of retaining the first 5 mm of runoff on site.

7.0 Erosion and Sediment Controls During Construction

Erosion and sediment controls will be installed prior to the beginning of any construction activities. They will be maintained until the Site is stabilized or as directed by the site engineer and/or Town of Caledon. The Erosion & Sediment Control locations and details will be provided at detailed design stage. However, some typical erosion and sediment controls could be included during construction on the Site.

Heavy Duty Silt Fencing

Silt fencing will be installed on the perimeter of the Site to intercept sheet flow. Additional silt fence may be added based on field decisions by the site engineer and Owner, prior to, during and following construction.

Rock Mud Mat

A rock mud mat will be installed at the entrance to the construction zone to prevent mud tracking from the Site onto surrounding lands and the perimeter roadway network. All construction traffic will be restricted to this access only.

8.0 Conclusions and Recommendations

Based on the information offered in this report, we offer the following conclusions:

- The Site will be serviced with a proposed septic system including a treatment system and dispersal bed. The septic system design flow is 30,000 L/day and will require an ECA from the MECP.
- The Site will be serviced with a new drilled well to provide the domestic water supply. The new drilled well will need to be tested by a hydrogeologist to confirm the pumping rate. A domestic water supply cistern can be designed in the event that the well cannot provide the anticipated water demands.
- The Site will be serviced with fire water cisterns to provide fire protection per the OBC Part 3.
- Stormwater quantity control objectives will be achieved by a landscaped SWM facility that will outlet to the Humber River tributary located at the southwest corner of the Site.
- Stormwater quality control objectives will be achieved via an enhanced grass swale with an underlying sand filter on the Site.
- Water balance is provided with a layer of clear stone below the pond to provide the 5mm water balance requirement.

Based on the above conclusions, we recommend the approval of the Official Plan Amendment and Zoning By-Law Amendment from the perspective of functional servicing and preliminary stormwater management.

Respectfully submitted,

C.F. CROZIER & ASSOCIATES INC.

Madeline Carter, P.Eng.
Project Engineer

C.F. CROZIER & ASSOCIATES INC.

JLysecki

Jessica Lysecki, P.Eng.
Project Engineer

/stm

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APPENDIX A

Septic System Calculations

CROZIER & ASSOCIATES
Consulting Engineers

ONSITE SEWAGE SYSTEM NON-RESIDENTIAL CALCULATION SHEET

Project Name: 6939 KING STREET
Project Number: 1990-5787
Date: 2020-12-04
Designed By: AS/MC
Checked By: KR

PRELIMINARY FLOW ESTIMATES

Description	Area (ft^2)	Area (m^2)	Unit	Unit Flow	Number of Units	Total Flow (L/day)	References/Notes
Proposed Place of Worship							
Sabha Hall	6160	572	per seat	8	970	7,760	Assumed 970 seats per the architectural plans.
Offices	912	85	per 9.3 m2	75	9	683	Office per floor area, or per employee. Employees unknown.
Cafeteria	3795	353	per meal	12	336	8,064	Assumed 2 meals per day and based on number of seats
Activity Hall with kitchen facility	5940	552	per seat	36	300	10,800	Assumed 300 seats with assembly hall use.
Mandir	559	52	per seat	8	100	800	
					SUBTOTAL AREA	**28,107**	
					Total Maximum Day Sewage Flow:	**28,107**	
					Design Sewage Flow:	30,000	

Pre-Treatment Options

Required septic tank size = 90000 L minimum

Propose Level IV Treatment (Y/N): Y

Native Percolation time, T = 50 min/cm

Imported Percolation time = 10 min/cm

Type A Dispersal Bed

Stone area required = 600 m^2

Sand area required = 3750 m^2 0.375 ha

Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
PROPOSED SMVS TEMPLE
6939 KING STREET
CALEDON, ONTARIO

Prepared for: Swaminarayan Mandir Vasna Sanstha Canada (SMVS)
114 Toryork Drive
Toronto, Ontario
M9L 1X6

Attention: Mr. Rasik Patel

File No. 1-20-0222-01
Issued: October 20, 2020

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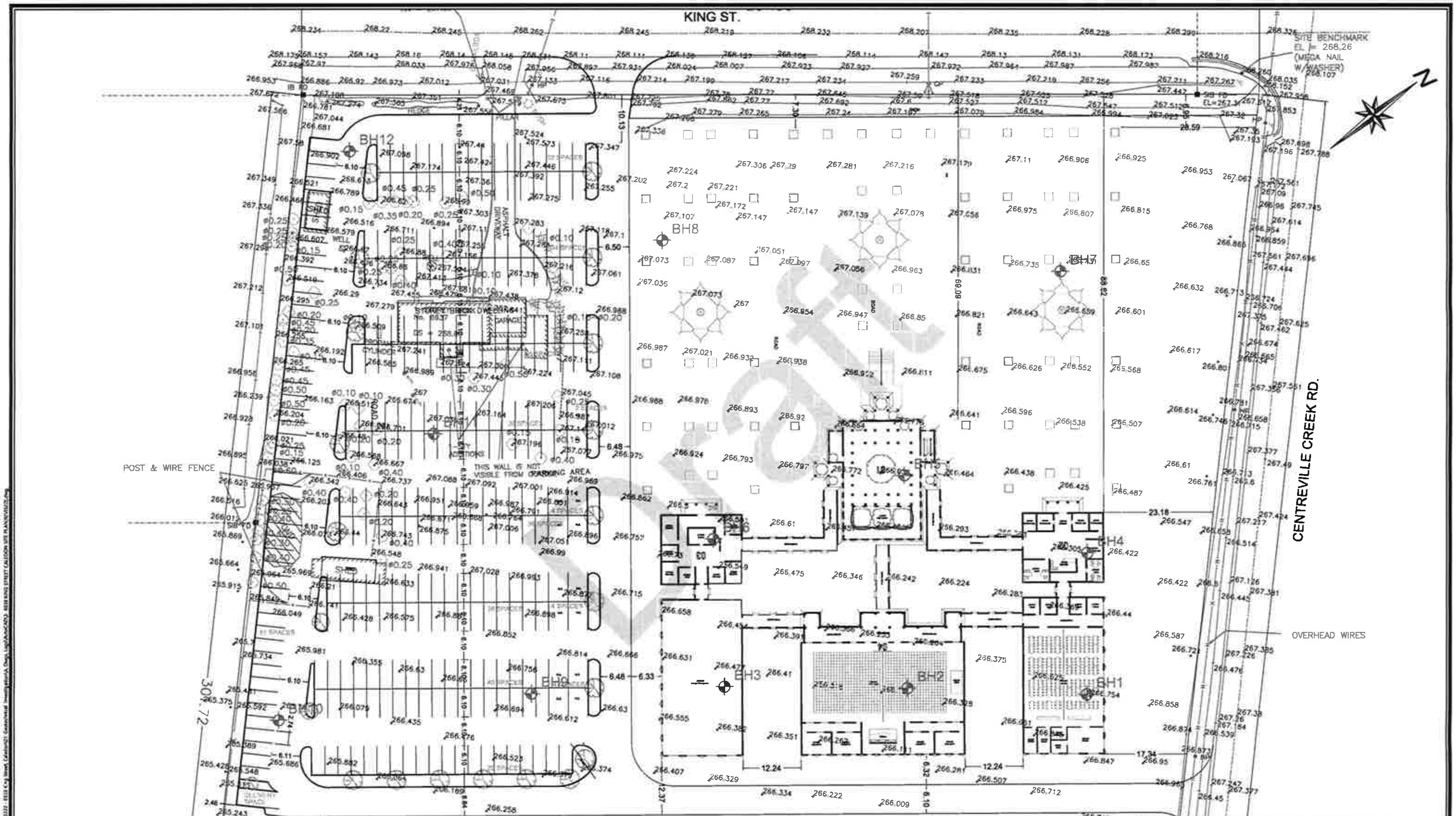

REFERENCE
Project: 6939 King Street, Kleinberg Ontario
Title: Site Plan, Dwg. No.: A1, Date: Aug. 06, 2020
By: Battaglia Architect Inc.

LEGEND
Approximate Borehole Location

0 10 20 30 Metres
SCALE

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

Title: BOREHOLE LOCATION PLAN

File No. 1-20-0222-01

FIGURE: 2

Terraprobe

LOG OF BOREHOLE 1

Project No.	: 1-20-0222-01	Client	: Swaminarayan Mandir Vasna Sansthan Canada	Originated by	: Saif
Date started	: July 14, 2020	Project	: 6939 King Street	Compiled by	: CM
Sheet No.	: 1 of 1	Location	: Caledon, Ontario	Checked by	: SZ

Position : E: 597212, N: 4855726 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Deidric 60, track-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	Elev / Depth (m)	SOIL PROFILE – Description	Sample Number	Type	SPT 'N' Value	Moisture (Natural Water Content)	Lab Data and Comments – Grain Size Distribution (%) (MIT) GR SA SI CL
0	266.8	GROUND SURFACE					
		FILL, clayey silt, trace sand, trace gravel, trace wood chips, very stiff, brown, moist	1	SS	17		
1			2	SS	21		
	265.6 / 1.2	**SILT AND CLAY to CLAYEY SILT**, trace to some sand, trace gravel, stiff to very stiff, brown, moist (GLACIAL TILL)					
			3	SS	17		2 14 48 36
2							
			4	SS	23		
3			5	SS	23		
4							
		...grey below	6	SS	14		
5							
6			7	SS	21		
7		...rock fragments					
8	258.7 / 8.1		8	SS	24		

Elevation Scale (m): 266, 265, 264, 263, 262, 261, 260, 259

Penetration Test Values (Blows / 0.3m): X Dynamic Cone; 10, 20, 30, 40
Undrained Shear Strength (kPa): O Unconfined; + Field Vane; ● Pocket Penetrometer; ■ Lab Vane; 40, 80, 120, 160

Moisture / Plasticity: Plastic Limit (PL), Natural Water Content (MC), Liquid Limit (LL); 10, 20, 30

Headspace Vapour (ppm) | Instrument Details | Unstabilized Water Level

END OF BOREHOLE

Borehole was dry and caved to 5.8 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Oct 5, 2020	1.0	265.9
Oct 13, 2020	5.7	261.1

file: 1-20-0222-01 bh logs.gpj

APPENDIX B

Water Demand Calculations

Domestic Water Demand - Ontario Building Code

			Notes & References
Peak Sewage Flow	30,000	L/day	Ontario Building Code - Table
Avg. Daily Demand =	21429	L/day	Using peaking factor
	29.762	**L/min**	Over a 12 hour period
Peaking Factors			
Max Day =	1.40		Peel Region Public Works Watermain Design Criteria
Peak Hour =	3.00		
Average Day =	29.76	L/min	
Max Day =	**41.67**	L/min	Max Day = (Average Day Demand) * (Max Day Factor)
Peak Hour =	**89.29**	L/min	Peak Hour = (Average Day Demand) * (Peak Hour Factor)

Criteria	Average Daily Water Demand (L/min)	Max Day Demand (L/min)	Peak Hourly Demand (L/min)
OBC Sewage & Peel Region	29.76	41.67	89.29

SMVS - Proposed 1 Storey Place of Worship
Fire Protection Volume Calculation
CFC File: 1990-5787

Fire Protection Water Supply Guideline
Part 3 of the Ontario Building Code (2006)

$$Q = KVS_{TOT}$$

Q = minimum supply of water in litres (L)
K = water supply coefficient
V = total building volume in cubic metres
S_{TOT} = total of spatial coefficient values from property line exposures on all sides

K =	25.0	Group A, Division 3 building with combustible construction conforming to OBC 3.2.2 (Table 1) [confirmed by architect]
V =	12567	Volume per total floor area and average height of 4m
S_{TOT} =	1	S_{TOT} As calculated

Q = 314175 L OR 314.175 m^3

Based on ranges listed in Table 2, the required minimum water supply flow rate is **9000 L/min**

150 L/s

APPENDIX C

Stormwater Management Calculations

CROZIER
CONSULTING ENGINEERS

Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - Input Parameters

Storm Data: Caledon

Time of Concentration: T_c = **10 min** (per city of Town of Caledon standards)

Return Period	A	B	C	I (mm/hr)
2 yr	1070	7.85	0.8759	85.72
5 yr	1593	11.00	0.8789	109.68
10 yr	2221	12.00	0.9080	134.16
25 yr	3158	15.00	0.9335	156.47
50 yr	3886	16.00	0.9495	176.19
100 yr	4688	17.00	0.9624	196.54

Pre - Development Conditions					
Catchment	**Land Use**	**Area** (ha)	**Area** (m^2)	**C**	**Weighted Average C**
101	*Pervious*	3.12	31,224	**0.25**	**0.13**
	Impervious	0.00	0	**0.9**	**0.00**
102	*Pervious*	2.97	29,724	**0.25**	**0.12**
	Impervious	0.00	0	**0.9**	**0.00**
Total Site		**6.09**	**60,948**	**-**	**0.25**

Post - Development Conditions (Controlled)					
Catchment	**Land Use**	**Area** (ha)	**Area** (m^2)	**C**	**Weighted Average C**
Controlled					
201	*Pervious*	0.35	3,500	**0.25**	**0.03**
	Impervious	2.48	24,848	**0.90**	**0.79**
Total Controlled		**2.83**	**28,348**	**-**	**0.82**
Uncontrolled					
202	*Pervious*	3.18	31,800	**0.25**	**0.24**
	Impervious	0.00	-	**0.9**	**0.00**
Uncontrolled to Ditch	*Pervious*	0.08	800	**0.25**	**0.01**
	Impervious	0.00	-	**0.9**	**0.00**
Total Uncontrolled		**3.26**	**32,600**	**-**	**0.25**
Total Site		**6.09**	**60,948**	**-**	**0.51**

Equations:

Peak Flow
$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$

Intensity
$i(T_d) = A / (T + B)^C$

CROZIER
CONSULTING ENGINEERS

Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - Peak Flows Summary

Humber River Unit Flow Rates (TRCA 2012)					
Return Period	Site Area (Ha)	A	B	Unit Flow Rate (L/s/ha)	Target Peak Flow (L/s)
2	2.83	9.506	-0.719	8.670	25
5		14.652	-1.136	13.331	38
10		17.957	-1.373	16.360	46
25		22.639	-1.741	20.614	58
50		26.566	-2.082	24.145	68
100		29.912	-2.316	27.219	77

Pre/Post-Development Uncontrolled Peak Flows (L/s)		
Return Period	Q_{pre}	Q_{post}
2 yr	170	558
5 yr	218	714
10 yr	266	873
25 yr	310	1,018
50 yr	350	1,146
100 yr	390	1,279

*Note Target based on Unit Flow Rates for Humber River Sub-Basin 36, Equation F (TRCA Stromwater Management Criteria August 2012 Version 1.0)

Equations:

Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Quantity Control Release Rates
$$Q_{target} = [A + B \times \ln(Area)](Area)$$

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - Summary

Storm Event (yr)	**Peak Flow Rate**			**Required Storage** (m^3)
	Q_{Target}(L/s)	**Post-Development** (L/s)		
		Uncontrolled	**Controlled**	
2	24.6	558	24.0	630
5	37.8	714	30.4	936
10	46.4	873	34.5	1122
25	58.4	1,018	49.5	1336
50	68.4	1,146	56.5	1503
100	77.2	1,279	62.5	1693

1. Contorlled flows using 75mm diameter orifice

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Dry Pond Outlet Orifice Design - 2 x 125 mm

Depth Increment (m) =	0.05
Inlet Elevation (m) =	263.95

Orifice: $Q=CA(2gH)^{0.5}$	Orifice 1	Orifice 2
Discharge Coef., Cd=	0.80	0.80
Orifice Diameter (mm) =	125	125
Area of Orifice (m^2) =	0.0123	0.0123
Orifice (Side/Bottom) =	Side	Side
Invert (m) =	**263.95**	**264.50**

Storage Rating Curve

	Water Elev. (m)	Depth (m)	Active Length (m)	Active Width (m)	Volume (m3)	Orifice1 Q (Side) L/s	Orifice2 Q (Side) L/s	Total Q
Bottom of Pond	263.95	0.00	28.50	58.50	0	0.00	0.00	0.00
	264.00	0.05	28.80	58.80	84	0.00	0.00	0.00
	264.05	0.10	29.10	59.10	169	8.42	0.00	8.42
	264.10	0.15	29.40	59.40	256	12.86	0.00	12.86
	264.15	0.20	29.70	59.70	344	16.13	0.00	16.13
	264.20	0.25	30.00	60.00	433	18.83	0.00	18.83
	264.25	0.30	30.30	60.30	524	21.19	0.00	21.19
	264.30	0.35	30.60	60.60	616	23.32	0.00	23.32
	264.35	0.40	30.90	60.90	709	25.26	0.00	25.26
	264.40	0.45	31.20	61.20	804	27.07	0.00	27.07
	264.45	0.50	31.50	61.50	900	28.76	0.00	28.76
	264.50	0.55	31.80	61.80	997	30.36	0.00	30.36
	264.55	0.60	32.10	62.10	1096	31.88	0.00	31.88
	264.60	0.65	32.40	62.40	1196	33.33	8.42	41.75
	264.65	0.70	32.70	62.70	1298	34.72	12.86	47.58
	264.70	0.75	33.00	63.00	1401	36.06	16.13	52.18
	264.75	0.80	33.30	63.30	1505	37.34	18.83	56.17
Top of Active Storage (264.84)	**264.80**	**0.85**	**33.60**	**63.60**	**1611**	**38.59**	**21.19**	**59.78**
	264.85	0.90	33.90	63.90	1718	39.80	23.32	63.11
	264.90	0.95	34.20	64.20	1827	40.97	25.26	66.23
	264.95	1.00	34.50	64.50	1937	42.11	27.07	69.17
	265.00	1.05	34.80	64.80	2049	43.21	28.76	71.98
	265.05	1.10	35.10	65.10	2162	44.29	30.36	74.66
	265.10	1.15	35.40	65.40	2276	45.35	31.88	77.23
	265.15	1.20	35.70	65.70	2392	46.38	33.33	79.71
	265.20	1.25	36.00	66.00	2509	47.39	34.72	82.11
	265.25	1.30	36.30	66.30	2628	48.38	36.06	84.43
	265.30	1.35	36.60	66.60	2749	49.34	37.34	86.69
	265.35	1.40	36.90	66.90	2870	50.29	38.59	88.88
	265.40	1.45	37.20	67.20	2994	51.22	39.80	91.02
Top of Pond	265.45	1.50	37.50	67.50	3119	52.14	40.97	93.10

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 100-Year Storm Event

Control Criteria

100 yr: Control Post-Development Peak Flows to Unit Flow Rate

100 yr: Uncontrolled Post-Development Flow:

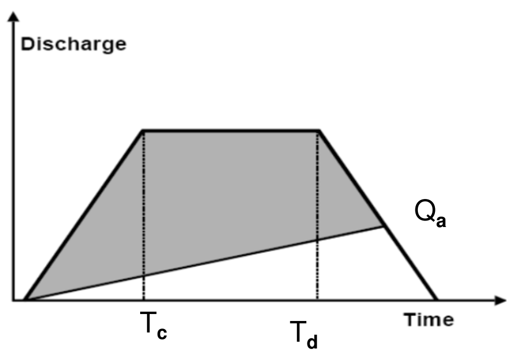
Q_{post} = 1,278.8 L/s

100 yr: Target Flow Rate:

Q_{target} = 77.159 L/s

Q_{actual} = 62.50 L/s

Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	196.54	600	1.279	729.8
20	145.13	1200	0.944	1076.9
30	115.28	1800	0.750	1275.2
40	95.75	2400	0.623	1401.5
50	81.95	3000	0.533	1487.3
75	60.40	4500	0.393	1609.2
100	47.93	6000	0.312	1664.8
125	39.78	7500	0.259	1688.0
150	**34.03**	**9000**	**0.221**	**1692.7**
175	29.75	10500	0.194	1685.9
200	26.45	12000	0.172	1671.3
225	23.81	13500	0.155	1651.1
250	21.66	15000	0.141	1626.8
275	19.87	16500	0.129	1599.4
300	18.36	18000	0.119	1569.6
325	17.07	19500	0.111	1537.8
335	16.60	20100	0.108	1524.6
Required Storage Volume:				**1692.7**



Peak Flow

$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage

$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 50-Year Storm Event

Control Criteria

50 yr: Control Post-Development Peak Flows to Unit Flow Rate

50 yr: Uncontrolled Post-Development Flow:

Q_{post} = 1,146 L/s

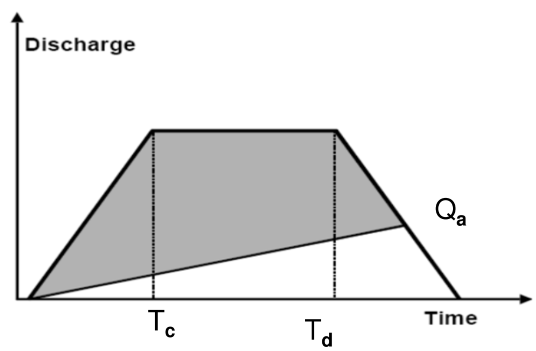
50 yr: Target Flow Rate:

Q_{target} = 68 L/s

Q_{actual} = 57 L/s

Storage Volume Determination

T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	176.19	600	1.146	654.0
20	129.36	1200	0.842	959.2
30	102.50	1800	0.667	1132.7
40	85.04	2400	0.553	1243.2
50	72.75	3000	0.473	1318.4
75	53.63	4500	0.349	1426.2
100	42.59	6000	0.277	1476.2
125	35.39	7500	0.230	1498.0
150	**30.30**	**9000**	**0.197**	**1503.5**
175	26.53	10500	0.173	1498.6
200	23.60	12000	0.154	1486.9
225	21.27	13500	0.138	1470.1
250	19.37	15000	0.126	1449.6
275	17.78	16500	0.116	1426.3
300	16.45	18000	0.107	1400.7
325	15.30	19500	0.100	1373.3
335	14.88	20100	0.097	1361.9
Required Storage Volume:				**1503.5**



Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage
$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 25-Year Storm Event

Control Criteria

25 yr: Control Post-Development Peak Flows to Unit Flow Rate

25 yr: Uncontrolled Post-Development Flow:

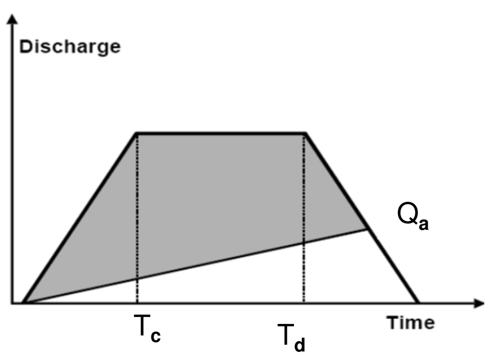
Q_{post}= 1,018 L/s

25 yr: Target Flow Rate:

Q_{target} = 58 L/s

Q_{actual} = 50 L/s

Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	156.47	600	1.018	581.2
20	114.29	1200	0.744	847.9
30	90.39	1800	0.588	999.3
40	74.95	2400	0.488	1096.2
50	64.13	3000	0.417	1162.7
75	47.33	4500	0.308	1259.6
100	37.65	6000	0.245	1306.5
125	31.33	7500	0.204	1328.6
150	**26.88**	**9000**	**0.175**	**1336.4**
175	23.56	10500	0.153	1335.0
200	20.99	12000	0.137	1327.3
225	18.94	13500	0.123	1315.1
250	17.27	15000	0.112	1299.5
275	15.88	16500	0.103	1281.3
300	14.70	18000	0.096	1261.0
325	13.69	19500	0.089	1239.0
Required Storage Volume:				**1336.4**



Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage
$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 10-Year Storm Event

Control Criteria

10 yr: Control Post-Development Peak Flows to Unit Flow Rate

10 yr: Uncontrolled Post-Development Flow:

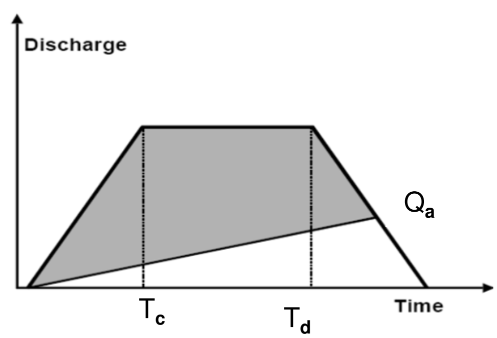
Q_{post} = 873 L/s

10 yr: Target Flow Rate:

Q_{target} = 46 L/s

Q_{actual} = 35 L/s

Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	134.16	600	0.873	503.1
20	95.47	1200	0.621	714.4
30	74.58	1800	0.485	832.1
40	61.44	2400	0.400	907.6
50	52.37	3000	0.341	960.1
75	38.50	4500	0.251	1039.3
100	30.61	6000	0.199	1081.2
125	25.49	7500	0.166	1104.3
150	21.89	9000	0.142	1116.5
175	**19.22**	**10500**	**0.125**	**1121.5**
200	17.15	12000	0.112	1121.6
225	15.50	13500	0.101	1118.1
250	14.15	15000	0.092	1111.9
275	13.03	16500	0.085	1103.4
300	12.07	18000	0.079	1093.3
325	11.26	19500	0.073	1081.7
Required Storage Volume:				**1121.6**



Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage
$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$

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Project: 6939 King Street
Project No.: 1990-5787
Created By: JA
Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 5-Year Storm Event

Control Criteria

5 yr: Control Post-Development Peak Flows to Unit Flow Rate

5 yr: Uncontrolled Post-Development Flow:

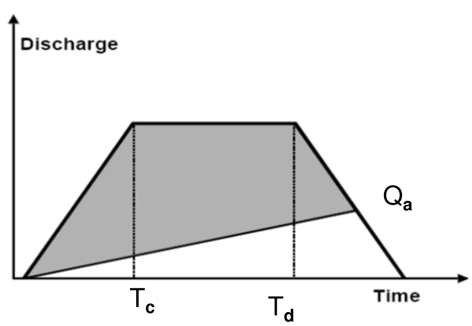
Q_{post} = 714 L/s

5 yr: Target Flow Rate:

Q_{target} = 38 L/s

Q_{actual} = 30 L/s

Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	109.68	600	0.714	409.9
20	77.89	1200	0.507	580.8
30	60.92	1800	0.396	677.0
40	50.28	2400	0.327	739.6
50	42.96	3000	0.280	783.9
75	31.77	4500	0.207	852.6
100	25.39	6000	0.165	890.7
125	21.23	7500	0.138	913.1
150	18.31	9000	0.119	926.2
175	16.13	10500	0.105	933.0
200	**14.43**	**12000**	**0.094**	**935.5**
225	13.08	13500	0.085	934.8
250	11.97	15000	0.078	931.5
275	11.05	16500	0.072	926.3
300	10.26	18000	0.067	919.4
325	9.59	19500	0.062	911.3
Required Storage Volume:				**935.5**



Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage
$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$

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Project No.: 1990-5787
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Checked By: JL
Date: 2020-11-17
Updated: 2020-12-22

Modified Rational Calculations - 2-Year Storm Event

Control Criteria

2 yr: Control Post-Development Peak Flows to Unit Flow Rate

2 yr: Uncontrolled Post-Development Flow:

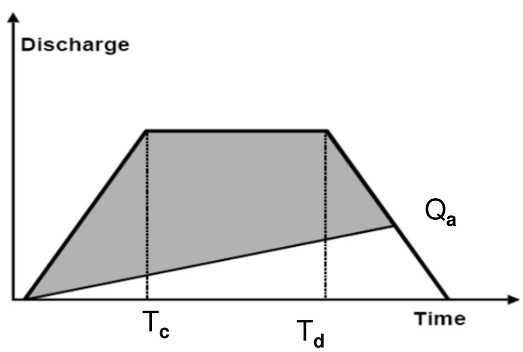
Q_{post}= 558 L/s

2 yr: Target Flow Rate:

Q_{target} = 25 L/s

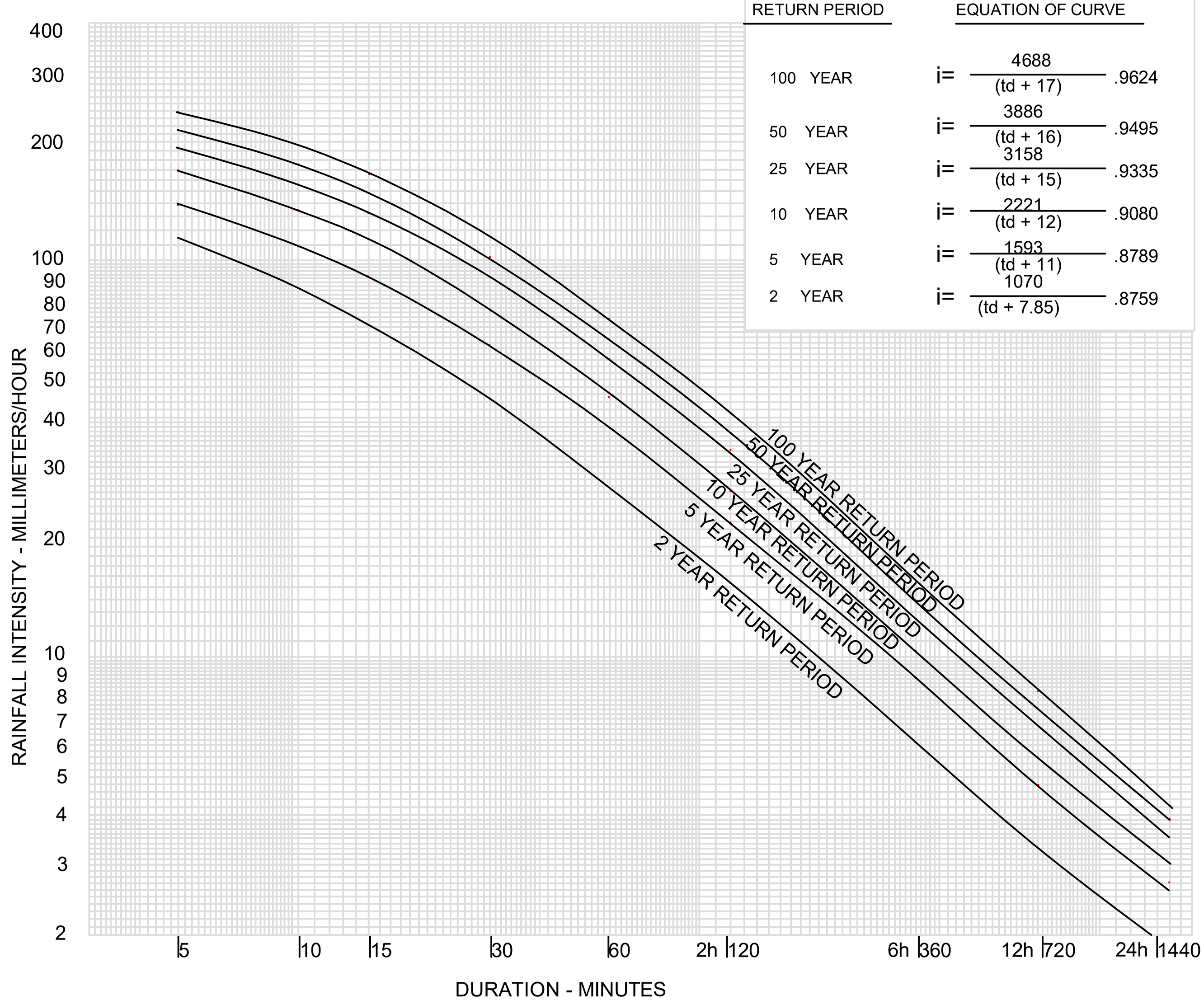
Q_{actual} = 24 L/s

Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	85.72	600	0.558	320.2
20	58.06	1200	0.378	431.7
30	44.38	1800	0.289	490.9
40	36.14	2400	0.235	528.3
50	30.60	3000	0.199	554.2
75	22.34	4500	0.145	593.0
100	17.74	6000	0.115	613.2
125	14.78	7500	0.096	623.8
150	**12.70**	**9000**	**0.083**	**628.8**
175	11.17	10500	0.073	629.9
200	9.98	12000	0.065	628.3
225	9.04	13500	0.059	624.7
250	8.27	15000	0.054	619.5
275	7.62	16500	0.050	613.1
300	7.08	18000	0.046	605.7
325	6.61	19500	0.043	597.4
Required Storage Volume:				**629.9**



Peak Flow
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i(T_d) \cdot A$$

Storage
$$S_d = Q_{post} \cdot T_d - Q_{target} (T_d + T_c) / 2$$



RETURN PERIOD	EQUATION OF CURVE
100 YEAR	$i = \frac{4688}{(td + 17)^{.9624}}$
50 YEAR	$i = \frac{3886}{(td + 16)^{.9495}}$
25 YEAR	$i = \frac{3158}{(td + 15)^{.9335}}$
10 YEAR	$i = \frac{2221}{(td + 12)^{.9080}}$
5 YEAR	$i = \frac{1593}{(td + 11)^{.8789}}$
2 YEAR	$i = \frac{1070}{(td + 7.85)^{.8759}}$

INLET TIMES

SUBURBAN RESIDENTIAL	
(ROOF DRAINS UNCONNECTED)	15 min
(ROOF DRAINS CONNECTED)	10 min
SUBURBAN, COMMERCIAL, INDUSTRIAL MULTIPLE FAMILY	10 min
DOWNTOWN COMMERCIAL, HIGH DENSITY APARTMENTS, EXPRESSWAYS	5 min

RUNOFF COEFFICIENT

COMMERCIAL	
- DOWNTOWN & SUBURBAN SHOPPING	0.90
INDUSTRIAL	
- DOWNTOWN	0.90
- SUBURBAN INDUSTRIAL PARKS	0.75
RESIDENTIAL	
- APARTMENTS	0.75
- ROW DWELLINGS	0.70
- DUPLEX DWELLINGS	0.70
- SEMIDETACHED - DOWNTOWN	0.60
- SINGLE FAMILY - DOWNTOWN	0.60
- SEMIDETACHED - SUBURBAN	0.50
- SINGLE FAMILY - SUBURBAN	0.40
SCHOOLS, CHURCHES, HOSPITALS	0.75
PARKS, CEMETERIES, RAIL YARDS	
(OVER 4 Ha)	0.20
(UNDER 4 Ha)	0.25
PARKING LOTS	
ASPHALT & GRAVEL	0.90

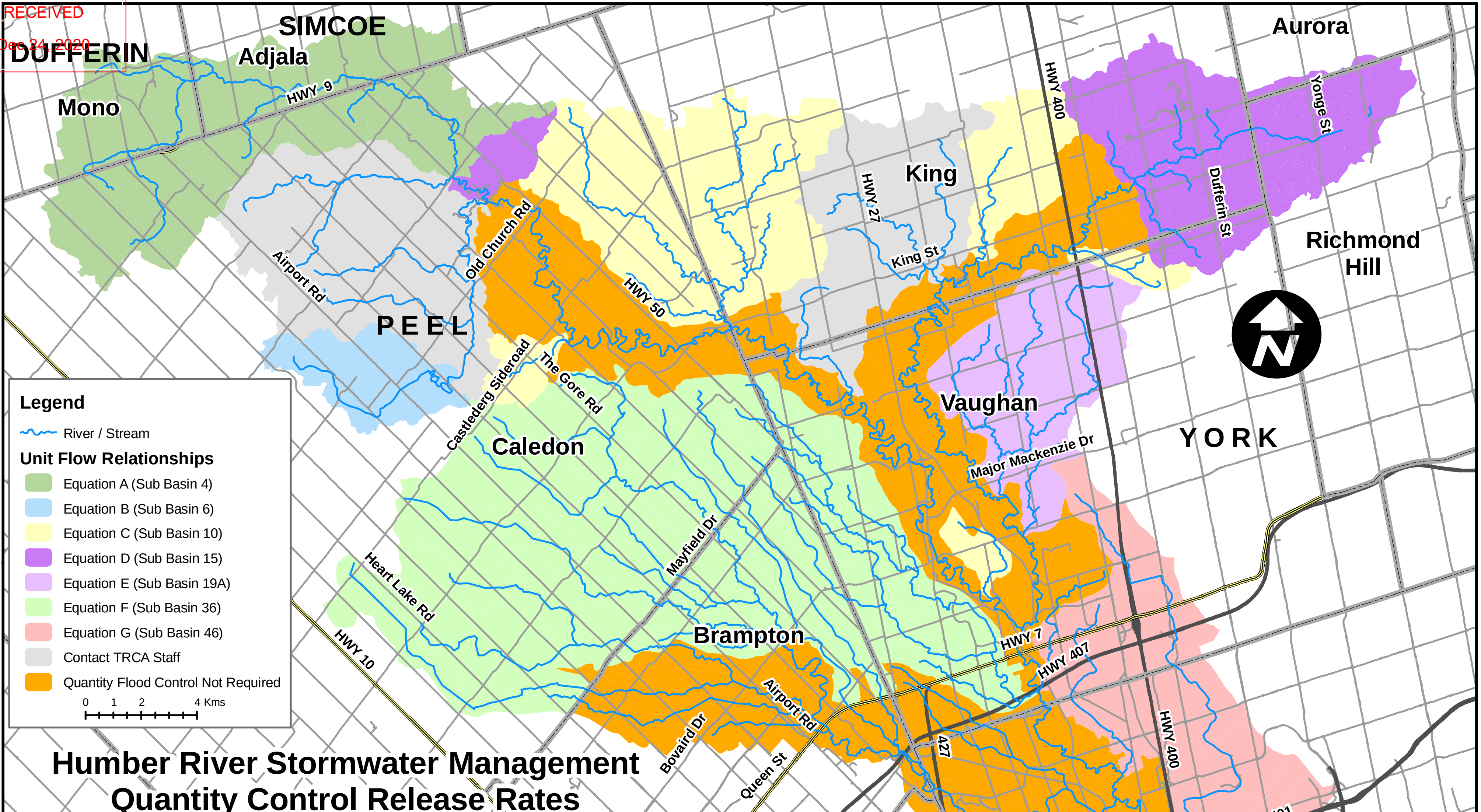
TOWN OF CALEDON

RAINFALL INTENSITY CURVES

NO.	REVISION	APR'D	DATE
3	ADDITION OF TEXT		APR 19
2	STANDARD 104 NOW 103		JAN 08
1	STANDARD 112.01 NOW 104		JUNE 08

APR'D: C.C.
DATE: FEB 2000
DRAWN: BJM
SCALE: N.T.S.

STANDARD No. 103



Humber River Stormwater Management Quantity Control Release Rates

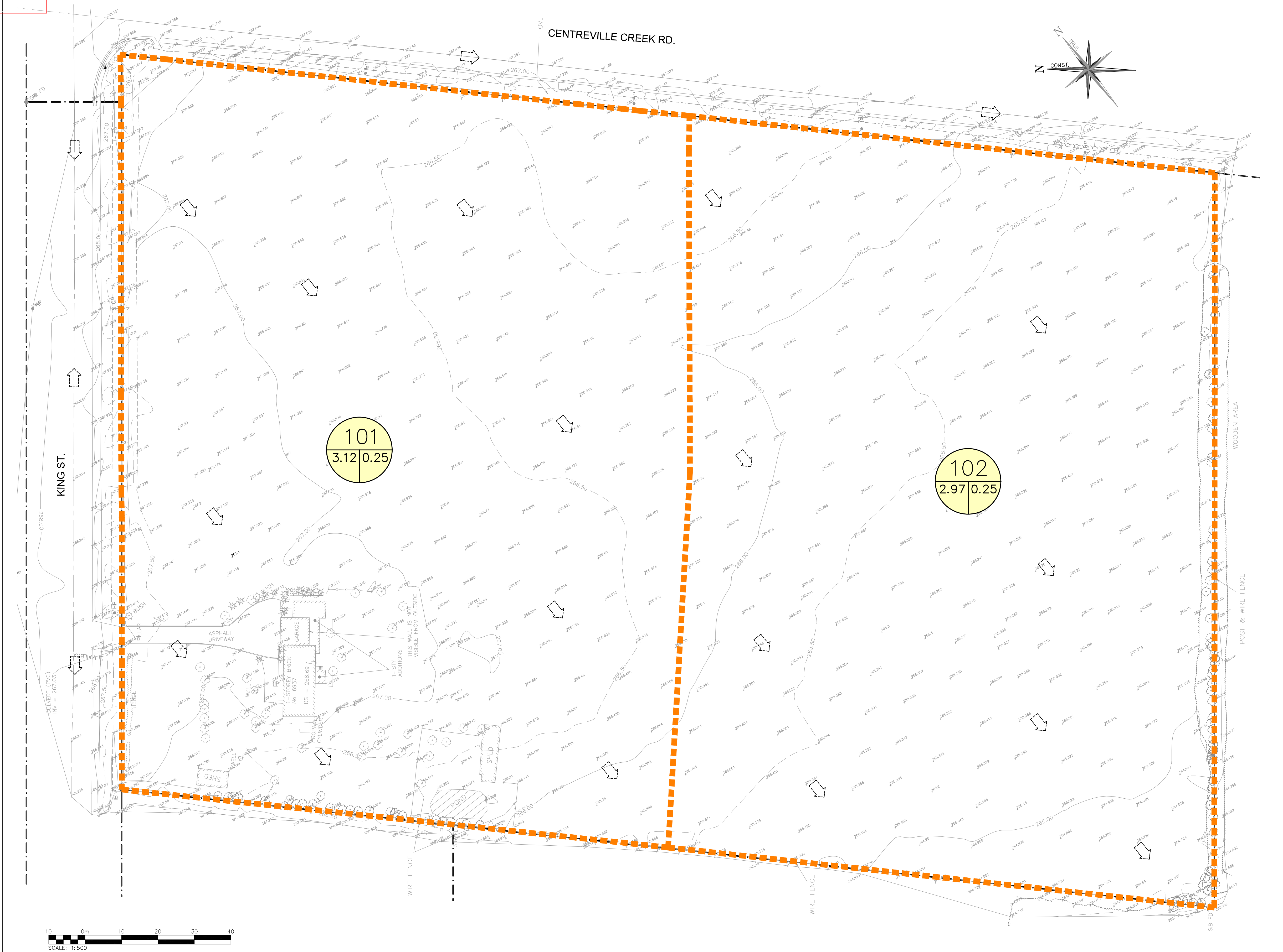
TABLE E.1: SUMMARY OF UNIT FLOW RELATIONSHIPS
HUMBER RIVER WATERSHED

TORONTO AND REGION Conservation for The Living City

Return Period	Equation A Sub-Basin 4	Equation B Sub-Basin 6	Equation C Sub-Basin 10	Equation D Sub-Basin 15	Equation E Sub-Basin 19A	Equation F Sub-Basin 36	Equation G Sub-Basin 46
100-Year	Q=6.086-0.445*In(A)	Q=15.199-0.751* In(A)	Q=14.140-1.096* In(A)	Q=20.388-1.899 *In(A)	Q=15.911-1.785 *In(A))	Q=29.912-2.316 *In(A)	Q=22.973-2.256 *In(A)
50-Year	Q=5.147-0.376 *In(A)	Q=12.692-0.623 *In(A)	Q=11.920-0.921*In(A)	Q= 17.051-1.577 *In(A)	Q=13.700-1.531 *In(A)	Q=26.566-2.082 *In(A)	Q=20.164-1.973 *In(A)
25-Year	Q=4.272-0.312 *In(A)	Q= I0.488-0.522 *In(A)	Q=9.838-0.757 *In(A)	Q=14.037-1.293 *In(A)	Q=11.653-1.297 *In(A)	Q=22.639-1.741 *In(A)	Q=17.381-1.690 *In(A)
10-Year	Q=3.192-0.233 *In(A)	Q=7.707-0.382 *In(A)	Q=7.443-0.578 *In(A)	Q=10.400-0.953 *In(A)	Q=9.213-1.031 *In(A)	Q=17.957-1.373 *In(A)	Q=13.877-1.342 *In(A)
5-Year	Q=2.410-0.175 *In(A)	Q=5.755-0.283 *In(A)	Q=5.557-0.427 *In(A)	Q=7.930-0.73I *In(A)	Q=7.358-0.829 *In(A)	Q=14.652-1.136 *In(A)	Q=II.468-1.123 *In(A)
2-Year	Q=1.439-0.106 *In(A)	Q=3.288-0.159 *In(A)	Q=3.142-0.233 *In(A)	Q=4.722-0.454 *In(A)	Q=4.567-0.503 *In(A)	Q=9.506-0.719 *In(A)	Q=7.745-0.762 *In(A)

Note:
1. Q - Unit flow (Lls/ha - litres per second per hectare).
2. A - Area in hectares (ha).

FIGURES



LEGEND

PROPERTY LINE	
EXISTING CONTOUR (0.5m)	
EXISTING CONTOUR (1.0m)	
x 215.00	EXISTING GRADE
EXISTING FENCE	
EXISTING TREE	
EXISTING OVERLAND FLOW DIRECTION	
STORM DRAINAGE CATCHMENT	
ID / A / RC	CATCHMENT I.D. / AREA (ha) | RUNOFF COEFFICIENT

Stamp | Stamp

PRELIMINARY

NOT FOR CONSTRUCTION

No.	ISSUE / REVISION	YYYY/MMM/DD
0	ISSUED FOR FIRST SUBMISSION	2020/DEC/23

ELEVATION NOTE:

ELEVATIONS SHOWN ON THIS PLAN ARE DERIVED FROM A COSINE BENCHMARK No. 00819758057

ELEVATION = 251.929m

SURVEY NOTES:

SURVEY COMPLETED BY P&C SURVEYING INC. (2019/DEC/06)
REFERENCE No.: 2019-1206

BEARINGS ARE UTM GRID, DERIVED FROM RTN OBSERVATIONS
UTM ZONE 17, NAD83 (GSRS) (2010.0)

DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.9996781

SITE PLAN NOTES:

DESIGN ELEMENTS ARE BASED ON SITE PLAN BY BATTAGLIA ARCHITECT INC.
DRAWING No.: A1, (2020/NOV/11)

DRAWING NOTES:

THIS DRAWING IS THE EXCLUSIVE PROPERTY OF C.F. CROZIER & ASSOCIATES INC. AND THE REPRODUCTION OF ANY PART OF IT WITHOUT PRIOR WRITTEN CONSENT OF THIS OFFICE IS STRICTLY PROHIBITED.

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, LEVELS, AND DATUMS ON SITE AND REPORT ANY DISCREPANCIES OR OMISSIONS TO THIS OFFICE PRIOR TO CONSTRUCTION.

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ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

Project

6939 KING STREET
TOWN OF CALEDON
REGION OF PEEL

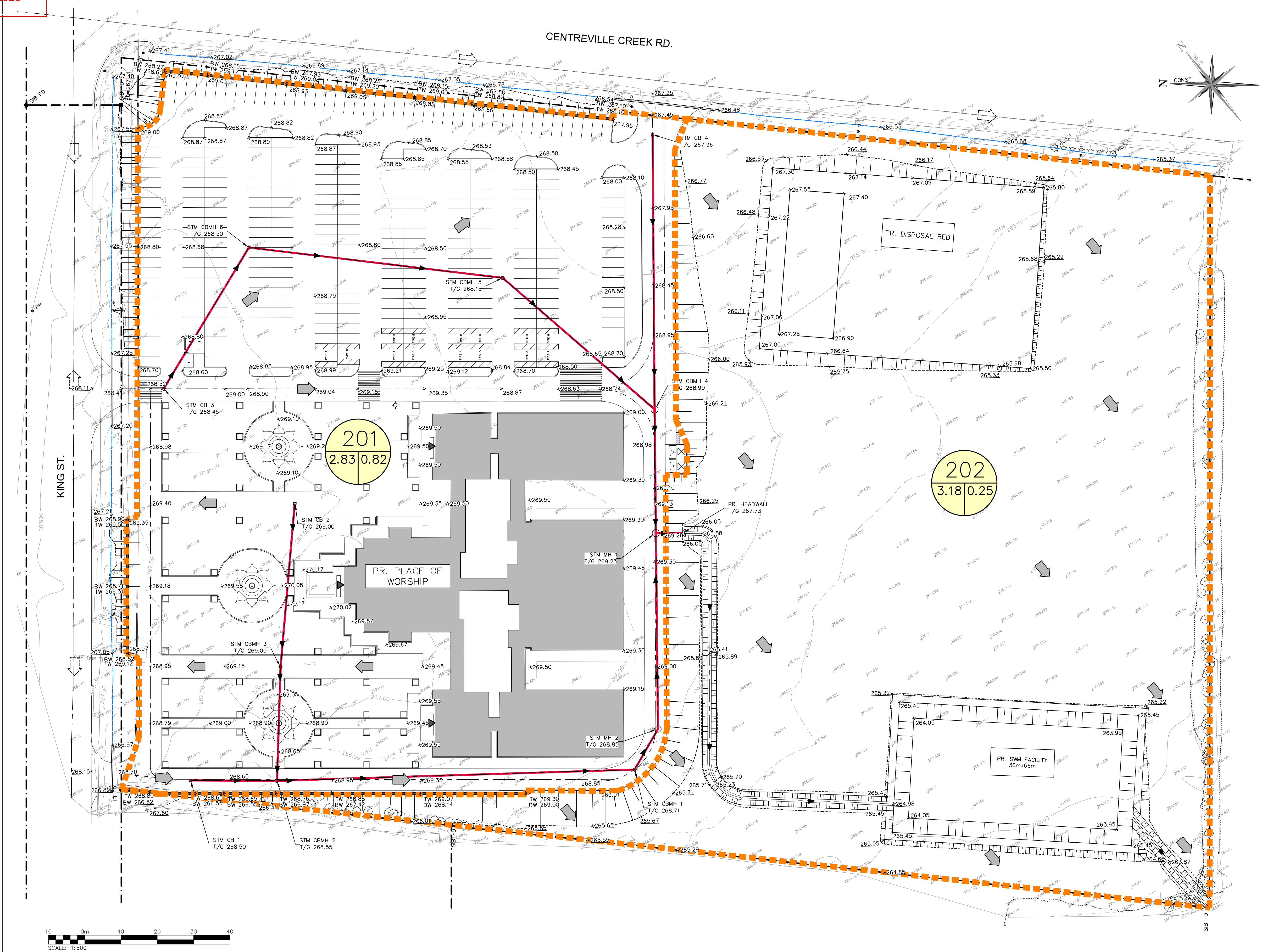
Drawing

PRE-DEVELOPMENT DRAINAGE PLAN

CROZIER
CONSULTING ENGINEERS

2800 HIGH POINT DRIVE
SUITE 100
MILTON, ON L9T 6P4
905-875-0026 T
905-875-4915 F
WWW.CFCROZIER.CA

Drawn	Design	Project No.	
N.C.	M.C./J.A.	1990-5787	
Check	**Check**	**Scale**	**Dwg.**
M.C./J.A.	J.L.	1:500	FIG. 1



LEGEND

- PROPERTY LINE
- EXISTING CONTOUR (0.5m)
- EXISTING CONTOUR (1.0m)
- EXISTING DITCH
- EXISTING GRADE (215.00)
- PROPOSED GRADE (×215.00)
- PROPOSED GRADE (TO MATCH EXISTING) (×215.00)
- EXISTING FENCE
- EXISTING TREE
- EXISTING OVERLAND FLOW DIRECTION
- PROPOSED OVERLAND FLOW DIRECTION
- STORM DRAINAGE CATCHMENT
- CATCHMENT I.D. (ID)
- AREA (ha) | RUNOFF COEFFICIENT (A | RC)
- PROPOSED RETAINING WALL
- PROPOSED SLOPE
- BUILDING ENTRANCE (PERSONNEL DOOR)
- PROPOSED STORM SEWER & MANHOLE
- PROPOSED CATCHBASIN MANHOLE
- PROPOSED SINGLE / DOUBLE CATCHBASIN

Stamp | Stamp

PRELIMINARY
NOT FOR CONSTRUCTION

No.	ISSUE / REVISION	YYYY/MMM/DD
0	ISSUED FOR FIRST SUBMISSION	2020/DEC/23

ELEVATION NOTE:

ELEVATIONS SHOWN ON THIS PLAN ARE DERIVED FROM A COSINE BENCHMARK No. 00819758057

ELEVATION = 251.929m

SURVEY NOTES:

SURVEY COMPLETED BY P&C SURVEYING INC. (2019/DEC/06)
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Drawn N.C.	Design M.C./J.A.	Project No.	1990-5787
Check M.C./J.A.	Check J.L.	Scale 1:500	Dwg. FIG. 2