FUNCTIONAL SERVICING & PRELIMINARY STORMWATER MANAGEMENT REPORT

13247 & 13233 NUNNVILLE ROAD

TOWN OF CALEDON

PREPARED FOR:

BOLTON MIDTOWN DEVELOPMENTS INC.

PREPARED BY:

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

JANUARY 2020

CFCA FILE NO. 649-5291

CROZIER
CONSULTING ENGINEERS

Revision Number	Date	Comments
Rev.0	August 2, 2019	Issued for 1st Submission
Rev.1	January 24, 2020	Issued for 2nd Submission: Revised Section 3.2, 6.1, and 6.2

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1.0 Introduction

C.F. Crozier & Associates Inc. (Crozier) was retained by Bolton Midtown Developments Inc. (BMDI) to prepare a Functional Servicing and Preliminary Stormwater Management Report in support of concurrent Official Plan Amendment (OPA) and Zoning By-Law Amendment (ZBA) applications for the properties located at 13247 & 13233 Nunnville Road in the Town of Caledon.

This report was previously submitted in August 2019 and has been revised to address agency comments. The major updates to the report include revising the watermain size (section 3.2), including a storm sewer design sheet (Section 6.1), and revising the stormwater quality control strategy (section 6.3).

This report demonstrates how the proposed development's functional servicing and stormwater management will integrate with the area's existing water, sanitary and stormwater infrastructure.

2.0 Site Description

The subject property is part of an established residential area in Bolton. The property combines 13247 & 13233 Nunnville Road lots. Each lot has an existing detached residential building. The proposed development covers an area of approximately 3.3 ha with a developable area of approximately 2.0 ha. The developable area is proposed to include 29 single detached dwellings, a municipal road, and associated landscaped areas. The remainder of the site is environmentally sensitive and is regulated by the Toronto and Region Conservation Authority (TRCA). The limit of development was confirmed and staked in the field on June 18, 2019 with TRCA and Town representation present.

The subject property is bounded by:

- A TRCA Regulated Environmental Policy Area (EPA) to the north
- Albion-Vaughan Road to the east
- Existing residential properties to the south and west

A review of the Bolton Urban Community Water and Wastewater Analysis (Aecom, March 2010) indicates that the subject property is within Intensification Area No.4 which considers intensification and subsequent future impacts on the water and wastewater systems.

3.0 Water Servicing

The Region of Peel is responsible for the operation and maintenance of the public water supply and treatment system in the Town of Caledon. Any local water supply system will connect to the Region's municipal water network.

3.1 Existing Water Servicing

According to Allto Construction as-constructed drawing 36211-D dated October 2007, there is an existing 150 mm diameter PVC watermain along Nunnville Road and an existing fire hydrant located north of the proposed development along the west boulevard of Nunnville Road.

3.2 Design Water Demand

The Region of Peel Sanitary Design Criteria (March 2017) was used to determine an equivalent population estimate for the proposed residential development. The results are provided in Table 1 and detailed calculations are provided in Appendix A.

Table 1: Equivalent Population Estimate

Standard	Site Area	Persons/ha	Total Persons
Region of Peel Sanitary Design Criteria (March 2017)	2.0	70	140

The Region of Peel Watermain Design Criteria (June 2010) was used to determine the maximum domestic water demand generated by the proposed development based on the equivalent population estimate for the site. Table 2 summarizes the estimated design water demand. Appendix A contains detailed calculations.

Table 2: Estimated Design Water Demand

Standard	Average Daily Demand (L/s)	Maximum Daily Demand (L/s)	Peak Hourly Demand (L/s)
Region of Peel Watermain Design Criteria (June 2010)	0.45	0.91	1.36

For this application, the domestic water service will be designed to convey a peak domestic design water demand of 1.36 L/s.

3.3 Fire Flow Demand

The Ontario Fire Marshall Fire Protection Water Supply Guideline for Part 3 in the Ontario Building Code (October 1999) was used to estimate the fire flow demand for the proposed development.

Based on dwelling volume, and exposure distance, the preliminary fire demand using this method is 45 L/s. Detailed calculations are provided in Appendix A.

Hydrant flow testing was carried out by Aquazition on the existing 150 mm diameter municipal watermain on Nunnville Road. Based on the hydrant test results, at 276 kPa (40 psi) residual pressure, a minimum of 171 L/s (2,708 US GPM) projected flow is available within the 150 mm diameter municipal watermain on Nunnville Road. Detailed results of the hydrant flow test and projected fire flows are provided in Appendix A.

3.4 Proposed Water Servicing

In conformance with the Bolton Urban Community Water and Wastewater Analysis (Aecom, March 2010) for Intensification Area No. 4, water servicing for the subdivision will be provided through the existing 150mm diameter watermain in Nunnville Road. A proposed 150mm diameter watermain will extend from the existing watermain, up the proposed road and loop around the cul-de-sac.

Each lot will be serviced with individual 25 mm diameter water services, connecting to the proposed private 150 mm diameter watermain. All proposed water service connections will be equipped with a valve and box near the property line. A water meter will be installed within the building envelope per the mechanical design and specifications. The proposed water servicing plan is shown on Drawing C102.

4.0 Sanitary Servicing

The Region of Peel is responsible for the operation and maintenance of the public sewage collection and treatment system in the Town of Caledon. Any local sewage system will connect to the Region's municipal sanitary sewage network.

4.1 Existing Sanitary Servicing

According to Southridge Estates Phase 7 drawing 25231-D (Falby Burnside & Associates, May 1997) there is an existing 250mm diameter sanitary sewer flowing south on Nunnville Road. Additionally, according to Trunk Sanitary Sewers (Region of Peel, July 2018) there is a proposed 900mm diameter trunk sanitary sewer proposed on Nunnville Road to connect to the existing trunk sanitary sewer on Albion-Vaughan Road.

4.2 Design Sanitary Flow

The Region of Peel Sanitary Design Criteria (March 2017) and the equivalent population estimate from Section 3.2, was used to determine the estimated design sanitary flow for the proposed development. Estimated design sanitary calculations are provided in Table 3, and detailed calculations are provided in Appendix B.

Table 3: Estimated Design Sanitary Flow

Standard	Average Day (L/s)	Peaking Factor	Infiltration Flow (L/s)	Total Flow (L/s)
Region of Peel Sanitary Design Criteria (March 2017)	0.49	4.20	0.40	2.46

The proposed sanitary service must convey a total design sanitary demand of 2.46 L/s determined according to the Region of Peel Sanitary Design Criteria (March 2017).

4.3 Proposed Sanitary Servicing

In conformance with the Bolton Urban Community Water and Wastewater Analysis (Aecom, March 2010) for Intensification Area No. 4, sanitary servicing will be provided through a connection to the proposed 900mm diameter sanitary trunk sewer on Nunnville Road. A 250mm diameter sub-trunk sewer will extend from the proposed trunk sewer to service the subdivision. Further coordination with the Region is required for the detailed design of the connection, but preliminary conversations with the Region's project manager, Joanna Pietkiewicz, indicated that a sub-trunk sewer connection is permitted. Refer to the attached email correspondence in Appendix B for details.

Each lot will be serviced with individual 125 mm diameter sanitary services extending from the proposed 250mm diameter sanitary sub-trunk sewer. The proposed 250 mm diameter sanitary sub-trunk sewer, installed at 2.0%, has a capacity of approximately 82.0 L/s (assuming 80% full). 82.0 L/s is greater than the design sanitary flow of 2.46 L/s and therefore the sub-trunk sewer has enough capacity to service the proposed development. Drawing C102 shows the proposed sanitary sub-trunk sewer.

5.0 Drainage Conditions

5.1 Existing Drainage Conditions

According to the topographic survey (R-PE Surveying Ltd., May 2019) the site slopes west to east and existing stormwater flows are conveyed overland to three outlets.

- Catchment 101 (0.22 ha) demonstrates drainage from the north of the site discharging through the adjacent property to the TRCA regulated Environmental Policy Area (EPA).
- Catchment 102 (1.58 ha) demonstrates drainage discharging to the existing east ditch parallel to Albion-Vaughan Road.
- Catchment 103 (0.20 ha) demonstrates drainage from the north of the site discharging to the adjacent residential property

Figure 1 illustrates the delineation of the drainage areas and existing drainage conditions. The existing residential dwellings are not connected to the municipal storm infrastructure.

5.2 Proposed Drainage Conditions

The developable area is proposed to include 29 single detached dwellings, a municipal road, and associated landscaped areas. Only stormwater flows within the development limit will be considered for this analysis. Lands outside the development limit will maintain their existing drainage patterns.

The subject property has been divided into three post-development stormwater catchment areas as shown on Figure 2**.** The grading of the site results in the following post-development drainage catchments:

- Catchment 201 (0.53 ha) discharging uncontrolled to the existing east ditch.
- Catchment 202 (1.21 ha) discharging controlled to the existing ditch through the internal storm system.
- Catchment 203 (0.26 ha) discharging controlled to the existing ditch through the internal storm system.

In accordance with Town of Caledon standards, stormwater flows will be attenuated so the post-development peak flows for all storm events match or are less than the pre-development peak flows for all storm events. The controlled catchments 202 and 203 are independently controlled by orifice tubes. The pre-to-post control will be achieved using orifice tubes downstream of the oversized storm sewers.

Emergency flows will be directed south for both catchments, eventually discharging to the existing ditch. The overland flow route for an emergency flow scenario is outlined in Figure 2.

6.0 Stormwater Management

The proposed stormwater management design must comply with the Town of Caledon Development Standards Manual (V5, 2019). Table 4 provides a summary of the stormwater management criteria based on the stormwater management design guidelines.

Table 4: Summary of Stormwater Management Controls

Control Parameter	Catchment 202
Quantity Control	Post-development peak stormwater flows must be equal or less than pre-development peak stormwater flows.
Quality Control	Achieve Ontario Ministry of the Environment, Conservation and Parks (MECP) Enhanced Level of protection (80% total suspended solids (TSS) removal)
Water Balance	Retain 5 mm rainfall event on-site
Erosion and Sediment Controls	Provided during construction and until the site is stabilized.

6.1 Stormwater Quantity Control

The Modified Rational Method was used to determine the pre-development and post-development flow rates for the site using the Town's IDF rainfall data according to Town Standard Drawing No. 103. The peak flow rates were then used to determine if any stormwater quantity control was required for the proposed development.

As described in Section 5.2, Catchment 201 will be an uncontrolled area discharging to the existing ditch. No quantity controls will be provided for this catchment, and instead the quantity requirements will be compensated for by over-controlling Catchment 202 and 203.

Controlled stormwater from Catchment 202 and 203 will outlet to the existing east ditch. Orifice pipes to restrict flow, and oversized storm sewers will be used to control the post-development peak stormwater flows to the pre-development peak stormwater flows. A summary of the peak flow rates, high water levels, and storage volumes for Catchment 202 and 203 is presented in Table 5.

Table 5: Summary of Peak Flow Rates and Storage Volumes

Storm Event (yr)	**Catchment 202**			
	Flowrate (250mm Orifice Tube) (L/s)	**High Water Level** (m)	**Required Storage** (m^3)	**Provided Storage** (m^3)
2	19	243.12	216	668
5	22	243.45	330	
10	23	243.68	407	
25	25	244.00	514	
50	27	244.27	587	
100	30	244.76	663	
Storm Event (yr)	**Catchment 203**			
	Flowrate (75mm Orifice Tube) L/s	**High Water Level** (m)	**Required Storage** (m^3)	**Provided Storage** (m^3)
2	14	242.72	29	104
5	16	242.80	45	
10	17	242.97	58	
25	18	243.19	75	
50	20	243.34	88	
100	21	243.52	103	

As shown in Table 5, a 250 mm diameter orifice tube and a 75mm diameter orifice tube are required for Catchments 202 and 203 respectively to control post-development peak flows to pre-development peak flows and meet the quantity control criteria. The orifice pipes will be located upstream of the Jellyfish Filter to attenuate peak flows. Detailed orifice sizing will be provided at detailed design.

Based on the orifice control, the high-water level in catchment 202 is 244.76 which is below the lowest catchbasin elevation in Catchment 202 of 245.38. The high-water level in catchment 203 is 243.52 which is below the lowest catchbasin elevation in Catchment 203 of 244.50. Therefore, the stormwater storage is fully contained within the oversized storm sewers.

2400 mm diameter storm sewers at 0.5% provide storage capacity for attenuated peak flows. The volume of storage capacity of the storm sewers for each catchment exceeds the required storage volume therefore the oversized storm sewers have capacity to provide storage volume up to and including the 100-year storm event. A storm sewer design sheet prepared for the site demonstrates that all mainline storm sewers have velocities between 1.12 m/s – 3.87 m/s which is within the Town's acceptable range of 0.75 m/s – 4.0 m/s (Town of Caledon Development Standards Manual V5, 2019, Section 1.4.2.2.2).

A summary of the total peak flows for the site is illustrated in Table 6.

Table 6: Summary of Peak Flow Rates

Storm Event (yr)	**Flow Rates** (L/s)				
	Pre-Development	**Post-Development**			
		Uncontrolled Flow	**Controlled Flow**		**Total**
	Q_{101}	Q_{201}	$Q_{202\text{-}250\text{ mm orifice}}$	$Q_{203\text{-}75\text{ mm orifice}}$	Q_{total}
2	**95**	62	19	14	**95**
5	**121**	79	22	16	**117**
10	**148**	97	23	17	**137**
25	**173**	113	25	18	**156**
50	**195**	127	27	20	**174**
100	**217**	142	30	21	**193**

As shown in Table 6, the post-development peak flows for all storm events are less than the pre-development peak flows, therefore the quantity control criterion is achieved.

Refer to Appendix C for detailed calculations including the storm sewer design sheet.

6.2 Stormwater Quality Control

Stormwater quality controls for the site must incorporate measures to provide an Enhanced Level of Protection (Level 1) according to the MECP (March 2003) guidelines. Enhanced water quality protection involves the removal of at least 80% of total suspended solids (TSS) from 90% of the annual runoff volume. An area breakdown and associated TSS removal rate is provided in Table 7.

Table 7: Area Breakdown and Associated TSS Removal

Catchment		Area (ha)	% of Total Development Area	TSS Removal Efficiency	Total TSS Removal
Catchment 201	Landscape/ Roof Water	0.53	26.5%	80.0%	21.2%
Catchment 202/203	Landscape	0.69	34.5%	80.0%	27.6%
	Jellyfish Filter	0.78	39.0%	89.0%	34.7%
	Total Site	**2.00**	**100.0%**	**-**	**84%**

Catchment 201, which discharges uncontrolled to the ditch, contributes 21.2% TSS removal from the site. Discharge from Catchment 202/203 is divided into landscaped area and impervious area. The discharge from the landscaped area is considered clean water and therefore 80% TSS removal efficiency was applied. The impervious area is routed through a water quality control device which provides 89% TSS removal efficiency. Between the landscaped areas and the Jellyfish Filter, the TSS removal efficiency of the site is 84% therefore achieving the criteria. Refer to Appendix C for details on the water quality control device (Jellyfish Filter) including sizing and ETV certification details.

6.3 Water Balance

The minimum volume requirement to promote water balance is retention of the 5 mm rainfall event. The water balance retention volume was calculated considering initial abstraction of runoff based on various surfaces types.

Water balance for Catchment 201 and the landscaped portion of Catchment 202/203 will be achieved through attenuation of stormwater runoff over the existing natural landscaped area. Water balance for the impervious areas of Catchment 202/203 will be achieved by providing a minimum topsoil depth of 0.30 m over the landscaped areas. A storage volume of approximately 39 m^3 is required to achieve the water balance criteria (5mm x 0.78 ha of impervious area in Catchment 202/203).

The total water balance volume will be stored in the topsoil of the landscaped area in Catchment 202 (0.71 ha). Considering an attenuation rainfall depth of 15mm over an area of 0.71 ha, approximately 107 m^3 of rainfall volume is available to be stored in the topsoil. Using typical topsoil parameters, 150mm of additional topsoil has approximately 160 m^3 of capacity for rainfall storage. Since the capacity of storage in the topsoil and the physical volume of rainfall exceed the required storage volume, we conclude that with a total topsoil depth of 300mm (150mm original + 150mm additional), 39 m^3 of rainfall volume can successfully be retained. Detailed calculations for the topsoil retention are included in Appendix C.

7.0 Erosion and Sediment Controls During Construction

Erosion and sediment controls (ESC) will be installed prior to the start of any construction activities and will be maintained until the site is stabilized or as directed by the Site Engineer or the Town of Caledon. The Erosion & Sediment Control Plan (Drawing C101) identifies the location of the recommended control features. The contractor will inspect the ESC after each significant rainfall event to ensure they are maintained in proper working condition.

Sediment Control Fencing

Sediment control fencing will be installed on the perimeter of the site to intercept sheet flow. Adjacent to the sensitive EPA lands, double silt fence with straw bales will be installed for additional protection. Based on field decisions, the Site Engineer and the Owner may add additional sediment control fencing prior to, during, and following construction.

Rock Mud Mat

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

Filter cloth in Catchbasins

Filter cloth will be installed in the existing nearby storm sewer catch basins. The filter cloth will provide sediment control to prevent silt and sediment from entering the stormwater system. Filter fabric for silt control should be Terra Fix 270R or approved equivalent.

ESC Controls for Outlet

During detailed design ESC controls will be provided as required for the storm outlet. These controls may include products such as coir logs or features such as a level spreader.

ESC Controls for Slope

During detailed design ESC controls will be provided as required for the Catchbasins in proximity to the existing slope.

8.0 Lot Level Controls

Lot level controls can be applied at the individual Plot Plan stage of each lot to further promote at-source retention of stormwater. These measures include:

1. Disconnecting and redirecting roof leaders to rear yard ponding areas, shallow soakaway pits or rain barrels.
2. Sump pumping foundation drains to rear yards, if required
3. Shallow infiltration trenches and soakaway pits
4. Grassed swales and vegetated filter strips

We recommend that these techniques be incorporated into the detailed design.

9.0 Conclusions & Recommendations

The proposed development can be serviced for water, sanitary, and stormwater in accordance with the Town of Caledon requirements and standards. Our conclusions and recommendations include:

1. Water demand for the proposed development will be provided using individual 25 mm domestic water services connected to the proposed 150mm diameter watermain. The proposed watermain will connect to the existing 150mm diameter watermain on Nunnville Road.
2. Fire hydrants are proposed to provide fire protection to the development.
3. Sanitary servicing for the proposed development will be provided using a 250 mm diameter PVC sanitary sub-trunk sewer, which connects to the proposed 900mm diameter sanitary trunk sewer in Nunnville Road. Individual 125 mm diameter laterals will branch off the proposed sub-trunk to service each unit.
4. Stormwater runoff from Catchment 201 will flow uncontrolled to the existing ditch. Stormwater runoff from catchment 202 and 203 will flow controlled into the existing ditch.
5. Quantity control has been provided using two independent orifice pipes downstream of the oversized storm sewers. A maximum storage volume of 668 m^3 is provided for Catchment 202 and 104 m^3 is provided for Catchment 203, which is sufficient to meet post-to-pre storage requirements.
6. A treatment train approach using landscaped areas and a Jellyfish Filter will provide an enhanced level of protection (80% TSS removal) for stormwater quality control for Catchments 201, 202 and 203.
7. Water balance for Catchment 201 and the pervious areas of Catchment 202 and 203 will be provided through attenuation in the existing natural landscaped area. Water balance for the impervious areas of Catchment 202 and 203 will be provided through 39 m^3 of storage in additional topsoil depth.
8. Erosion and Sediment Controls will be implemented on-site during construction and will be maintained until the site is stabilized.

Based on the above conclusions we support the proposed development application from the perspective of water supply, sanitary servicing, and stormwater management.

Respectfully submitted,

C.F. CROZIER & ASSOCIATES INC.

N. Segal

Nicole Segal, M.M.Sc., E.I.T.
Land Development

C.F. CROZIER & ASSOCIATES INC.

K.J. Firth, P.Eng.
Partner

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APPENDIX A

Water Demand Calculations

Connection Demand Table

WATER CONNECTION

Connection point [3)]			
Existing 150mm diameter watermain on Nunnville Road			
Pressure zone of connection point		5	
Total equivalent population to be serviced [1)]		140 persons	
Total lands to be serviced		2 ha	
Hydrant flow test			
	Hydrant flow test location	Nunnville Road	
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	379	92	
Maximum water pressure	448	55	

No.	Water Demand		
	Demand type	**Demand**	**Units**
1	Average day flow	0.45	l/s
2	Maximum day flow	0.91	l/s
3	Peak hour flow	1.36	l/s
4	Fire flow [2)]	45	l/s
Analysis			
5	Maximum day plus fire flow	45.91	l/s

WASTEWATER CONNECTION

Phase 1

Connection point [4)]		
Future 900mm diameter trunk sanitary sewer on Nunnville Road		
Total equivalent population to be serviced		140 persons
Total lands to be serviced		2 ha
6	Wastewater sewer effluent (in l/s)	2.46

[1)] Please refer to design criteria for population equivalencies
[2)] Please reference the Fire Underwriters Survey Document
[3)] Please specify the connection point ID
[4)] Please specify the connection point (wastewater line or manhole ID)
Also, the “total equivalent population to be serviced” and the “total lands to be serviced” should reference the connection point. (the FSR should contain one copy of Site Servicing Plan)

Please include the graphs associated with the hydrant flow test information table
Please provide Professional Engineer's signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

Domestic Water Demand

Site Area:	2.00	ha
Population Density:	70	persons/ha
Number of units:	29	
Population:	140	

Notes & References

Region of Peel Public Works Design, Specifications & Procedures Manual - Linear Infrastructure Sanitary Sewer Design Criteria (March, 2017) - 2.1 (Less than 10m frontage)

Design Parameters

Average Demand (L/capita/d)
280

Region of Peel Public Works Watermain Design Criteria Section 2.3: Water Demands (June, 2010)

Water Demand:

Average Daily Demand =	39,200	L/day
	0.45	**L/s**

Peaking Factors

Max Day =	2.0
Peak Hour =	3.0

Region of Peel Public Works Watermain Design Criteria Section 2.3: Water Demands (June, 2010)

Average Day =	0.45	L/s
Max Day =	**0.91**	L/s
Peak Hour =	**1.36**	L/s

Max Day = Average Day Demand * Max Day
Peak Hour = Average Day Demand * Peak Hour

Municipality	Average Daily Water Demand (L/s)	Max Day Demand (L/s)	Peak Hourly Demand (L/s)
Region of Peel	0.45	0.91	1.36

Project Name:	13247 & 13233 Nunnville Rd
Project No.:	649-5291
Date:	24-Jan-20
Design:	NRS
Design:	NRS

Fire Protection Water Supply Guidelines: Part 3 of OBC (2018)

Nunnville Road Subdivision

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

C Building classification per OBC 3.1.2.1

K = 23 Group C (Table 1 OFM guideline)
V = 2376 264 sq.m. total floor area by 9m height
S_{TOT} = 0.2 S_{TOT} Need Not Exceed 2.0

Q = 10,930 L

Based on ranges listed in Table 2 (OFM guideline), the required minimum water supply flow rate is **2700 L/min**

45 L/s

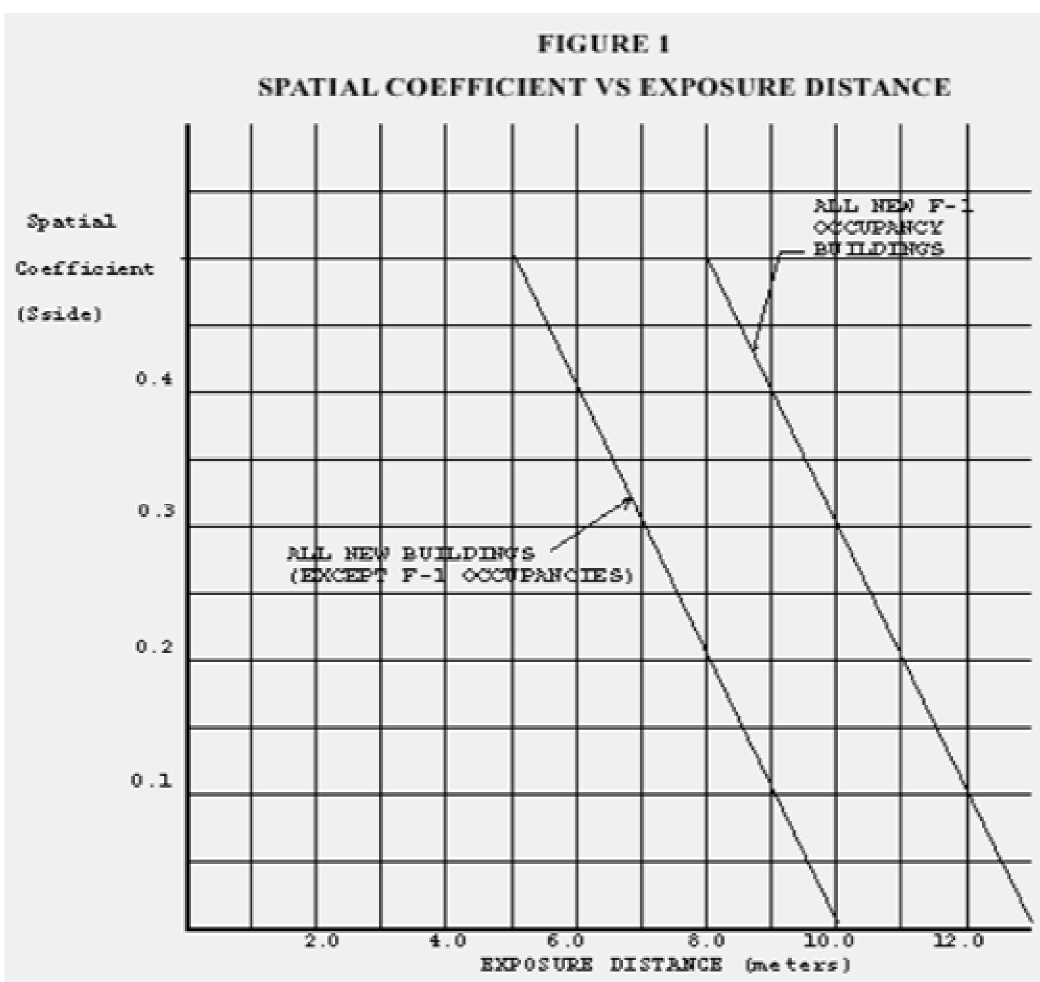


Figure 1 Source:
OFM-TG-03-1999
FIRE PROTECTION WATER SUPPLY GUIDELINE FOR PART 3 IN THE ONTARIO BUILDING CODE, October 1999

CROZIER
CONSULTING ENGINEERS

PROJECT: Nunnville Road
PROJECT No.: 649-5291
DATE: 2019.04.23
UPDATE: 2019.04.23
DESIGN: NAS

Projected Fire Flow Calculations - 1

Test	Hydrant Location / ID	Static Pressure	Residual Pressure during Test	Flow from Hydrant Test	Desired Residual Pressure*	Projected Fire Flow Available at 20 psi	
		Ps	Pt	Qt	Pr	Qr	
		(psi)	(psi)	(USGPM)	(psi)	(USGPM)	L/s
1	Nunnville Road	67	65	870	40	3,547	224
2			62	1207		3,001	189

$Q_r = Q_t \times ((P_s - P_r)/(P_s - P_t))^{0.54}$ Formula to determine available flow as per AWWA M17 (1989)

NOTE: Projected fire flows are calculated on the basis of hydrant tests carried out by Aqualization on April 18, 2019 at 12:15 pm.

Note Region of peel operation pressures 40-100 psi

CROZIER
CONSULTING ENGINEERS

PROJECT: Nunnville Road
PROJECT No.: 649-5291
DATE: 2019.04.23
UPDATE: 2019.04.23
DESIGN: NAS
CHECK:

Projected Fire Flow Calculations - 2

Test	Hydrant Location / ID	Static Pressure	Residual Pressure during Test	Flow from Hydrant Test	Desired Residual Pressure*	Projected Fire Flow Available at 20 psi	
		Ps	Pt	Qt	Pr	Qr	
		(psi)	(psi)	(USGPM)	(psi)	USGPM	L/s
1	Nunnville Road	62	61	870	40	4,618	291
2			55	1459		2,708	171

$Q_r = Q_t \times ((P_s - P_r)/(P_s - P_t))^{0.54}$ Formula to determine available flow as per AWWA M17 (1989)

NOTE: Projected fire flows are calculated on the basis of hydrant tests carried out by Aqualization on April 18, 2019 at 12:15 pm.

Note Region of peel operation pressures 40-100 psi

AQUAZITION

5-200 Connie Cres. Concord ON L4K 1M1 Phone 416-883-9777 Fax 905-303-6977

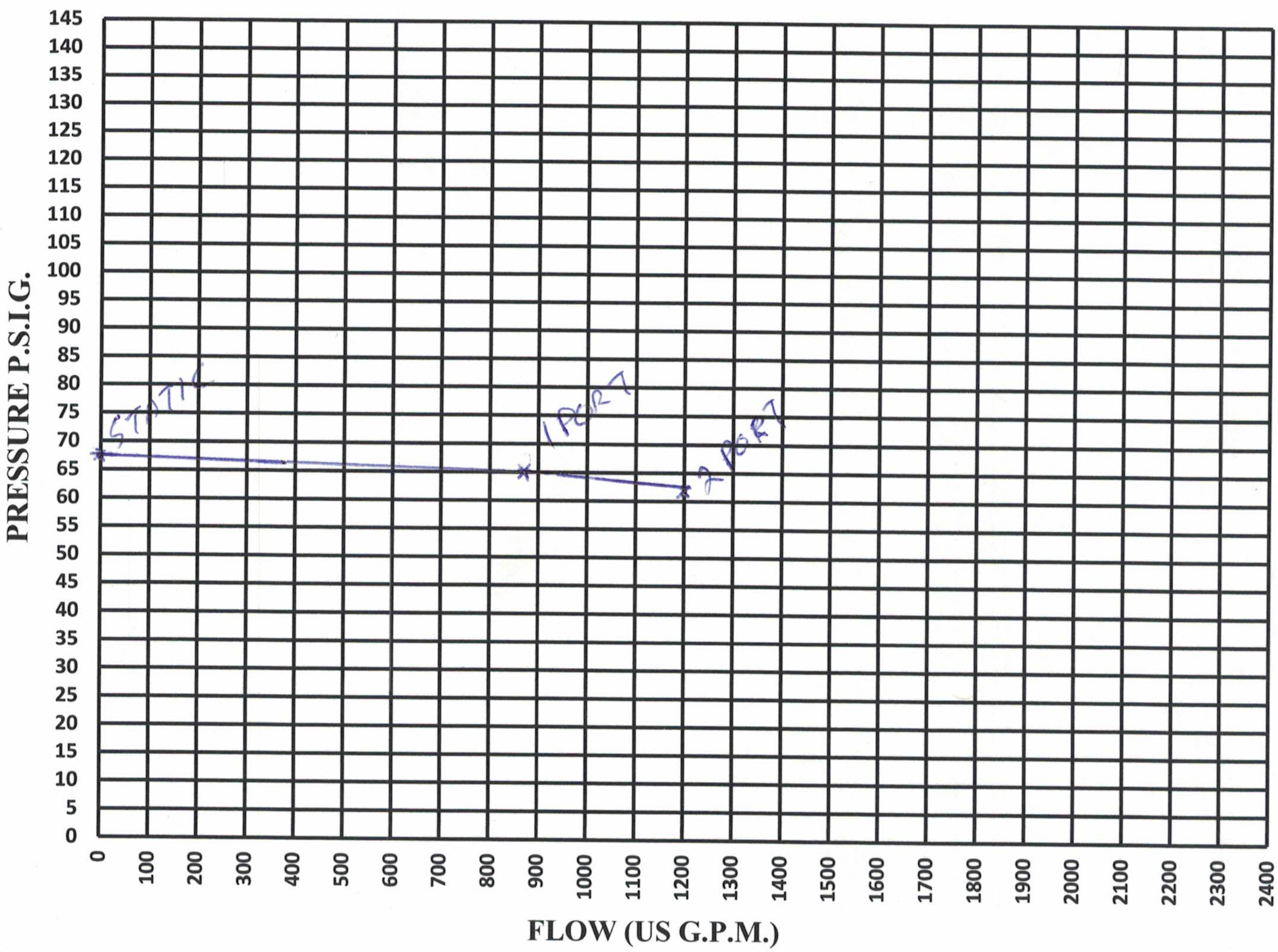
FLOW TEST REPORT

LOCATION OF RESIDUAL HYDRANT 13160 Nunnville Road

LOCATION OF FLOW HYDRANT 13259 Nunnville Road

TIME OF TEST 12:00 WATERMAIN SIZE 150mm STATIC PRESSURE 67

NUMBER OF OUTLETS	PITOT PRESSURE	FLOW (US G.P.M.)	RESIDUAL PRESSURE
One 2 ½" hydrant port	27	870	65
Two 2 ½" hydrant port	13	1207	62



PROJECT LOCATION Nunnville Road, Bolton

DATE 04/18/19

COMPANY NAME SALU Inc.
(PRINT NAME)

AQUAZITION EMPLOYEE JASON WALING
(PRINT NAME)

AQUAZITION

5-200 Connie Cres. Concord ON L4K 1M1 Phone 416-883-9777 Fax 905-303-6977

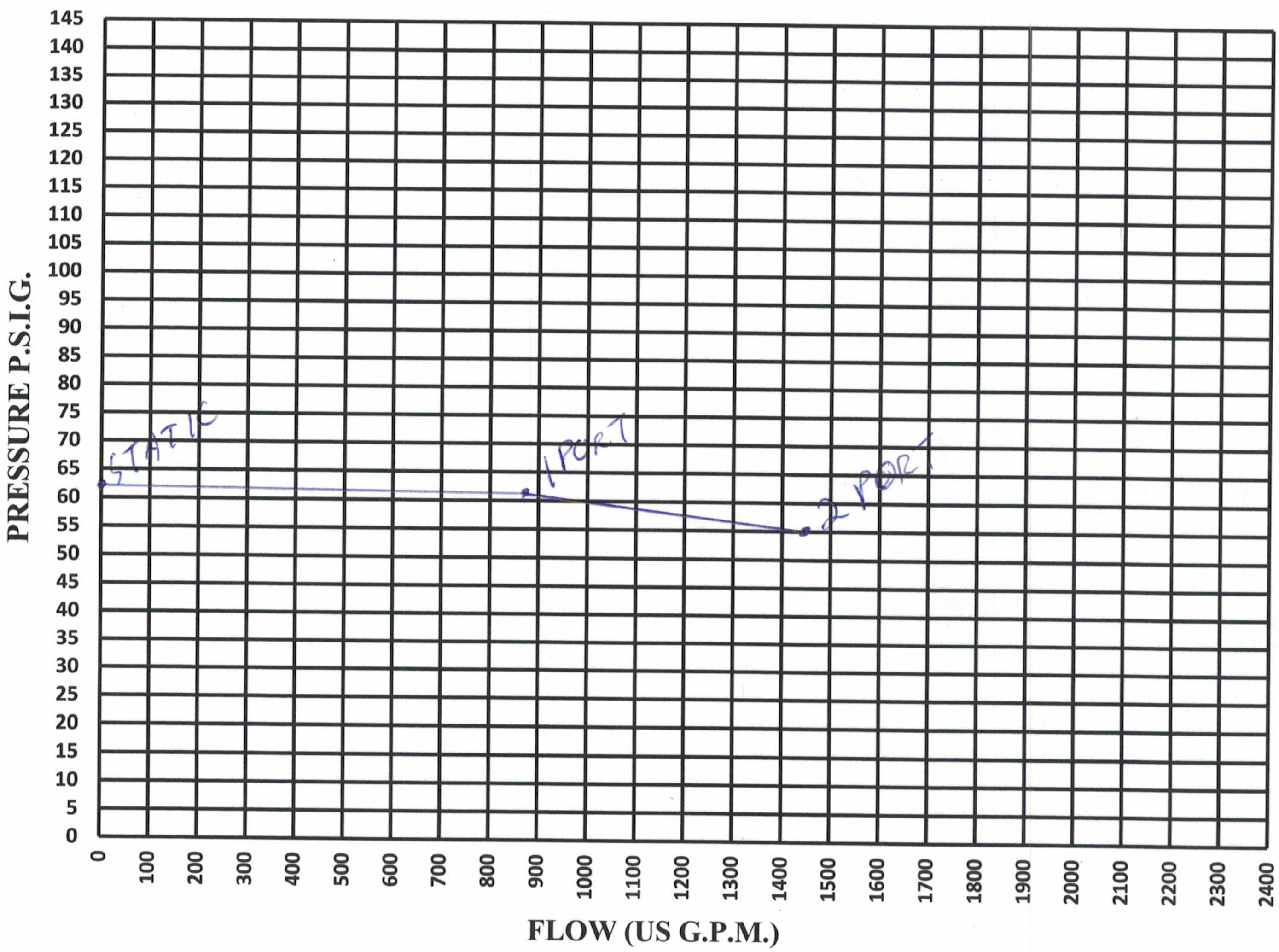
FLOW TEST REPORT

LOCATION OF RESIDUAL HYDRANT 13259 NUNNVILLE RD

LOCATION OF FLOW HYDRANT 13160 NUNNVILLERD

TIME OF TEST 12:15 WATERMAIN SIZE 150 mm STATIC PRESSURE 62

NUMBER OF OUTLETS	PITOT PRESSURE	FLOW (US G.P.M.)	RESIDUAL PRESSURE
One 2 ½" hydrant port	27	870	61
Two 2 ½" hydrant port	19	1459	55



PROJECT LOCATION NUNNVILLE RD BOLTON

DATE 04/18/19

COMPANY NAME SALU INC
(PRINT NAME)

AQUAZITION EMPLOYEE J. MORABIN
(PRINT NAME)

APPENDIX B

Sanitary Demand Calculations

Project: 13247 & 13233 Nunnville Rd
Project No.: 649-5291
Prepared By: MB
Checked By: NC
Date: 2019.02.07
Revised: 2020.01.24

SANITARY CALCULATIONS
13247 & 13233 Nunnville Rd Proposed Residential Development

POPULATION ESTIMATE

Apartment Building

Region of Peel Population Density	70	people/ha
Number of Units	29	units
Total Population	140	persons
Total Developed Area	2.0	ha

SANITARY DESIGN FLOW - REGION OF PEEL METHOD

Average daily demand	302.8	L/person * day
Equivalent Population	140	persons
Harmon Peaking Factor (M)	4.20	
M = 1+(14/(4+p^0.5))		
Average Daily Flow	42392	L/day
	0.49	**L/s**
Peak Flow	178072.27	L/day
	2.06	**L/s**
Infiltration	0.0002	m3/s/ha
	0.0004	m3/s
	0.40	**L/s**
Total Sanitary Flow	**2.46**	**L/s**

References

Region of Peel Public Works Design, Specifications & Procedures Manual - Linear Infrastructure Sanitary Sewer Design Criteria (March, 2017) - 2.1

Region of Peel Public Works Design, Specifications & Procedures Manual - Linear Infrastructure - Sanitary Sewer Design Criteria (March, 2017) - 2.2

Region of Peel Public Works Design, Specifications & Procedures Manual - Linear Infrastructure - Sanitary Sewer Design Criteria (March, 2017) - 2.3

Nicole Segal

From:	Pietkiewicz, Joanna <joanna.pietkiewicz@peelregion.ca>
Sent:	September 18, 2019 12:25 PM
To:	Nicole Segal; Sam Morra
Subject:	RE: Albion Vaughan Phase 2 - Additional sanitary MH on 900mm sanitary trunk
Categories:	Filed to Sharepoint

Thank you, Nicole.
We will incorporate the proposed data into our design and send you our revised drawing for your information.

Thank you,

Joanna Pietkiewicz, P. Eng.
Project Manager, Capital Works
Wastewater Collection and Conveyance
Region of Peel
10 Peel Center Dr.
Brampton, ON L6T 4B9
905-791-7800 ext. 7815

Region of Peel
working with you

From: Nicole Segal <nsegal@cfcrozier.ca>
Sent: September 18, 2019 9:29 AM
To: Sam Morra <sammorra@bell.net>; Pietkiewicz, Joanna <joanna.pietkiewicz@peelregion.ca>
Subject: RE: Albion Vaughan Phase 2 - Additional sanitary MH on 900mm sanitary trunk

Hi Joanna,

The following link contains our servicing linework in CAD, can you overlay it into your design to get the exact location?

https://cfcrozier.sharefile.com/d-s799cda84bd14a74a
PLEASE NOTE THE LINK EXPIRES IN 7 DAYS

We are proposing an invert at the trunk sewer of 283.55. The pipe is sloped at 1% so the invert at property line is 238.60.

Thanks,
Nicole

Nicole Segal M.M.Sc., EIT | Engineering Intern
C.F. Crozier & Associates Consulting Engineers
2800 High Point Drive, Suite 100 | Milton, ON L9T 6P4
cfcrozier.ca | nsegal@cfcrozier.ca
tel: 905.875.0026 ext: 329

CROZIER
CONSULTING ENGINEERS

From: Sam Morra <sammorra@bell.net>
Sent: September 17, 2019 10:47 PM
To: Pietkiewicz, Joanna <joanna.pietkiewicz@peelregion.ca>; Nicole Segal <nsegal@cfcrozier.ca>
Subject: Re: Albion Vaughan Phase 2 - Additional sanitary MH on 900mm sanitary trunk

Hi Joanna

You're welcome.

Hi Nicole

Please provide Joanna with the exact location and invert at the property line of our sanitary sewer.

Thanks
Sam Morra

Sent from my iPhone

On Sep 17, 2019, at 11:07 AM, Pietkiewicz, Joanna <joanna.pietkiewicz@peelregion.ca> wrote:

> Hi Sam,
>
> Thank you for your email response.
>
> I will confirm with Nicole exact location of the proposed maintenance hole and required invert at the property line (we need this information as soon as possible; we are at 90% detailed design stage now).
>
> Based on your information provided in your email we will propose to install the 1800mm dia maintenance hole with complete benching, drop pipe and a stub to or past property line (approx. 1m) based on the confirmation that the sewer will be constructed by open cut. We need the proposed invert as soon as possible.
>
> The implementation of the maintenance hole and stub will likely require localized relocation of the underground Bell service lines (BS dashed line) or abandonment if the lots are vacant as per the sketch shown below.
>
> Thank you for your agreement to acquire for temporary easement. I will ask Melissa Turner, our Real Property Agent, to contact you with regards to this location.

<image004.jpg>

Thank you,

Joanna Pietkiewicz, P. Eng.
Project Manager, Capital Works
Wastewater Collection and Conveyance
Region of Peel
10 Peel Center Dr.
Brampton, ON L6T 4B9
905-791-7800 ext. 7815

<image001.gif>

From: sammorra sammorra <sammorra@bell.net>
Sent: September 16, 2019 10:12 PM
To: Pietkiewicz, Joanna <joanna.pietkiewicz@peelregion.ca>
Cc: Nicole Segal <nsegal@cfcrozier.ca>
Subject: Re: Albion Vaughan Phase 2 - Additional sanitary MH on 900mm sanitary trunk

Hi Joanna

Many of your issues below involve input from me so I will answer the questions as follows and Nicole can provide additional input when she is able:

1. Nicole will give you the exact station and invert for the 250mm sewer and the stub to the property line.

2. We are proposing to construct the 250mm dia pipe by open cut. We appreciate the 1800mm dia benched Maintenance hole with drop pipe and a stub to property line.

3. Our Proposed construction commencement will be in late 2020 or early 2021. The lots will be vacant around the same time.

4. Yes, the Region has been cooperative so I would be agreeable to the Region acquiring a temporary easement to facilitate construction even if the houses are not vacant.

5. I'm trying to visualize the issue with the Bell underground service lines. Perhaps you can send us a sketch of the potential conflict?

Please let us know if you require anything further.

Thanks

Sam Morra, P. Eng.

President

Bolton Midtown Developments Inc.

---------- Original Message ----------
From: "Pietkiewicz, Joanna" <joanna.pietkiewicz@peelregion.ca>
Date: September 13, 2019 at 3:16 PM

Hi Nicole,

Before our Consultant get started on the design, please confirm the following:

1. Exact station for the MH and required invert for the 250mm sewer as well as stub (if required) slope; the given station(4+480) does not match your preliminary design drawing (4+474)
2. How you are proposing to construct 250mm dia pipe, by open cut? We would then construct 1800mm dia maintenance hole with complete benching, drop pipe and a stub
3. What is your proposed construction commencement, when the lots will be vacant?
4. Would it be possible to acquire a temporary easement, if vacant, to facilitate our construction?
5. Please note implementation of the MH will likely require localized relocation of the underground Bell service lines/ abandonment if the lots are vacant in time

I have meeting next week on Wednesday with my Consultant and would appreciate to have your response.

Thank you,

Joanna Pietkiewicz, P. Eng.

Project Manager, Capital Works

Wastewater Collection and Conveyance

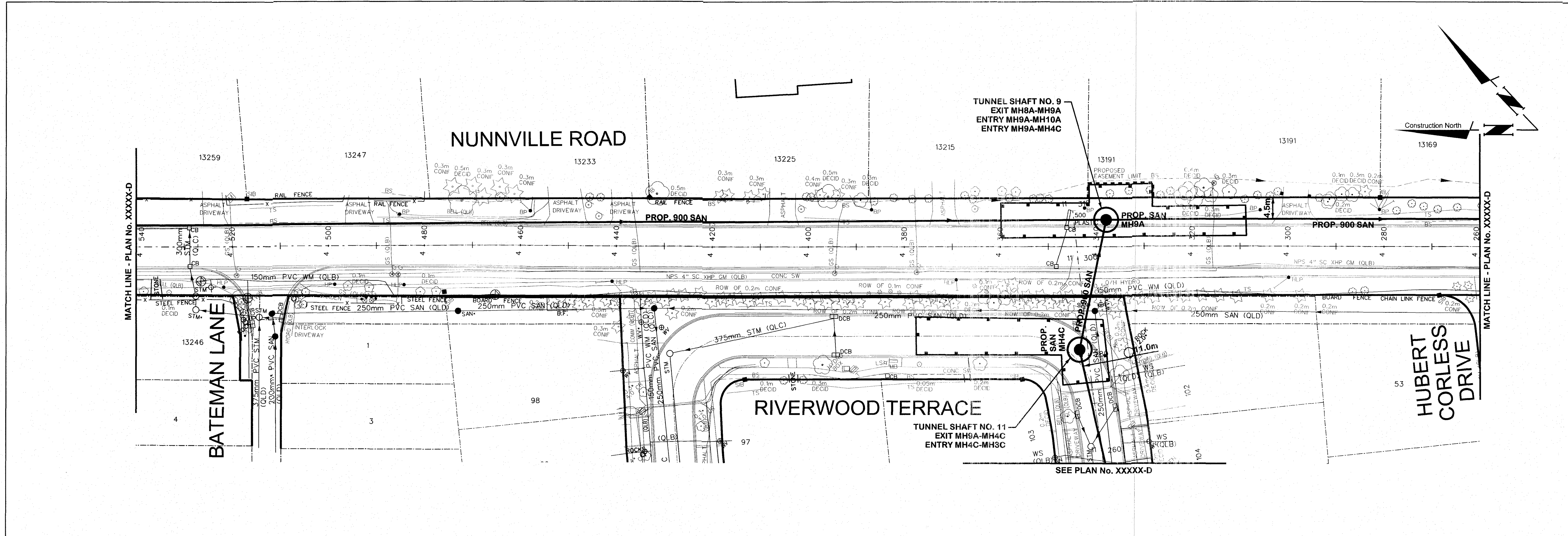
Region of Peel

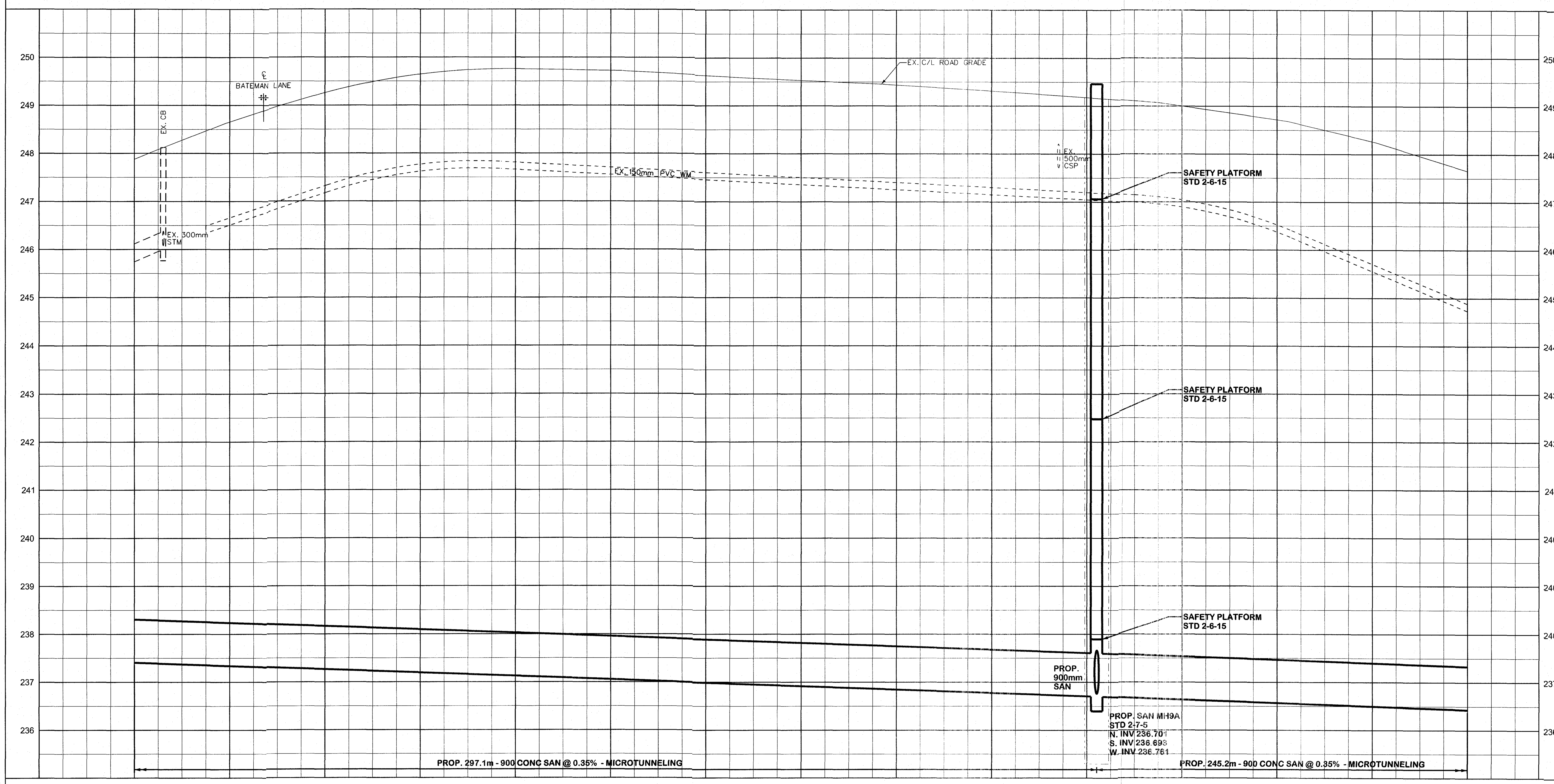
10 Peel Center Dr.

Brampton, ON L6T 4B9

905-791-7800 ext. 7815

<image001.gif>





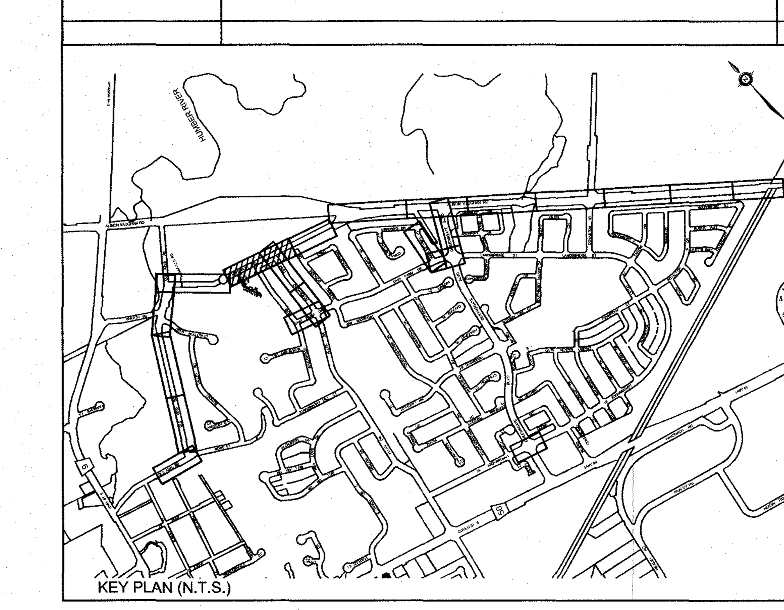
247.87	248.65	249.26	249.64	249.75	249.72	249.63	249.53	249.44	249.31	249.18	249.00	248.71	248.27	247.65	EX. ROAD ELEV.
4+540	4+520	4+500	4+480	4+460	4+440	4+420	4+400	4+380	4+360	4+340	4+320	4+300	4+280	4+260	ROAD CHAINAGE

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS			GAS MAINS		
STORM SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		
TRANSIT			HYDRO ONE		
PARKS & REC.			CTV		
ONT. CLEAN WATER			COMMUNIC. CABLES		

REVISIONS

DATE	DETAILS	INIT.
JANUARY 2019	50% DESIGN SUBMISSION	S.K.


KEY PLAN (N.T.S.)

Chisholm, Fleming and Associates
consulting engineers

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
20C Existing Water Service, Size In mm
WS25 Proposed Water Service, Size In mm
B.M. No. 37 Elev. 227.187m
Description
Location: North face at the east corner of a red insul. brick house # 11970 located on the west side of Highway 50 approx. 0.30 km south of Mayfield Road
The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by Chkd. Approved by

NOTICE TO CONTRACTOR
48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

Region of Peel
working with you

TRUNK SANITARY SEWERS

(FROM NORTH OF CP RAIL TO BOLTON SPS)

NUNNVILLE ROAD
PROP. 900mm DIA. TRUNK SANITARY SEWER

STA. 4+260 TO STA. 4+540

CAD Area	X-XX	Area	X-XX	Project No.	17-2280 & 17-2192
Checked by	N.M.	Drawn by	S.K.		
Date	JULY 2018	Sheet	10 of 33	Plan No.	XXXXX-D

APPENDIX C

Stormwater Design Calculations

Stormwater Quality Control Summary

Catchment		Area (ha)	% of Total Development Area	TSS Removal Efficiency	Total TSS Removal
Catchment 201	Landscape/Roof Water	0.53	26.5%	80.0%	21.2%
Catchment 202/203	Landscape	0.69	34.5%	80.0%	27.6%
	Impervious Area	0.78	39.0%	89.0%	34.7%
	Total Site	**2.00**	**100.0%**	**-**	**84%**

Enhanced Topsoil Design

Storage Required	Total Area of Additional Topsoil	Extra Topsoil Depth[1]	Soil Volume	Soil Porosity*	Soil Field Capacity*	Avalable Storage Volume[2]
m3	ha	mm	m3			m3
39.00	0.71	150.00	1065.00	0.47	0.32	159.75

*Geotechnical Consultant to confirm site-specific topsoil design parameters

1. Topsoil depth in addition to the 150mm minimum covering the site, total topsoil depth = 300mm (150mm + 150mm = 300mm topsoil)
2. Water volume that can be stored for a given soil = (soil volume) x (soil porosity – soil field capacity)

Total Contributing Area:	0.71 ha	
Allowable Infiltration Rainfall Depth:	15 mm	As per Chapter 4.5.5 from MOE SWMPG (4 hour 15mm event)
Available Runoff Volume to be infiltrated:	106.5 m3	

CROZIER CONSULTING ENGINEERS

Nunnville Road
STORM SEWER DESIGN SHEET

100 YEAR DESIGN STORM - TOWN OF CALEDON[1]

A	**4688**	**B**	**17**	**C**	**0.9624**

PROJECT: Nunnville Road
PROJECT No.: 649-5291
FILE: Storm Sewer Design
DATE: July 31, 2019
Revised: January 24, 2020
Design: HL
Review: NRS

INITIAL TIME OF CONCENTRATION (min) 10.00

MANNINGS "n" 0.013

FROM MH	TO MH	AREA (A) Ha	RUN-OFF COEFF	A x C	Cummul. A x C	TIME OF CONC. min	I mm/hr	Q m3/sec	LENGTH m	SLOPE %	PIPE DIA. mm	VEL. m/sec	INITIAL Tc min	TIME OF CONC min	ACC. TIME OF CONC. min	CAPACITY m3/s	FALL m	% Capacity
MH A1	MH A2	1.14	0.60	0.68	0.68	10.00	196.54	0.37	145.10	0.50	2400	3.87	10.00	0.62	10.62	**17.50**	0.73	2%
MH A2	MH A3 (JF8)	0.07	0.60	0.04	0.73	10.62	192.26	0.39	9.70	0.50	2400	3.87	10.62	0.04	10.67	**17.50**	0.05	2%
MH A6	MH A5	0.16	0.60	0.10	0.10	10.00	196.54	0.05	11.80	0.50	2400	3.87	10.00	0.05	10.05	**17.50**	0.06	0%
MH A5	MH A4	0.06	0.60	0.04	0.13	10.05	196.18	0.07	15.40	0.50	2400	3.87	10.05	0.07	10.12	**17.50**	0.08	0%
MH A4	MH A3 (JF8)	0.04	0.60	0.02	0.15	10.12	195.72	0.08	8.40	0.50	2400	3.87	10.12	0.04	10.15	**17.50**	0.04	0%
MH A3 (JF8)	OUTLET 3							0.05	55.7	0.50	375	1.12	0.00	0.83	0.83	**0.12**	0.28	41%

1. A, B, and C coefficients as per Town of Caledon Design Requirements.

Superpipe Storage Calculations - Catchment 202

100-year Storm Elevation: 244.76 **masl**

Storm Sewer Network Parameters							Water Depth at Sewer Invert (m)		Water-Filled Area in Sewer (m^2)		Storage Volume in Sewer (m^3)
To	From	Length (m)	Slope (%)	DS Invert	US Invert	Size (mm)	DS Invert	US Invert	DS Invert	US Invert	
A2	A1	145.1	0.5	241.96	242.69	2400	2.80	2.07	4.52	4.14	628.7
A3	A2	8.7	0.5	241.83	241.88	2400	2.93	2.88	4.52	4.52	39.4
											668.0

Note:

1. Water Depth in each sewer is calculated as Storm Event Water Elevation - Invert Elevation (DS or US). In cases where the sewer invert is above the storm water elevation, the water depth is equal to 0.
2. Water-Filled Areas are calculated using the following equation, where R = Sewer radius (m) and h = Water depth in sewer (m):

$$\text{Area} = R^2 \cos^{-1}\left(\frac{R-h}{R}\right) - (R-h)\sqrt{2Rh - h^2}$$

3. In cases where the sewer cross-section is full, the Water-Filled Area is calculated as π*R2.
4. MH structure is not included in storage calculation, each MH structure provides approximately 8m^3 storage.
5. Storage Volume is calculated as the Sewer Length multiplied by the average of the DS and US Water-Filled Areas.

Total Storage in the system is calculated as the sum of the storage volume in each sewer.

CROZIER
CONSULTING ENGINEERS

Project: 13247&13233 Nunnville Road, Bolton
Project No.: 0649-5291

Created By: HL/JT
Checked By: RA/NSR

Date: 2020-01-24
Updated: 2020-01-24

Superpipe Storage Calculations - Catchment 203

100-year Storm Elevation: 243.52 **masl**

Storm Sewer Network Parameters								Water Depth at Sewer Invert (m)		Water-Filled Area in Sewer (m^2)		Storage Volume in Sewer (m^3)
To	From	Length (m)	Slope (%)	DS Invert	US Invert	Size (mm)	Size (m)	DS Invert	US Invert	DS Invert	US Invert	
A3	A4	7.4	0.5	241.83	241.87	2400	2.4	1.69	1.65	3.41	3.32	24.9
A4	A5	15.4	0.5	241.95	242.03	2400	2.4	1.57	1.49	3.14	2.95	46.9
A5	A6	11.8	0.5	242.11	242.17	2400	2.4	1.41	1.35	2.76	2.62	31.8
												103.5

Note:

1. Water Depth in each sewer is calculated as Storm Event Water Elevation - Invert Elevation (DS or US). In cases where the sewer invert is above the storm water elevation, the water depth is equal to 0.
2. Water-Filled Areas are calculated using the following equation, where R = Sewer radius (m) and h = Water depth in sewer (m):

$$\text{Area} = R^2 \cos^{-1}\left(\frac{R-h}{R}\right) - (R-h)\sqrt{2Rh - h^2}$$

3. In cases where the sewer cross-section is full, the Water-Filled Area is calculated as π*R2.
4. MH structure is not included in storage calculation, each MH structure provides approximately 8m^3 storage.
5. Storage Volume is calculated as the Sewer Length multiplied by the average of the DS and US Water-Filled Areas.

Total Storage in the system is calculated as the sum of the storage volume in each sewer.

CROZIER
CONSULTING ENGINEERS

Project: Nunnville Road
Project No.: 649-5291
Created By: NRS
Modified by: HL/NRS
Date: 2019-06-05
Updated: 2020-01-24

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	8	0.8759	85.72
5 yr	1593	11	0.8789	109.68
10 yr	2221	12	0.9080	134.16
25 yr	3158	15	0.9335	156.47
50 yr	3886	16	0.9495	176.19
100 yr	4688	17	0.9624	196.54

Pre - Development Conditions - Catchment 102				
Catchment ID	**Area** (ha)	**Area** (m^2)	C	**Weighted Average C**[1]
101	1.58	15800	0.25	0.25
Total Site	**1.58**	15800	-	**0.25**

Note: Currently only 102 drains to outlet; Catchment 101 drains north and 103 drains south

Post- Development Conditions				
Catchment ID	**Area** (ha)	**Area** (m^2)	**C**	**Weighted Average C**[1]
201 (Uncontrolled)	0.53	5300	0.49	0.13
202 (Controlled)	1.21	12100	0.60	0.36
203 (Controlled)	0.26	2600	0.60	0.08
Total Site	**2.00**	20000	--	**0.57**

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: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Target Flow Rates Summary

Storm Event (yr)	Flow Rates (L/s)					
	Uncontrolled		Target Flow	Controlled Flow		
	Q_{101}	Q_{201}	$Q_{202,203} = Q_{101} - Q_{201}$	$Q_{202\text{-}250\ mm\ orifice}$	$Q_{203\text{-}75\ mm\ orifice}$	Q_{total}
2	95	62	33	19	14	33
5	121	79	42	22	16	38
10	148	97	48	23	17	39
25	173	113	60	25	18	43
50	195	127	68	27	20	47
100	217	142	71	30	21	51

Storm Event (yr)	Catchment 202			
	Flowrate (250mm Orifice Tube) L/s	High Water Level m	Required Storage m^3	Provided Storage m^3
2	19	243.12	216	668
5	22	243.45	330	
10	23	243.68	407	
25	25	244.00	514	
50	27	244.27	587	
100	30	244.76	663	

Storm Event (yr)	Catchment 203			
	Flowrate (75mm Orifice Tube) L/s	High Water Level m	Required Storage m^3	Provided Storage m^3
2	14	242.72	29	104
5	16	242.80	45	
10	17	242.97	58	
25	18	243.19	75	
50	20	243.34	88	
100	21	243.52	103	

Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 100-Year Storm Event

Control Criteria

100 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

100 yr: Uncontrolled Post-Development Flow:

Q_{201} = 141.7 L/s | Q_{203} = 85.8 L/s

Q_{202} = 399.5 L/s

100 yr: Target Flow Rate:

Q_{101} = 217 L/s

$Q_{target\ 202,203}$ = 75.6 L/s

$Q_{orifice\ 202}$= 29.8 L/s (250mm orifice) | $Q_{orifice\ 203}$= 21 L/s (75mm orifice)

HWL_{202}= 244.8 m | HWL_{203}= 243.5 m

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	0.516	291.8
20	145.13	1200	0.381	430.6
30	115.28	1800	0.303	509.2
40	95.75	2400	0.251	558.8
50	81.95	3000	0.215	592.1
60	71.69	3600	0.188	615.2
70	63.74	4200	0.167	631.5
80	57.40	4800	0.151	643.1
90	52.23	5400	0.137	651.3
100	47.93	6000	0.126	656.8
110	44.29	6600	0.116	660.3
120	41.17	7200	0.108	662.3
130	38.47	7800	0.101	662.9
140	36.11	8400	0.095	662.5
150	34.03	9000	0.089	661.2
160	32.18	9600	0.085	659.2
Required Storage Volume:				**662.9**

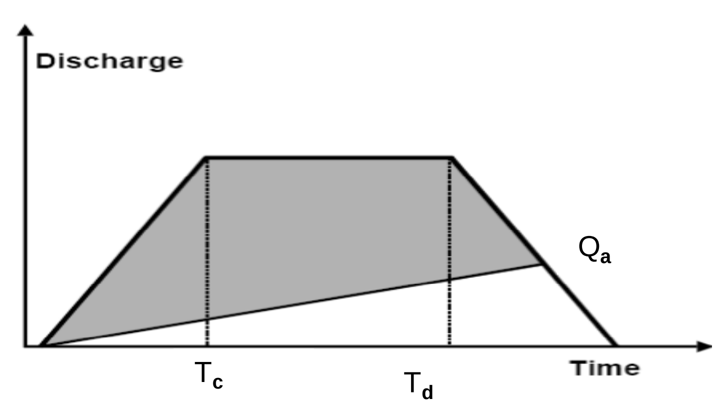
Storage Volume Determination				
T_d (min)	i (mm/hr)	T_d (sec)	Q_{Uncont} (m^3/s)	S_d (m^3)
10	196.54	600	0.111	54.3
20	145.13	1200	0.082	79.8
30	115.28	1800	0.065	92.5
40	95.75	2400	0.054	98.9
50	81.95	3000	0.046	101.9
60	71.69	3600	0.040	102.6
70	63.74	4200	0.036	101.9
80	57.40	4800	0.032	100.1
90	52.23	5400	0.029	97.7
100	47.93	6000	0.027	94.6
110	44.29	6600	0.025	91.2
120	41.17	7200	0.023	87.4
130	38.47	7800	0.022	83.3
140	36.11	8400	0.020	78.9
150	34.03	9000	0.019	74.4
160	32.18	9600	0.018	69.8
Required Storage Volume:				**102.6**

Peak Flow

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

Storage

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



CROZIER CONSULTING ENGINEERS

Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 50-Year Storm Event

Control Criteria

50 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

50 yr: Uncontrolled Post-Development Flow:

Q_{201} = 127.1 L/s Q_{203} = 77.0 L/s

Q_{202} = 358.2 L/s

50 yr: Target Flow Rate:

Q_{101} = 195 L/s

$Q_{target\ 202,203}$ = 67.8 L/s

$Q_{orifice\ 202}$= 27.1 L/s (250mm orifice) $Q_{orifice\ 203}$= 20 L/s (75mm orifice)

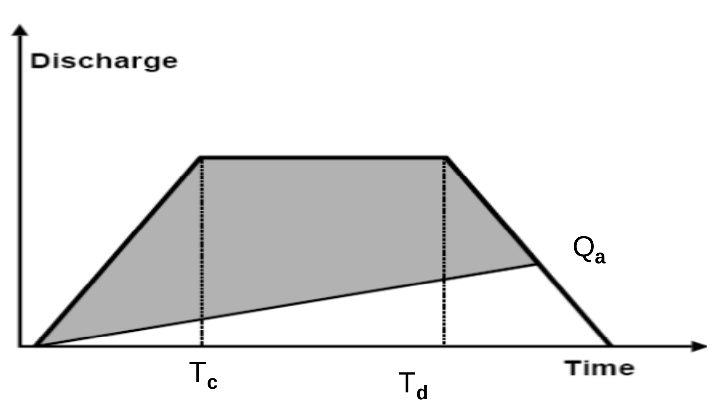
HWL_{202}= 244.3 m HWL_{203}= 243.3 m

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	0.463	261.4
20	129.36	1200	0.340	383.3
30	102.50	1800	0.269	452.0
40	85.04	2400	0.223	495.3
50	72.75	3000	0.191	524.4
60	63.63	3600	0.167	544.7
70	56.58	4200	0.149	559.1
80	50.97	4800	0.134	569.4
90	46.40	5400	0.122	576.7
100	42.59	6000	0.112	581.7
110	39.37	6600	0.103	584.9
120	36.62	7200	0.096	586.7
130	34.23	7800	0.090	587.4
140	32.15	8400	0.084	587.2
150	30.30	9000	0.080	586.2
160	28.67	9600	0.075	584.5
Required Storage Volume:				**587.4**

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	0.099	48.0
20	129.36	1200	0.073	70.1
30	102.50	1800	0.058	80.7
40	85.04	2400	0.048	85.9
50	72.75	3000	0.041	88.1
60	63.63	3600	0.036	88.3
70	56.58	4200	0.032	87.3
80	50.97	4800	0.029	85.4
90	46.40	5400	0.026	82.9
100	42.59	6000	0.024	79.9
110	39.37	6600	0.022	76.5
120	36.62	7200	0.021	72.7
130	34.23	7800	0.019	68.8
140	32.15	8400	0.018	64.6
150	30.30	9000	0.017	60.3
160	28.67	9600	0.016	55.9
Required Storage Volume:				**88.3**

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

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



Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 25 -Year Storm Event

Control Criteria

25 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

25 yr: Uncontrolled Post-Development Flow:

Q_{201} = 113 L/s Q_{203} = 68 L/s

Q_{202} = 318 L/s

25 yr: Target Flow Rate:

Q_{101} = 173 L/s

$Q_{target\ 202,203}$ = 60 L/s

$Q_{orifice\ 202}$= 25 L/s (250mm orifice) $Q_{orifice\ 203}$= 18 L/s (75mm orifice)

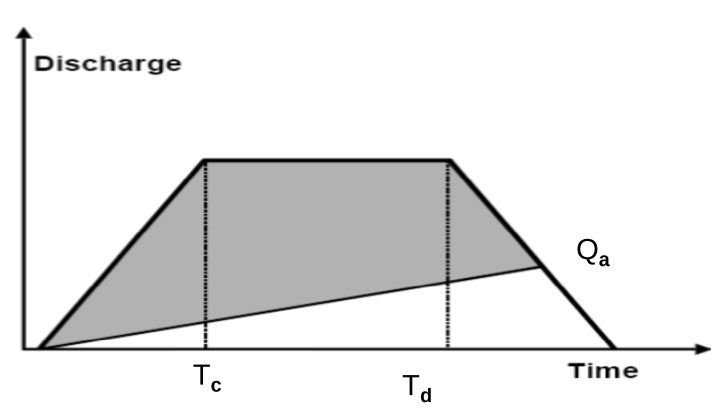
HWL_{202}= 244.0 m HWL_{203}= 243.2 m

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	0.411	231.3
20	114.29	1200	0.300	337.3
30	90.39	1800	0.237	396.8
40	74.95	2400	0.197	434.2
50	64.13	3000	0.168	459.4
60	56.11	3600	0.147	477.0
70	49.92	4200	0.131	489.5
80	45.00	4800	0.118	498.5
90	40.99	5400	0.108	504.8
100	37.65	6000	0.099	509.2
110	34.83	6600	0.091	512.0
120	32.41	7200	0.085	513.6
130	30.32	7800	0.080	514.2
140	28.49	8400	0.075	513.9
150	26.88	9000	0.071	513.0
160	25.44	9600	0.067	511.5
Required Storage Volume:				**514.2**

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	0.088	41.9
20	114.29	1200	0.065	60.8
30	90.39	1800	0.051	69.7
40	74.95	2400	0.042	73.9
50	64.13	3000	0.036	75.5
60	56.11	3600	0.032	75.4
70	49.92	4200	0.028	74.2
80	45.00	4800	0.025	72.2
90	40.99	5400	0.023	69.7
100	37.65	6000	0.021	66.8
110	34.83	6600	0.020	63.5
120	32.41	7200	0.018	60.0
130	30.32	7800	0.017	56.2
140	28.49	8400	0.016	52.3
150	26.88	9000	0.015	48.2
160	25.44	9600	0.014	44.0
Required Storage Volume:				**75.5**

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

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



CROZIER
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Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 10 -Year Storm Event

Control Criteria

10 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

10 yr: Uncontrolled Post-Development Flow:

Q_{201} = 97 L/s Q_{203} = 59 L/s

Q_{202} = 273 L/s

10 yr: Target Flow Rate:

Q_{101} = 148 L/s

$Q_{target\ 202,203}$ = 52 L/s

$Q_{orifice\ 202}$= 23 L/s (300mm orifice) $Q_{orifice\ 203}$= 17 L/s (75mm orifice)

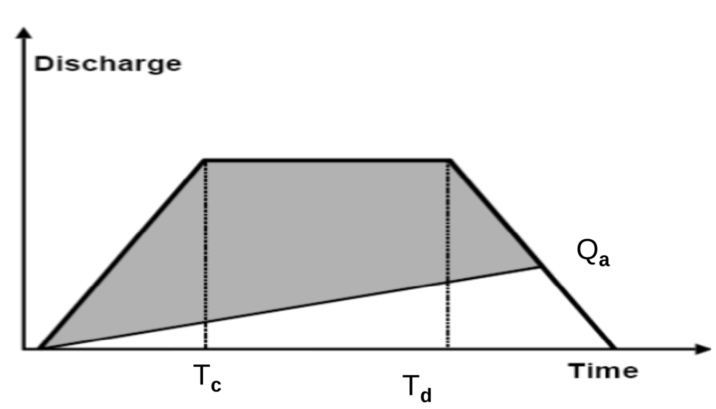
HWL_{202}= 243.7 m HWL_{203}= 243.0 m

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	0.352	197.4
20	95.47	1200	0.251	279.9
30	74.58	1800	0.196	324.5
40	61.44	2400	0.161	352.2
50	52.37	3000	0.138	370.5
60	45.72	3600	0.120	383.2
70	40.63	4200	0.107	392.0
80	36.60	4800	0.096	398.2
90	33.32	5400	0.088	402.4
100	30.61	6000	0.080	405.2
110	28.32	6600	0.074	406.8
120	26.37	7200	0.069	407.4
130	24.68	7800	0.065	407.3
140	23.20	8400	0.061	406.5
150	21.89	9000	0.058	405.2
160	20.73	9600	0.054	403.5
Required Storage Volume:				**407.4**

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	0.076	35.3
20	95.47	1200	0.054	49.4
30	74.58	1800	0.042	55.5
40	61.44	2400	0.035	57.9
50	52.37	3000	0.030	58.2
60	45.72	3600	0.026	57.4
70	40.63	4200	0.023	55.7
80	36.60	4800	0.021	53.5
90	33.32	5400	0.019	50.9
100	30.61	6000	0.017	47.9
110	28.32	6600	0.016	44.7
120	26.37	7200	0.015	41.2
130	24.68	7800	0.014	37.6
140	23.20	8400	0.013	33.9
150	21.89	9000	0.012	30.1
160	20.73	9600	0.012	26.1
Required Storage Volume:				**58.2**

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

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



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Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 5 -Year Storm Event

Control Criteria

5 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

5 yr: Uncontrolled Post-Development Flow:

Q_{201} = 79 L/s　　　Q_{203} = 48 L/s

Q_{202} = 223 L/s

5 yr: Target Flow Rate:

Q_{101} = 121 L/s

$Q_{target\ 202,203}$ = 42 L/s

$Q_{orifice\ 202}$= 22 L/s (250mm orifice)　　　$Q_{orifice\ 203}$= 16 L/s (75mm orifice)

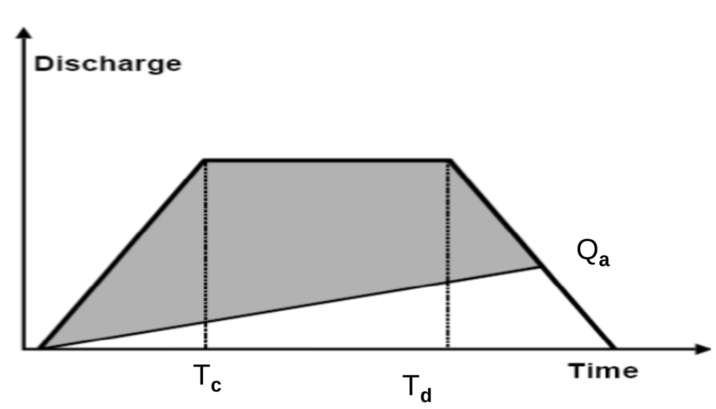
HWL_{202}= 243.5 m　　　HWL_{203}= 242.8 m

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	0.288	159.7
20	77.89	1200	0.205	225.8
30	60.92	1800	0.160	261.8
40	50.28	2400	0.132	284.2
50	42.96	3000	0.113	299.2
60	37.60	3600	0.099	309.6
70	33.48	4200	0.088	317.0
80	30.23	4800	0.079	322.1
90	27.58	5400	0.072	325.7
100	25.39	6000	0.067	328.0
110	23.53	6600	0.062	329.3
120	21.95	7200	0.058	329.9
130	20.57	7800	0.054	329.8
140	19.37	8400	0.051	329.1
150	18.31	9000	0.048	328.0
160	17.36	9600	0.046	326.5
Required Storage Volume:				**329.9**

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	0.062	27.8
20	77.89	1200	0.044	38.8
30	60.92	1800	0.034	43.3
40	50.28	2400	0.028	44.9
50	42.96	3000	0.024	44.8
60	37.60	3600	0.021	43.8
70	33.48	4200	0.019	42.2
80	30.23	4800	0.017	40.0
90	27.58	5400	0.016	37.6
100	25.39	6000	0.014	34.8
110	23.53	6600	0.013	31.9
120	21.95	7200	0.012	28.7
130	20.57	7800	0.012	25.5
140	19.37	8400	0.011	22.1
150	18.31	9000	0.010	18.6
160	17.36	9600	0.010	15.0
Required Storage Volume:				**44.9**

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

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



Project: Nunnville Road
Project No.: 649-5291
Created By: HL
Reviewed By: NRS
Date: 2020-01-23
Updated: 2020-01-24

Modified Rational Calculations - 2 -Year Storm Event

Control Criteria

2 yr: Control Post-Development Peak Flows to Pre-Development Peak Flow

2 yr: Uncontrolled Post-Development Flow:

Q_{201} = 62 L/s

Q_{202} = 174 L/s

Q_{203} = 37 L/s

2 yr: Target Flow Rate:

Q_{101} = 95 L/s

$Q_{target\ 202,203}$ = 32.98 L/s

$Q_{orifice\ 202}$= 19 L/s (250mm orifice)

HWL_{202}= 243.1 m

$Q_{orifice\ 203}$= 14 L/s (75mm orifice)

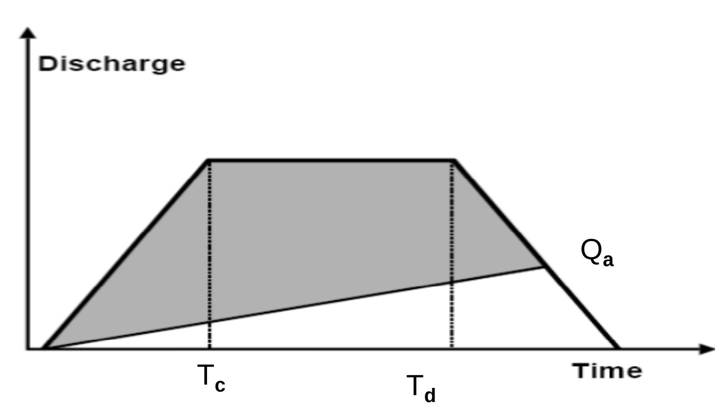
HWL_{203}= 242.7 m

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	0.225	123.5
20	58.06	1200	0.152	165.7
30	44.38	1800	0.117	186.7
40	36.14	2400	0.095	198.9
50	30.60	3000	0.080	206.5
60	26.62	3600	0.070	211.2
70	23.60	4200	0.062	214.1
80	21.23	4800	0.056	215.6
90	19.31	5400	0.051	216.2
100	17.74	6000	0.047	216.0
110	16.41	6600	0.043	215.2
120	15.28	7200	0.040	213.9
130	14.30	7800	0.038	212.2
140	13.45	8400	0.035	210.2
150	12.70	9000	0.033	207.9
160	12.04	9600	0.032	205.4
Required Storage Volume:				**216.2**

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	0.048	20.9
20	58.06	1200	0.033	27.1
30	44.38	1800	0.025	28.8
40	36.14	2400	0.020	28.5
50	30.60	3000	0.017	27.3
60	26.62	3600	0.015	25.5
70	23.60	4200	0.013	23.3
80	21.23	4800	0.012	20.8
90	19.31	5400	0.011	18.1
100	17.74	6000	0.010	15.2
110	16.41	6600	0.009	12.2
120	15.28	7200	0.009	9.0
130	14.30	7800	0.008	5.8
140	13.45	8400	0.008	2.6
150	12.70	9000	0.007	-0.8
160	12.04	9600	0.007	-4.1
Required Storage Volume:				**28.8**

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

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



Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

Jellyfish® Filter JF4-2-1

Developed by Imbrium Systems, Inc.,
Whitby, Ontario, Canada

In accordance with

ISO 14034:2016

Environmental management —
Environmental technology verification (ETV)

John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

August 3, 2017
Vancouver, BC, Canada

GLOBE PERFORMANCE SOLUTIONS

Verification Body
GLOBE Performance Solutions
404 – 999 Canada Place | Vancouver, B.C | Canada |V6C 3E2

Technology description and application

The Jellyfish® Filter is an engineered stormwater quality treatment technology designed to remove a variety of stormwater pollutants including floatable trash and debris, oil, coarse and fine suspended sediments, and particulate-bound pollutants such as nutrients, heavy metals, and hydrocarbons. The Jellyfish Filter combines gravitational pre-treatment (sedimentation and floatation) and membrane filtration in a single compact structure. The system utilizes membrane filtration cartridges comprised of multiple pleated filter elements ("filtration tentacles") that provide high filtration surface area with the associated advantages of high flow rate, high sediment capacity, and low filtration flux rate.

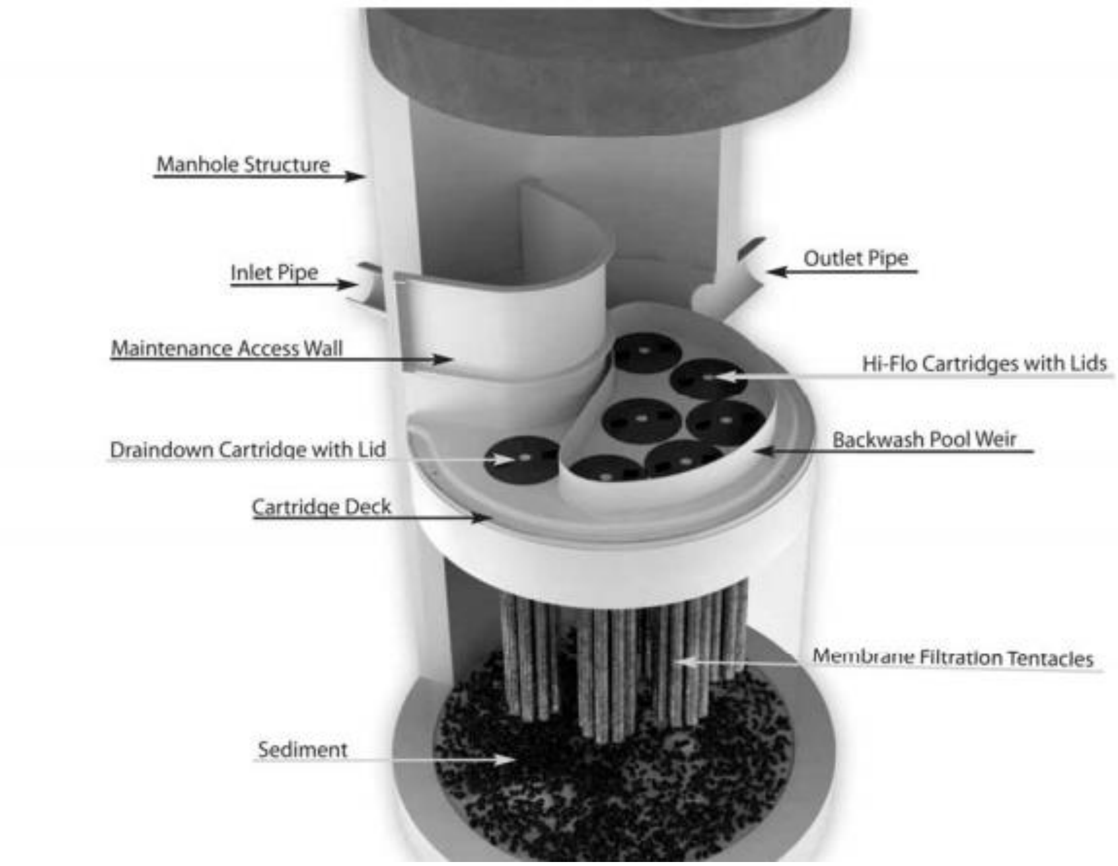


Figure 1. Cut-away graphic of a Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge

Figure 1 depicts a cut-away graphic of a typical 6-ft diameter Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge (JF6-6-1). Stormwater influent enters the system through the inlet pipe and builds a pond behind the maintenance access wall, with the pond elevation providing driving head. Flow is channeled downward into the lower chamber beneath the cartridge deck. A flexible separator skirt (not shown in the graphic) surrounds the filtration zone where the filtration tentacles of each cartridge are suspended, and the volume between the vessel wall and the outside surface of the separator skirt comprises a pretreatment channel. As flow spreads throughout the pretreatment channel, floatable pollutants accumulate at the surface of the pond behind the maintenance access wall and also beneath the cartridge deck in the pretreatment channel, while coarse sediments settle to the sump. Flow proceeds under the separator skirt and upward into the filtration zone, entering each filtration tentacle and depositing fine suspended sediment and associated particulate-bound pollutants on the outside surface of the membranes. Filtered water proceeds up the center tube of each tentacle, with the flow from each tentacle combining under the cartridge lid, and discharging to the top of the cartridge deck through the cartridge lid orifice. Filtered effluent from the hi-flo cartridges enters a pool enclosed by a 15-cm high weir, and if storm intensity and resultant driving head is sufficient, filtered water overflows the weir and proceeds across the cartridge deck to the outlet pipe. Filtered effluent discharging from the draindown cartridge(s) passes directly to the outlet pipe, and requires only a minimal amount of driving head (2.5 cm) to provide forward flow. As storm intensity subsides and driving head drops below 15 cm, filtered water within the backwash pool reverses direction and passes backward through the hi-flo cartridges, and thereby dislodges sediment from the membranes which subsequently settles to the sump below the filtration zone. During this passive backwashing process, water in the lower chamber is displaced only through the draindown cartridge(s). Additional self-cleaning processes include gravity, as well as vibrational pulses emitted when flow exits the orifice of each cartridge lid, and these combined processes significantly extend the cartridge service life and maintenance cleaning interval. Sediment removal from the sump by vacuum is required when sediment depths reach 30 cm, and cartridges are typically removed, externally rinsed, and recommissioned on an annual basis, or as site-specific maintenance conditions require. Filtration tentacle replacement is typically required every 3 – 5 years.

Performance conditions

The data and results published in this Technology Fact Sheet were obtained from a field monitoring program conducted on a Jellyfish® Filter JF4-2-1 (4-ft diameter manhole with 2 hi-flo cartridges and 1 draindown cartridge), in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009). Testing was completed by researchers led by Dr. John Sansalone at the University of Florida's Engineering School of Sustainable Infrastructure and Environment. The drainage area providing stormwater runoff to the test unit varied between 502 m^2 and 799 m^2 (5400 ft^2 to 8600 ft^2) depending on storm intensity and wind direction. The unit was monitored for a total of 25 TARP qualifying storm events (i.e. $\geq$ 2.5 mm of rainfall) contributing cumulative rainfall of 381 mm (15 in) over the 13-month period between May 28, 2010 and June 27, 2011. Only TARP-qualified storms were routed through the unit, and maintenance was not required during the testing period based on sediment accumulation less than the depth indicated for maintenance, and also based on hydraulic testing performed on the system after the conclusion of monitoring.

Table 1 shows the specified and achieved amended TARP criteria for storm selection and sampling.
Table 2 shows the observed ranges of operational conditions that occurred over the testing period.

Table 1. Specified and achieved amended TARP criteria for storm selection and sampling

Description	Criteria value	Achieved value
Total rainfall	$\geq$ 2.5 mm (0.1 in)	> 2.5 mm (0.1 in)
Minimum inter-event period	6 hrs	10 hrs
Minimum flow-weighted composite sample storm coverage	70% including as much of the first 20% of the storm	100%
Minimum influent/effluent samples	10, but a minimum of 5 subsamples for composite samples	Minimum of 8 subsamples for composite samples
Total sampled rainfall	Minimum 381 mm (15 in)	384 mm (15.01 in)
Number of storms	Minimum 20	25

Table 2. Observed operational conditions for events monitored over the study period

Operational condition	Observed range
Storm durations	26 – 691 min
Previous dry hours	10 - 910 hrs
Rainfall depth	3 – 50 mm
Initial rainfall to runoff lag time	1 – 34 min
Runoff volume	206 – 13,229 L
Peak rainfall intensity	5 – 137 mm/hr
Peak runoff flow rate	0.5 – 14.3 L/s
Event median flow rate	0.01 – 5.5 L/s

The 4-ft diameter test unit has sedimentation surface area of 1.17 m^2 (12.56 ft^2). Each of the three filter cartridges employed in the test unit uses filtration tentacles of 137 cm (54 in) length, with filter surface area of 35.4 m^2 (381 ft^2) per cartridge, and total filter surface area of 106.2 m^2 (1143 ft^2) for the three cartridges combined. The design treatment flow rate is 5 L/s (80 gal/min) for each of the two hi-flo cartridges and 2.5 L/s (40 gal/min) for the single draindown cartridge, for a total design treatment flow rate of 12.6 L/s (200 gal/min) at design driving head of 457 mm (18 in). This translates to a filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) for the draindown cartridge. The design flow rate for each cartridge is controlled by the sizing of the orifice in the cartridge lid. The distance from the bottom of the filtration tentacles to the sump is 61 cm (24 in).

Performance claims

The Jellyfish® Filter demonstrated the removal efficiencies indicated in **Table 3** for respective constituents during field monitoring of 25 TARP qualified storm events with cumulative rainfall of 381 mm, conducted in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009), and using the following design parameters:

- System hydraulic loading rate (system treatment flow rate per unit of sedimentation surface area) of 10.8 L/s/m^2 (15.9 gal/min/ft^2) or lower
- Filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) or lower for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) or lower for each draindown cartridge
- Distance from the bottom of the filtration tentacles to the sump of 61 cm (24 in) or greater
- Driving head of 457 mm (18 in) or greater

Table 3. Mean, median and 95% confidence interval (median) for removal efficiencies of selected stormwater constituents

Parameter	Mean	Median	Median - 95% Lower Limit	Median - 95% Upper Limit
TSS	84.7	85.6	82.8	89.8
SSC	97.5	98.3	97.1	98.7
Total phosphorus	48.8	49.1	43.3	60.1
Total nitrogen	37.9	39.3	31.2	54.6
Zinc	55.3	69	39	75
Copper	83.0	91.7	75.1	98.9
Oil and grease	60.1	60	42.7	100

N.B. As with any field test of stormwater treatment devices, removal efficiencies will vary based on pollutant influent concentrations and other site specific conditions.

Performance results

The frequency of rainfall depths monitored during the study is presented in **Figure 2**. The median and 90th percentile rainfall depths were 11 mm and 31.7 mm, respectively. These values represent the depth of rainfall that is not exceeded in 50 and 90 percent of the monitored rainfall events.

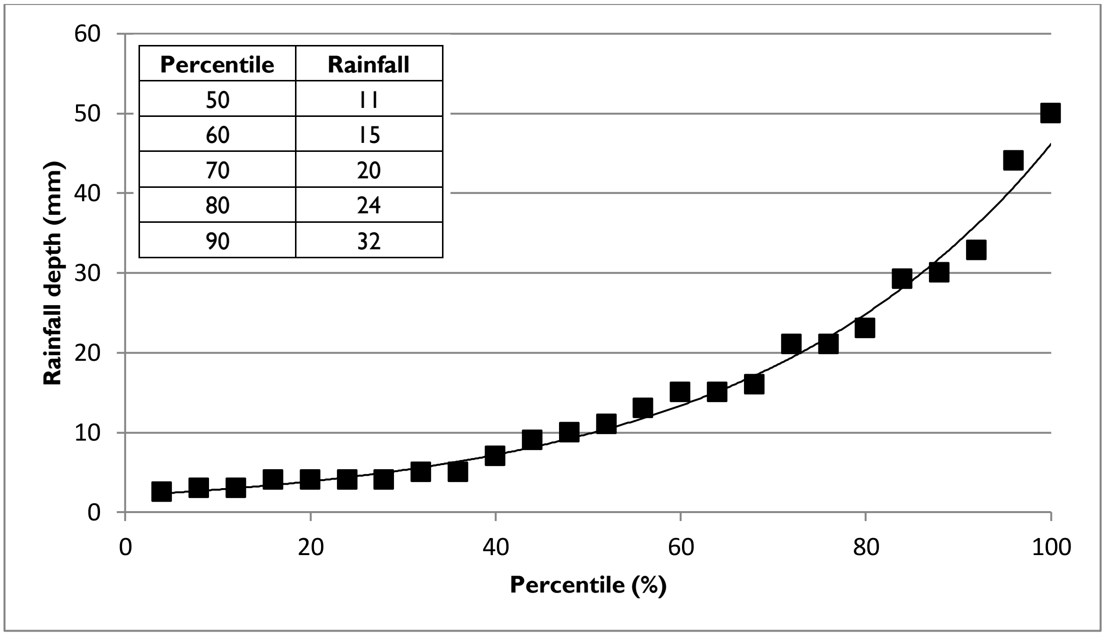


Figure 2. Rainfall depth frequency curve

Sediment removal performance was assessed by measuring the event mean concentration and mass of suspended sediment entering and leaving the unit during runoff events. This involved sampling the full cross-section of influent and effluent flows manually at 2 - 10 minute intervals for the full duration of each storm event and combining discrete samples into flow-weighted composites. Comparing the theoretical mass recovery from the sump calculated by the difference between the influent and effluent mass to the actual dry weight of the recovered sump mass showed an overall mass balance recovery of 94.5% over the study period.

The median d50 particle size (i.e. 50th percentile particle size) of the influent and effluent was 82 and 3 µm, respectively (**Figure 3**). The median influent particles sizes ranged between 22 and 263 µm, whereas median effluent particle sizes ranged between 1 and 11 µm.

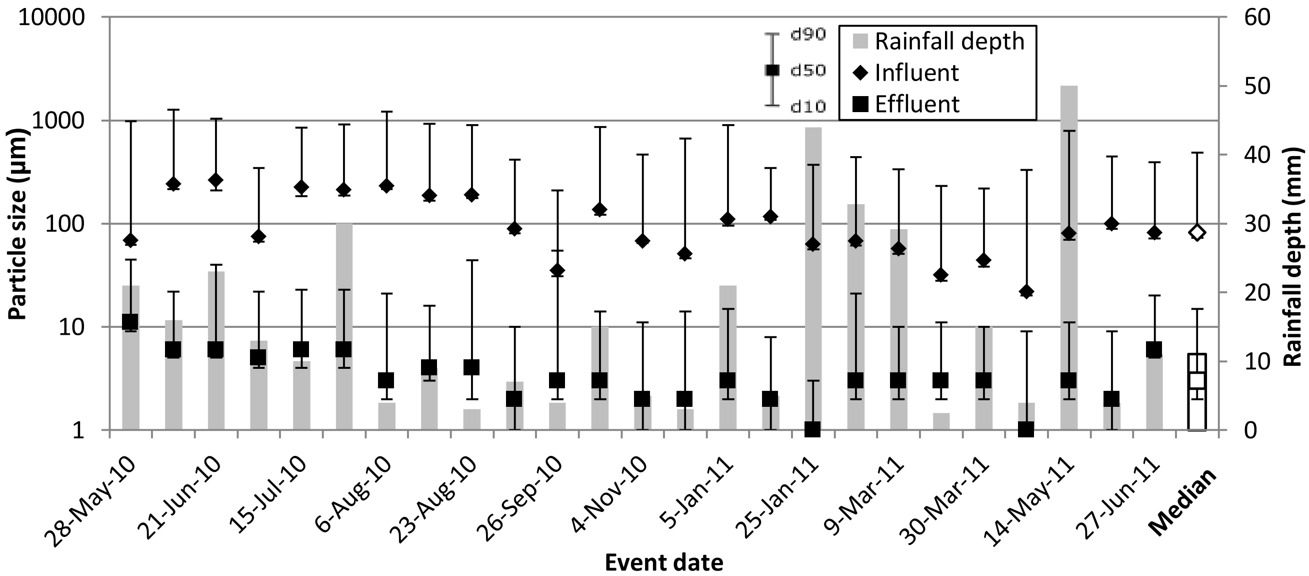


Figure 3. The rainfall depth and d10, d50, and d90 particle sizes of the influent and effluent composite samples for each monitored storm event over the 13-month testing period

Sampling of flows into and out of the Jellyfish Filter over the testing period showed statistically significant reductions ($p < 0.05$; Wilcoxon signed-rank test) in influent event mean concentrations for all selected stormwater constituents (**Table 4** and **Figure 4**). Effluent event mean Suspended Sediment Concentrations (SSC) were below 19 mg/L during all monitored events. Load-based removal rates were also calculated based on the sum of loads over the study period. These removal rages ranged from 46.3 for Total Nitrogen to 98.6 for SSC (**Table 4**).

Table 4. Summary statistics for influent and effluent event mean concentrations for selected constituents

Water Quality Variable	Sampling Location	Min	Max	Median	Range	Mean	SD	Load based removal efficiency (%)
TSS	Influent (mg/L)	16.30	261.00	79.30	244.70	86.26	51.37	87.2
	Effluent (mg/L)	3.20	21.70	11.80	18.50	10.99	4.79	
SSC	Influent (mg/L)	78.20	1401.70	444.50	1323.50	482.26	338.34	98.6
	Effluent (mg/L)	2.80	18.10	7.30	15.30	7.88	3.77	
TP	Influent (µg/L)	887.00	8793.00	3063.00	7906.00	3550.20	1914.50	64.2
	Effluent (µg/L)	472.00	4769.00	1480.00	4297.00	1688.08	1059.98	
TN	Influent (µg/L)	1170.00	10479.00	3110.00	9309.00	3519.32	2161.47	46.3
	Effluent (µg/L)	553.00	6579.00	1610.00	6026.00	2091.76	1613.61	
Zn	Influent (µg/L)	0.005	7600.00	1500.00	7600.00	1792.00	1852.91	76.1
	Effluent (µg/L)	0.005	2760.00	450.00	2760.00	561.64	594.70	
Cu	Influent (µg/L)	0.001	880.40	79.50	880.40	171.28	229.33	92.1
	Effluent (µg/L)	0.001	51.30	6.90	51.30	14.36	17.22	
Oil and Grease	Influent (mg/L)	0.20	4.06	0.93	3.86	1.07	0.82	46.4
	Effluent (mg/L)	0.00	2.32	0.35	2.32	0.50	0.60	

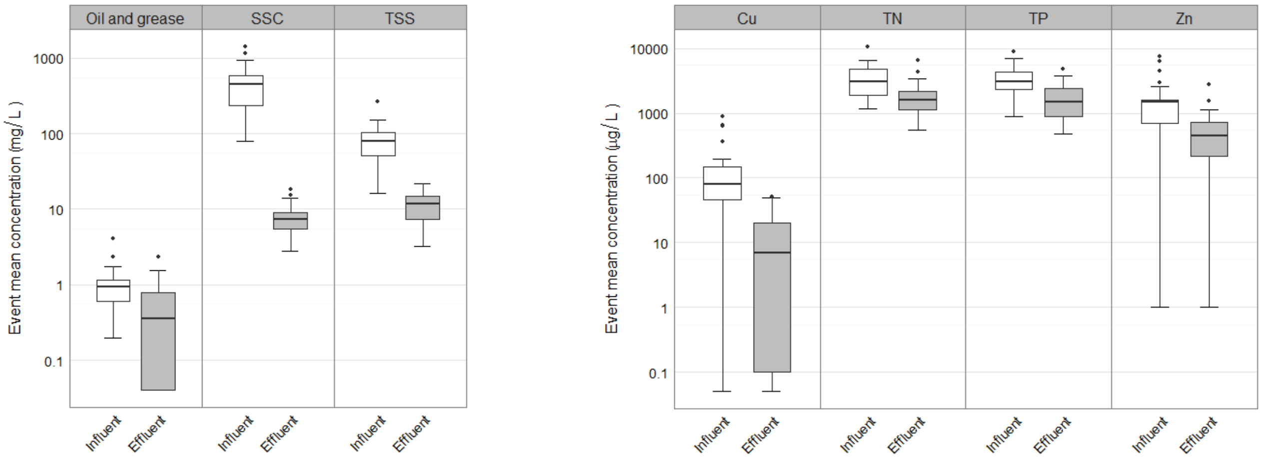


Figure 4. Boxplots showing the distribution of influent and effluent event mean concentrations (EMC) for selected stormwater constituents over the study period

Verification

The verification was completed by the Verification Expert, Toronto and Region Conservation Authority, contracted by GLOBE Performance Solutions, using the International Standard ***ISO 14034:2016 Environmental management -- Environmental technology verification (ETV)***. Data and information provided by Imbrium Systems to support the performance claim included the performance monitoring report prepared by University of Florida, Engineering School of Sustainable Infrastructure and Environment, and dated November 2011. This report is based on testing completed in accordance with the Technology Acceptance Reciprocity Partnership (TARP) Tier II Protocol (2003) and New Jersey Tier II Stormwater Test Requirements--Amendments to TARP Tier II Protocol (NJDEP, 2009).

What is ISO14034:2016 Environmental management – Environmental technology verification (ETV)?

ISO 14034:2016 specifies principles, procedures and requirements for environmental technology verification (ETV), and was developed and published by the *International Organization for Standardization (ISO)*. The objective of ETV is to provide credible, reliable and independent verification of the performance of environmental technologies. An environmental technology is a technology that either results in an environmental added value or measures parameters that indicate an environmental impact. Such technologies have an increasingly important role in addressing environmental challenges and achieving sustainable development.

For more information on the Jellyfish® Filter please contact:

Imbrium Systems, Inc.
407 Fairview Drive
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Tel: 416-960-9900
info@imbriumsystems.com

For more information on ISO 14034:2016 / ETV please contact:

GLOBE Performance Solutions
World Trade Centre
404 – 999 Canada Place
Vancouver, BC
V6C 3E2 Canada
Tel: 604-695-5018 / Toll Free: 1-855-695-5018
etv@globeperformance.com

Limitation of verification
GLOBE Performance Solutions and the Verification Expert provide the verification services solely on the basis of the information supplied by the applicant or vendor and assume no liability thereafter. The responsibility for the information supplied remains solely with the applicant or vendor and the liability for the purchase, installation, and operation (whether consequential or otherwise) is not transferred to any other party as a result of the verification.

ON-LINE
Jellyfish Filter Sizing Report

Project Information

Date	Saturday, January 18, 2020
Project Name	13247 and 13233 Nunnville Rd.
Project Number	0649-5291
Location	Caledon

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.

Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF8-9-2-L1 is recommended to meet the water quality objective by treating a flow of 50.5 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This Jellyfish Model has a Peak Flow Capacity of 192.1 L/s. This model has a sediment capacity of 569 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Peak Capacity (L/s)	Bypass MAW	Sediment Capacity (kg)
JF8-9-2-L1	9	2	2.4	50.5	192.1	Yes	569

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance inverval is designed to be a minimum of 12 months, but this will vary significantly with on-line desings based on storm event conditions, site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.

Please see ***www.ImbriumSystems.com*** *for more information.*

Thank you for the opportunity to present this information to you and your client.

Jellyfish® Filter

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

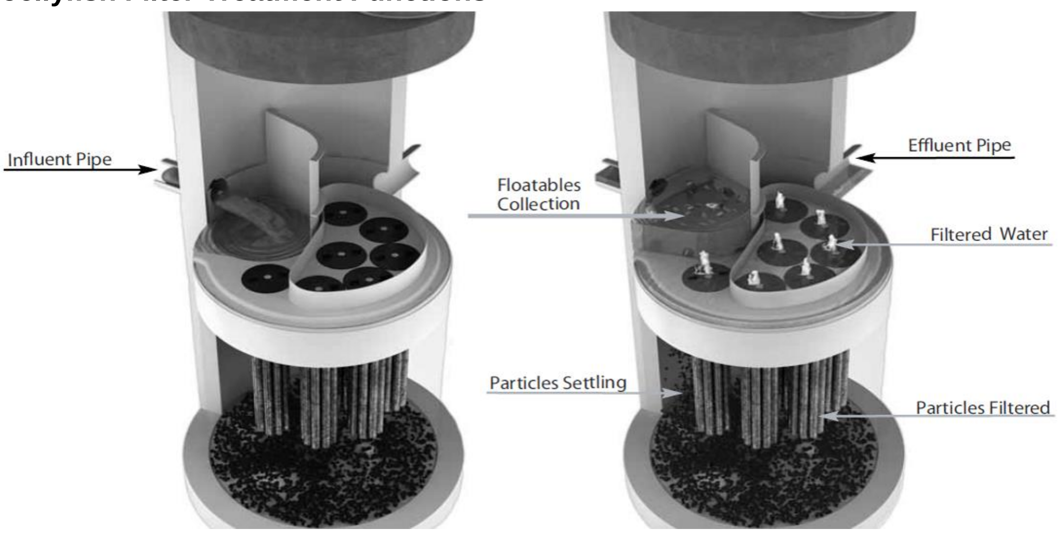
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 59% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

The Jellyfish filter has been field tested on an urban site with 25 TARP qualifying rain events and field monitored according to the TARP field test protocol, demonstrating:

- A median TSS removal efficiency of 89%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 59%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Saturday, January 18, 2020
Project Name:	13247 and 13233 Nunnville Rd.
Project Number:	0649-5291
Location:	Caledon

Designer Information

Company:	C.F. Crozier & Associates Inc.
Contact:	Heaven Lin
Phone #:	

Notes

Rainfall

Name:	TORONTO CENTRAL
State:	ON
ID:	100
Record:	1982 to 1999
Co-ords:	45°30'N, 90°30'W

Drainage Area

Total Area:	1.71 ha
Imperviousness:	53%

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	**25 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 5435 m³, with a suspended sediment concentration of 100 mg/L.	**544 kg***
Peak Flow	The flow which must pass through the unit, includes	**93 L/s**

* *Indicates that sediment loading is the limiting parameter in the sizing of this Jellyfish system.*

Recommendation

The Jellyfish Filter model JF8-9-2-L1 is recommended to meet the water quality objective by treating a flow of 50.5 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This Jellyfish Model has a Peak Flow Capacity of 192.1 L/s. This model has a sediment capacity of 569 kg, which meets or exceeds the estimated average annual sediment load.

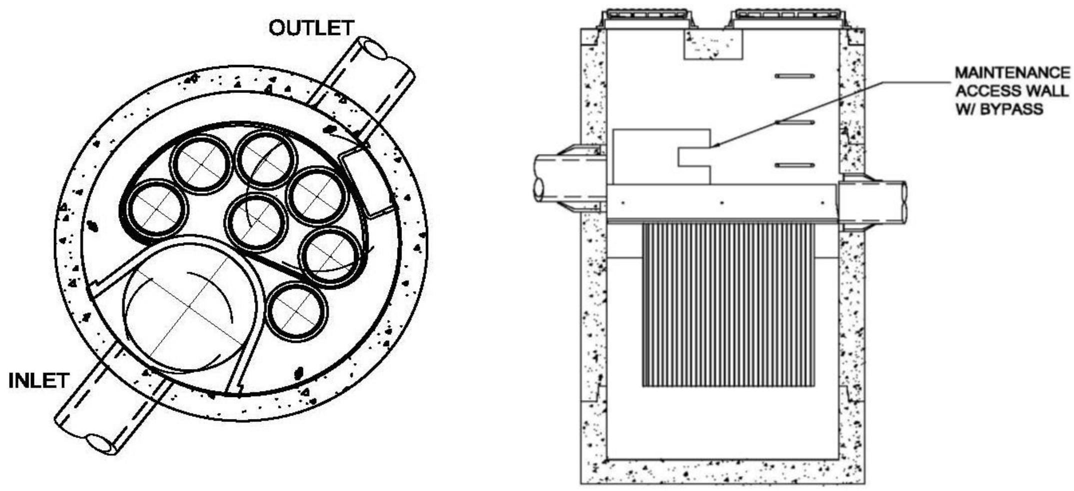
Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Bypass MAW	Addt'l Sump (ft)	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1-L0	1	1	No	0	1.2	2313	0.34	379	7.6	85
JF4-1-1-L1	1	1	Yes	1	1.2	2661	0.34	379	7.6	85
JF4-2-1-L0	2	1	No	0	1.2	2313	0.34	379	12.6	142
JF4-2-1-L1	2	1	Yes	1	1.2	2661	0.34	379	12.6	142
JF6-3-1-L0	3	1	No	0	1.8	5205	0.79	848	17.7	199
JF6-3-1-L1	3	1	Yes	1	1.8	6003	0.79	848	17.7	199
JF6-4-1-L0	4	1	No	0	1.8	5205	0.79	848	22.7	256
JF6-4-1-L1	4	1	Yes	1	1.8	6003	0.79	848	22.7	256
JF6-5-1-L0	5	1	No	0	1.8	5205	0.79	848	27.8	313
JF6-5-1-L1	5	1	Yes	1	1.8	6003	0.79	848	27.8	313
JF6-6-1-L0	6	1	No	0	1.8	5205	0.79	848	28.6	370
JF6-6-1-L1	6	1	Yes	1	1.8	6003	0.79	848	28.6	370
JF8-6-2-L0	6	2	No	0	2.4	9252	1.42	1469	35.3	398
JF8-6-2-L1	6	2	Yes	1	2.4	10675	1.42	1469	35.3	398
JF8-7-2-L0	7	2	No	0	2.4	9252	1.42	1469	40.4	455
JF8-7-2-L1	7	2	Yes	1	2.4	10675	1.42	1469	40.4	455
JF8-8-2-L0	8	2	No	0	2.4	9252	1.42	1469	45.4	512
JF8-8-2-L1	8	2	Yes	1	2.4	10675	1.42	1469	45.4	512
JF8-9-2-L0	9	2	No	0	2.4	9252	1.42	1469	50.5	569
JF8-9-2-L1	**9**	**2**	**Yes**	**1**	**2.4**	**10675**	**1.42**	**1469**	**50.5**	**569**
JF8-10-2-L0	10	2	No	0	2.4	9252	1.42	1469	50.5	626
JF8-10-2-L1	10	2	Yes	1	2.4	10675	1.42	1469	50.5	626
JF10-11-3-L0	11	3	No	0	3.0	14456	2.21	2302	63.1	711
JF10-11-3-L1	11	3	Yes	1	3.0	16678	2.21	2302	63.1	711
JF10-12-3-L0	12	3	No	0	3.0	14456	2.21	2302	68.2	768
JF10-12-3-L1	12	3	Yes	1	3.0	16678	2.21	2302	68.2	768
JF10-12-4-L0	12	4	No	0	3.0	14456	2.21	2302	70.7	796
JF10-12-4-L1	12	4	Yes	1	3.0	16678	2.21	2302	70.7	796
JF10-13-4-L0	13	4	No	0	3.0	14456	2.21	2302	75.7	853
JF10-13-4-L1	13	4	Yes	1	3.0	16678	2.21	2302	75.7	853
JF10-14-4-L0	14	4	No	0	3.0	14456	2.21	2302	78.9	910

JF10-14-4-L1	14	4	Yes	1	3.0	16678	2.21	2302	78.9	910
JF10-15-4-L0	15	4	No	0	3.0	14456	2.21	2302	78.9	967
JF10-15-4-L1	15	4	Yes	1	3.0	16678	2.21	2302	78.9	967
JF10-16-4-L0	16	4	No	0	3.0	14456	2.21	2302	78.9	1024
JF10-16-4-L1	16	4	Yes	1	3.0	16678	2.21	2302	78.9	1024
JF10-17-4-L0	17	4	No	0	3.0	14456	2.21	2302	78.9	1081
JF10-17-4-L1	17	4	Yes	1	3.0	16678	2.21	2302	78.9	1081
JF10-18-4-L0	18	4	No	0	3.0	14456	2.21	2302	78.9	1138
JF10-18-4-L1	18	4	Yes	1	3.0	16678	2.21	2302	78.9	1138
JF10-19-4-L0	19	4	No	0	3.0	14456	2.21	2302	78.9	1195
JF10-19-4-L1	19	4	Yes	1	3.0	16678	2.21	2302	78.9	1195
JF12-20-5-L0	20	5	No	0	3.6	20820	3.2	2771	113.6	1280
JF12-20-5-L1	20	5	Yes	1	3.6	24012	3.2	2771	113.6	1280
JF12-21-5-L0	21	5	No	0	3.6	20820	3.2	2771	113.7	1337
JF12-21-5-L1	21	5	Yes	1	3.6	24012	3.2	2771	113.7	1337
JF12-22-5-L0	22	5	No	0	3.6	20820	3.2	2771	113.7	1394
JF12-22-5-L1	22	5	Yes	1	3.6	24012	3.2	2771	113.7	1394
JF12-23-5-L0	23	5	No	0	3.6	20820	3.2	2771	113.7	1451
JF12-23-5-L1	23	5	Yes	1	3.6	24012	3.2	2771	113.7	1451
JF12-24-5-L0	24	5	No	0	3.6	20820	3.2	2771	113.7	1508
JF12-24-5-L1	24	5	Yes	1	3.6	24012	3.2	2771	113.7	1508
JF12-25-5-L0	25	5	No	0	3.6	20820	3.2	2771	113.7	1565
JF12-25-5-L1	25	5	Yes	1	3.6	24012	3.2	2771	113.7	1565
JF12-26-5-L0	26	5	No	0	3.6	20820	3.2	2771	113.7	1622
JF12-26-5-L1	26	5	Yes	1	3.6	24012	3.2	2771	113.7	1622
JF12-27-5-L0	27	5	No	0	3.6	20820	3.2	2771	113.7	1679
JF12-27-5-L1	27	5	Yes	1	3.6	24012	3.2	2771	113.7	1679

Jellyfish® Filter

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter. However, note that inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Online Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance. Alternative driving head values are only possible with off-line configuration.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation.
- The Jellyfish Filter can not accommodate multiple inlet pipes in an on-line configuration.
- Typical systems conform to the following pipe orientations:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Maximum Inlet Pipe Diameter (mm)
1.2	62º	150	300
1.8	59º	200	300
2.4	**52º**	**250**	**375**
3.0	48º	300	450
3.6	40º	300	450

- Outlet pipes to be one size larger than inlet pipes on new projects.
- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head caclulations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged condtions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish vault units can be designed.

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft^2 (0.142 lps/m^2).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Conseal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 <u>INSTALLATION</u>

4.1.1 <u>PRECAST DEVICE CONSTRUCTION SEQUENCE</u>

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL
In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after is has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

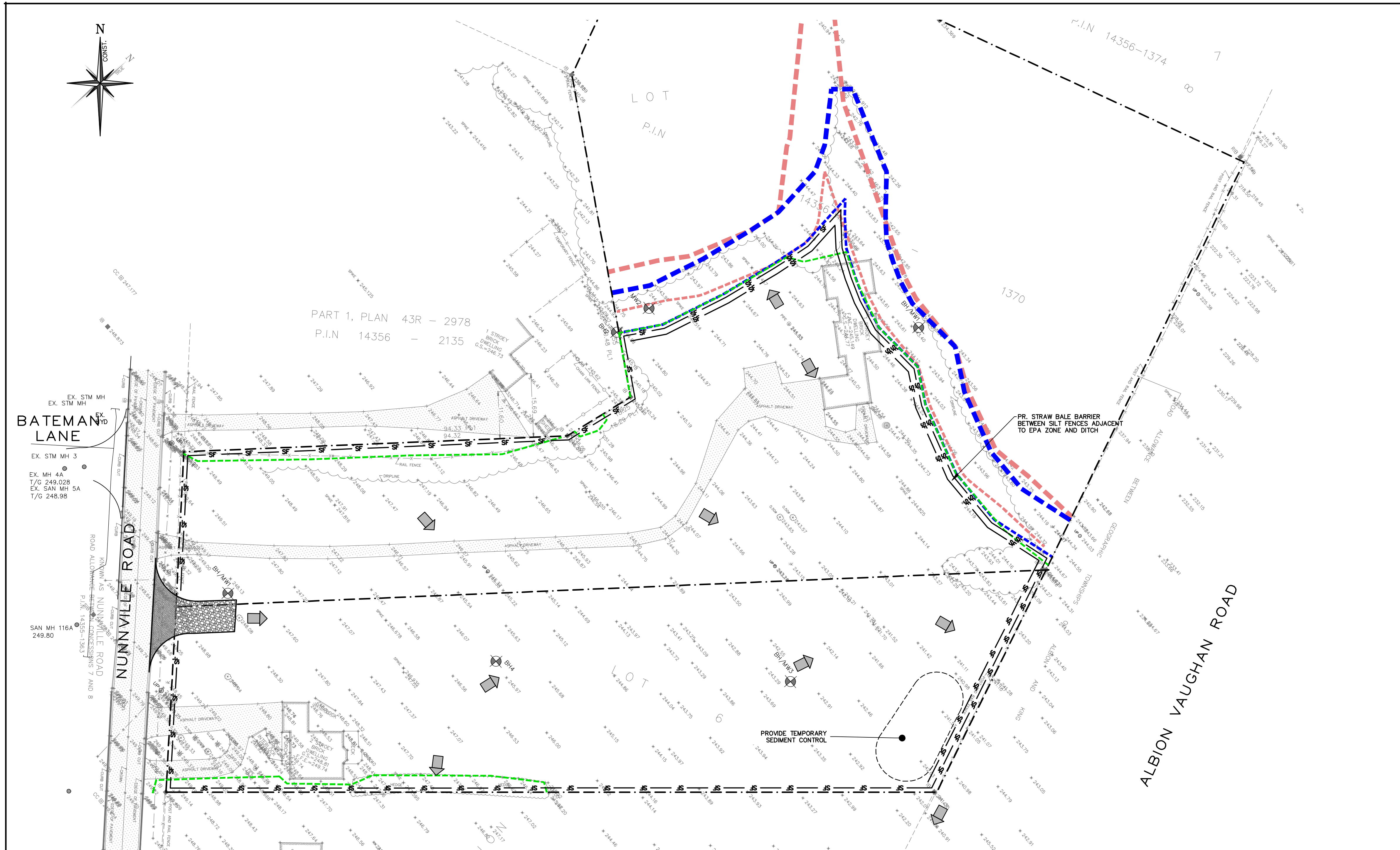
5.2.1 The manufacturer shall provide an Owner's Manual upon request.

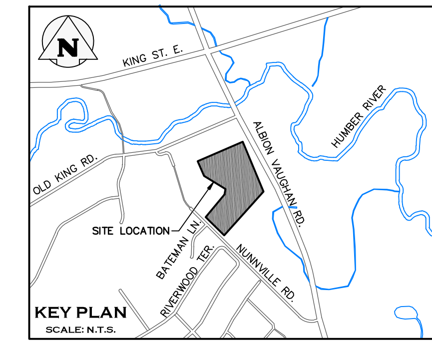
5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

DRAWINGS AND FIGURES





LEGEND

- PROPERTY LINE
- EXISTING CONTOUR (0.5m)
- EXISTING CONTOUR (1.0m)
- EXISTING DITCH
- EXISTING HYDRO POLE
- EXISTING FENCE
- EXISTING GRADE
- EXISTING OVERLAND FLOW DIRECTION
- MUD-MAT; SEE DETAIL
- SILT FENCE PER TOWN STANDARD
- BOREHOLE AND MONITORING WELL LOCATION (SOIL ENGINEERS JULY 26, 2019)
- TRCA STAKED DRIPLINE
- 10m TRCA SETBACK FROM STAKED DRIPLINE
- STABLE TOP OF SLOPE
- 10m SETBACK FROM STABLE TOP OF BANK
- TREE PRESERVATION LIMITS

EXISTING UTILITIES AND SERVICES

CONTRACTOR SHALL NOTE THAT THE CONSTRUCTION ZONE HAS NUMEROUS EXISTING UNDERGROUND UTILITIES AND SERVICES, SOME OF WHICH ARE TO BE ABANDONED OR REMOVED, AND OTHERS WHICH ARE TO BE PROTECTED AND MAINTAINED IN SERVICE.

PRIOR TO CONSTRUCTION, THE CONTRACTOR SHALL RETAIN THE SERVICES OF A COMPANY, WHICH SPECIALIZES IN SUBSURFACE UTILITY ENGINEERING FOR THE PURPOSES OF LOCATING, MARKING AND SURVEYING ALL UNDERGROUND UTILITIES AND SERVICES. ALL CURRENT METHODS SHALL BE USED FOR THESE LOCATIONS INCLUDING ELECTRONIC METHODS, VACUUM EXCAVATIONS, SURVEYING MANHOLES AND CHAMBERS ETC.

THE UTILITIES AND SERVICES SHALL BE SURVEYED AND TIED INTO THE PROJECT COORDINATE SYSTEM. A COPY OF THE SURVEY SHALL BE PROVIDED TO THE ENGINEER FOR RECORD PURPOSES.

ANY CONFLICT WITH THE PROPOSED WORKS SHALL BE BROUGHT TO THE ATTENTION OF THE ENGINEER.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR ALL LOCATIONS FOR PROTECTION AND TEMPORARY RELOCATION OF UNDERGROUND UTILITIES AND SERVICES AS REQUIRED FOR THE COMPLETE INSTALLATION OF THE PROPOSED WORKS.

10 0m 10 20 30 40

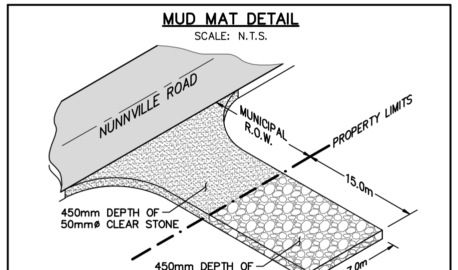
SCALE: 1:500

EROSION & SEDIMENT CONTROL NOTES:

1. EROSION & SEDIMENT CONTROL MEASURES MUST BE INSTALLED PRIOR TO THE COMMENCEMENT OF SITE WORKS.
2. EROSION & SEDIMENT CONTROLS MUST BE INSPECTED ON A REGULAR BASIS AND AFTER EVERY RAIN FALL EVENT, AND MUST BE MAINTAINED AND REPAIRED IN A TIMELY MANNER TO PREVENT SEDIMENT FROM LEAVING THE SITE.
3. EXISTING AND PROPOSED CATCHBASINS ARE TO BE PROTECTED WITH FILTER CLOTH AND 150mm OF 50mm STONE COVER DURING CONSTRUCTION.
4. IT IS REQUIRED TO STABILIZE ALL AREAS THAT WILL REMAIN DISTURBED FOR MORE THAN 30 DAYS.
5. MUD MAT, SILT FENCE, AND CATCHBASIN PROTECTION ARE NOT TO BE REMOVED UNTIL COMPLETION OF CONSTRUCTION.

MUD MAT DETAIL

SCALE: N.T.S.



NOTE:
GEOTEXTILE (TERRAFIX 270R OR APPROVED EQUAL) TO BE PLACED AS SEPARATION BARRIER BETWEEN EXISTING GROUND AND CLEAR STONE.

NOT FOR CONSTRUCTION

FOR REVIEW

No.	ISSUE / REVISION	YYYY/MMM/DD
1	ISSUED FOR 2nd SUBMISSION	2020/JAN/24
0	ISSUED FOR 1st SUBMISSION	2019/AUG/02

ELEVATION NOTE:

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO FIRST ORDER BENCHMARK NO. 00819758057 HAVING AN ORTHOMEERIC ELEVATION OF 251.929 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, 1978 ADJUSTMENT (CGVD-1928:1978).

SURVEY NOTES:

THE FIELD OBSERVATION REPRESENTED ON THIS PLAN WERE COMPLETED BY R-PE SURVEYING LTD ON THE MAY 9, 2019.

SITE PLAN NOTES:

DESIGN ELEMENTS SHOWN BASED ON DRAFT PLAN OF SUBDIVISION PREPARED BY WSP PROJ No.: 19M-00294-00 (2020/JAN/17)

DRAWING NOTES:

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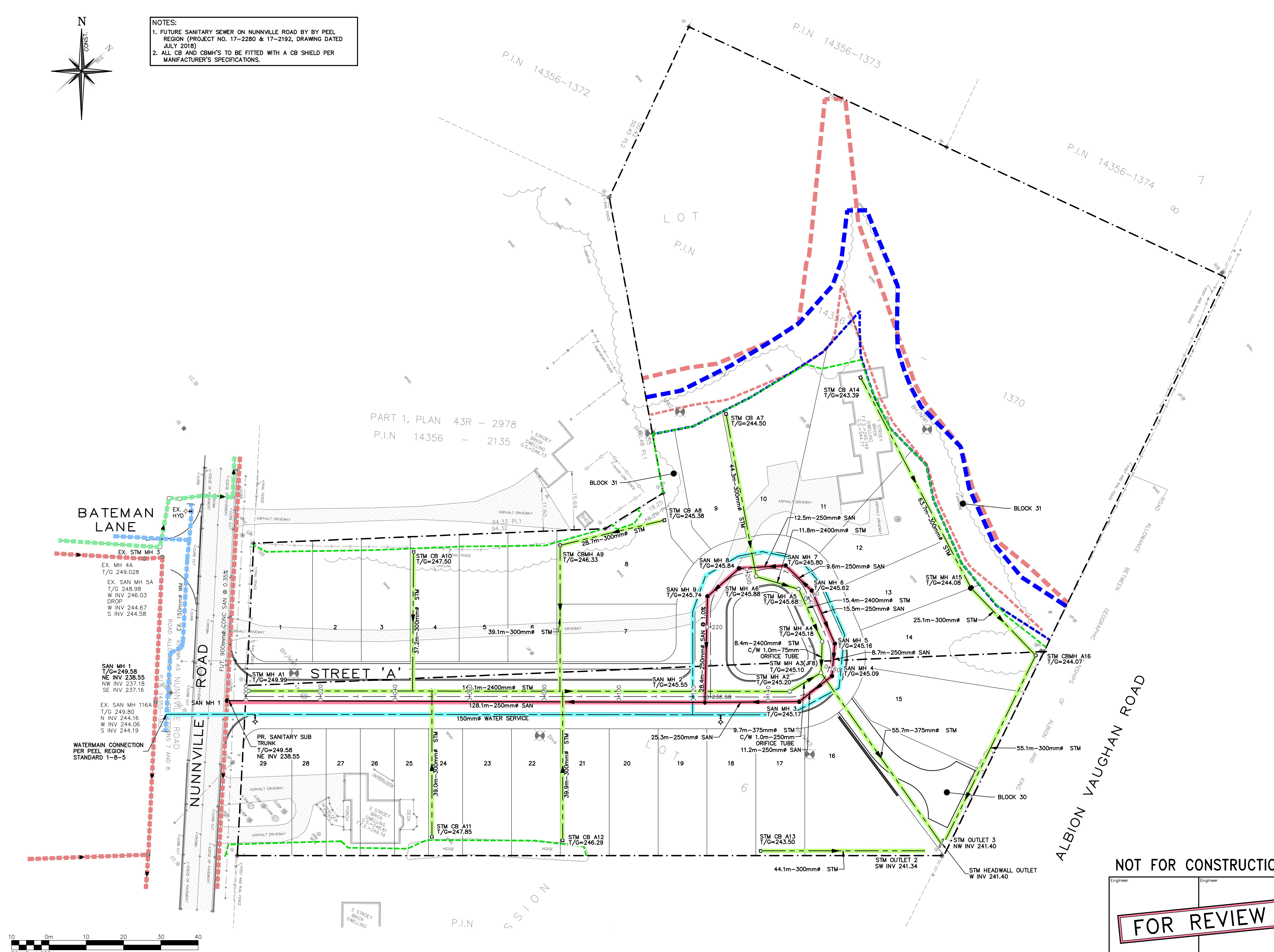
ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

Project
13247 & 13233 NUNNVILLE ROAD
BOLTON
TOWN OF CALEDON

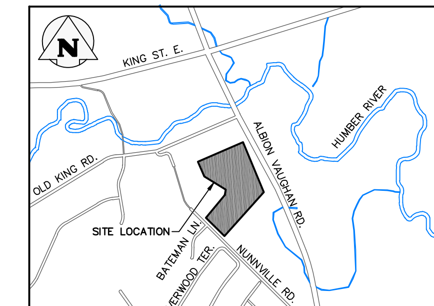
Drawing
EROSION & SEDIMENT CONTROL PLAN

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.	
P.S.	P.S./N.A.S.	649-5291	
Check	Check	Scale	Dwg.
S.C.S.	N.A.S./K.J.F	1:500	C101



NOTES:
1. FUTURE SANITARY SEWER ON NUNNVILLE ROAD BY BY PEEL REGION (PROJECT NO. 17-2280 & 17-2192, DRAWING DATED JULY 2018)
2. ALL CB AND CBMH'S TO BE FITTED WITH A CB SHIELD PER MANIFACTURER'S SPECIFICATIONS.



KEY PLAN
SCALE: N.T.S.

LEGEND

- PROPERTY LINE
- EXISTING WATERMAIN & GATE VALVE
- EXISTING STORM SEWER & MANHOLE
- EXISTING SINGLE / DOUBLE CATCHBASIN
- EXISTING SANITARY SEWER & MANHOLE
- PROPOSED WATERMAIN & GATE VALVE
- PROPOSED WATER SERVICE LATERAL (XXmmø)
- PROPOSED FIRE HYDRANT & GATE VALVE
- PROPOSED SIAMESE CONNECTION
- PROPOSED WATER METER
- PROPOSED BACKFLOW PREVENTOR
- PROPOSED STORM SEWER & MANHOLE
- PROPOSED SINGLE / DOUBLE CATCHBASIN
- PROPOSED SANITARY SEWER & MANHOLE
- TRCA STAKED DRIPLINE
- 10m TRCA SETBACK FROM STAKED DRIPLINE
- STAKED STABLE TOP OF SLOPE
- 10m SETBACK FROM STABLE TOP OF SLOPE
- BOREHOLE AND MONITORING WELL LOCATION (SOIL ENGINEERS JULY 26, 2019)
- TREE PRESERVATION LIMITS

No.	ISSUE / REVISION	YYYY/MMM/DD
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Project
13247 & 13233 NUNNVILLE ROAD
BOLTON
TOWN OF CALEDON

Drawing
PRELIMINARY SITE SERVICING PLAN

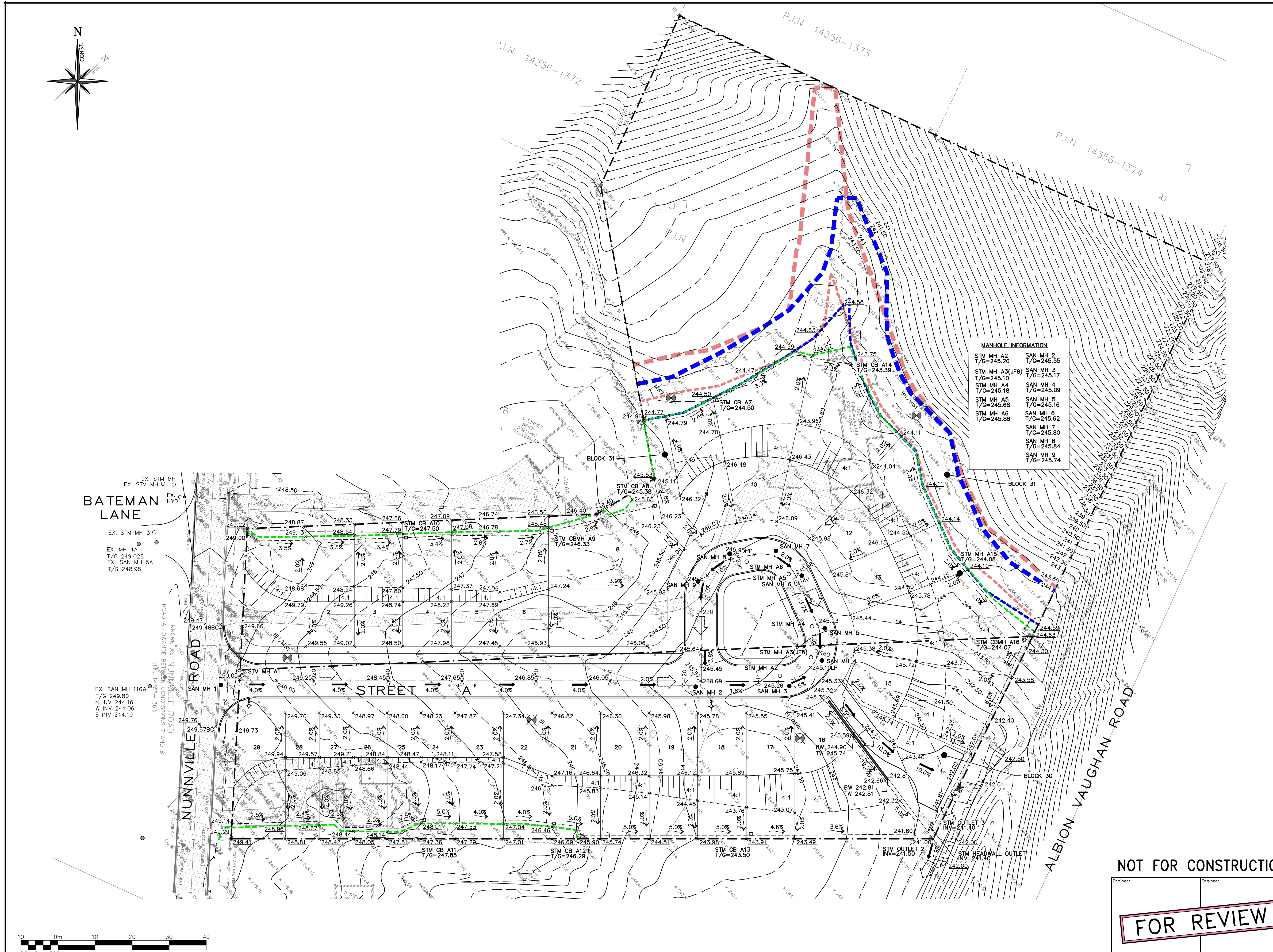
NOT FOR CONSTRUCTION

Engineer | Engineer

FOR REVIEW

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

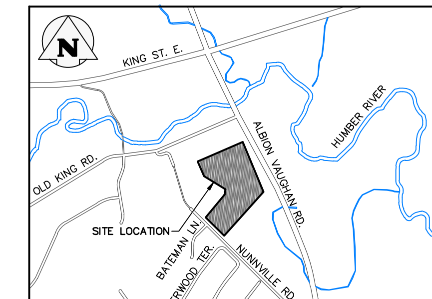
Drawn P.S.	Design P.S./N.A.S.	Project No.	649-5291
Check S.C.S.	Check N.A.S./K.J.F	Scale 1:500	Dwg. C102



MANHOLE INFORMATION

STM MH A2 T/G=245.20	SAN MH 2 T/G=245.55
STM MH A3(JF8) T/G=245.10	SAN MH 3 T/G=245.17
STM MH A4 T/G=245.18	SAN MH 4 T/G=245.09
STM MH A5 T/G=245.68	SAN MH 5 T/G=245.16
STM MH A6 T/G=245.88	SAN MH 6 T/G=245.62
	SAN MH 7 T/G=245.80
	SAN MH 8 T/G=245.84
	SAN MH 9 T/G=245.74

NOT FOR CONSTRUCTION



KEY PLAN
SCALE: N.T.S.

LEGEND

- PROPERTY LINE
- EXISTING CONTOUR (0.5m)
- EXISTING CONTOUR (1.0m)
- EXISTING DITCH
- EXISTING FENCE
- 215.00 EXISTING GRADE
- 215.00 PROPOSED GRADE
- 215.00 PROPOSED GRADE (TO MATCH EXISTING)
- 2.0% PROPOSED MINOR FLOW DIRECTION
- 2.0% PROPOSED GRASSED SWALE
- PROPOSED SLOPE (3:1 MAX.)
- TRCA STAKED DRIPLINE
- 10m TRCA SETBACK FROM STAKED DRIPLINE
- STABLE TOP OF SLOPE
- 10m SETBACK FROM STABLE TOP OF BANK
- PROPOSED MAJOR OVERLAND FLOW DIRECTION
- BOREHOLE AND MONITORING WELL LOCATION (SOIL ENGINEERS JULY 26, 2019)
- TREE PRESERVATION LIMITS

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Project

13247 & 13233 NUNNVILLE ROAD
BOLTON
TOWN OF CALEDON

Drawing

PRELIMINRAY SITE GRADING PLAN

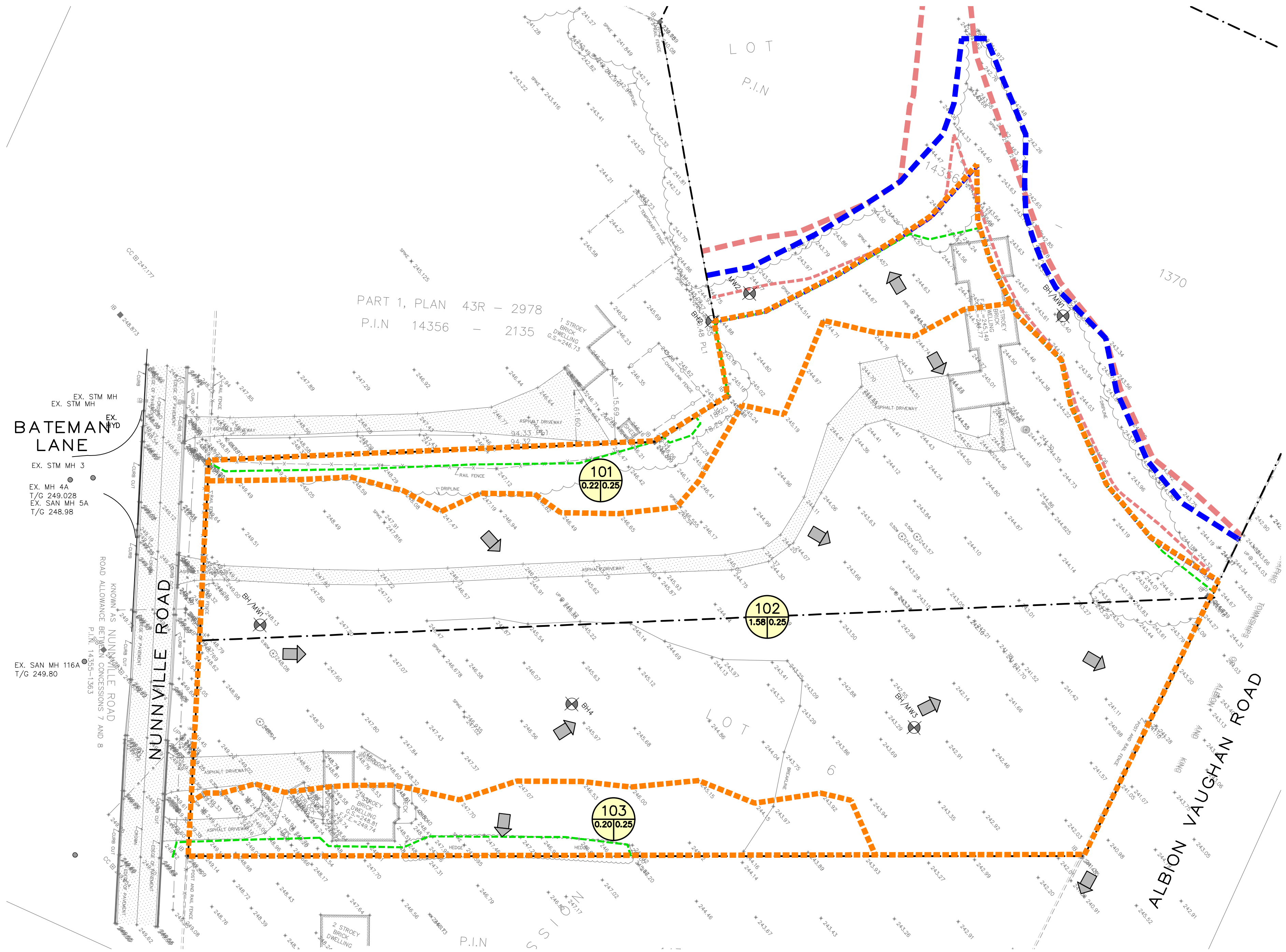
Engineer | Engineer

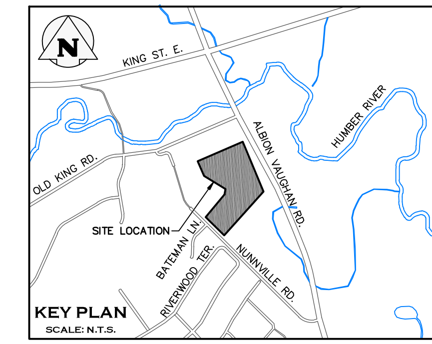
FOR REVIEW

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 P.S.	Design P.S./N.A.S.	Project No. 649-5291	
Check S.C.S.	Check N.A.S./K.J.F	Scale 1:500	Dwg. C103





LEGEND

- PROPERTY LINE
- EXISTING CONTOUR (0.5m)
- EXISTING CONTOUR (1.0m)
- EXISTING DITCH
- x 215.00 EXISTING GRADE
- EXISTING OVERLAND FLOW DIRECTION
- PRE DEVELOPMENT DRAINAGE CATCHMENT
- ID / A | RC — CATCHMENT I.D.; AREA (ha) | RUNOFF COEFFICIENT
- TRCA STAKED DRIPLINE
- 10m TRCA SETBACK FROM STAKED DRIPLINE
- STABLE TOP OF BANK
- 10m SETBACK FROM STABLE TOP OF BANK
- BOREHOLE AND MONITORING WELL LOCATION (SOIL ENGINEERS JULY 26, 2019)
- TREE PRESERVATION LIMITS

No.	ISSUE / REVISION	YYYY/MMM/DD
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SITE PLAN NOTES:

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Project

13247 & 13233 NUNNVILLE ROAD
BOLTON
TOWN OF CALEDON

Drawing

PRE–DEVELOPMENT DRAINAGE PALN

NOT FOR CONSTRUCTION

Engineer | Engineer

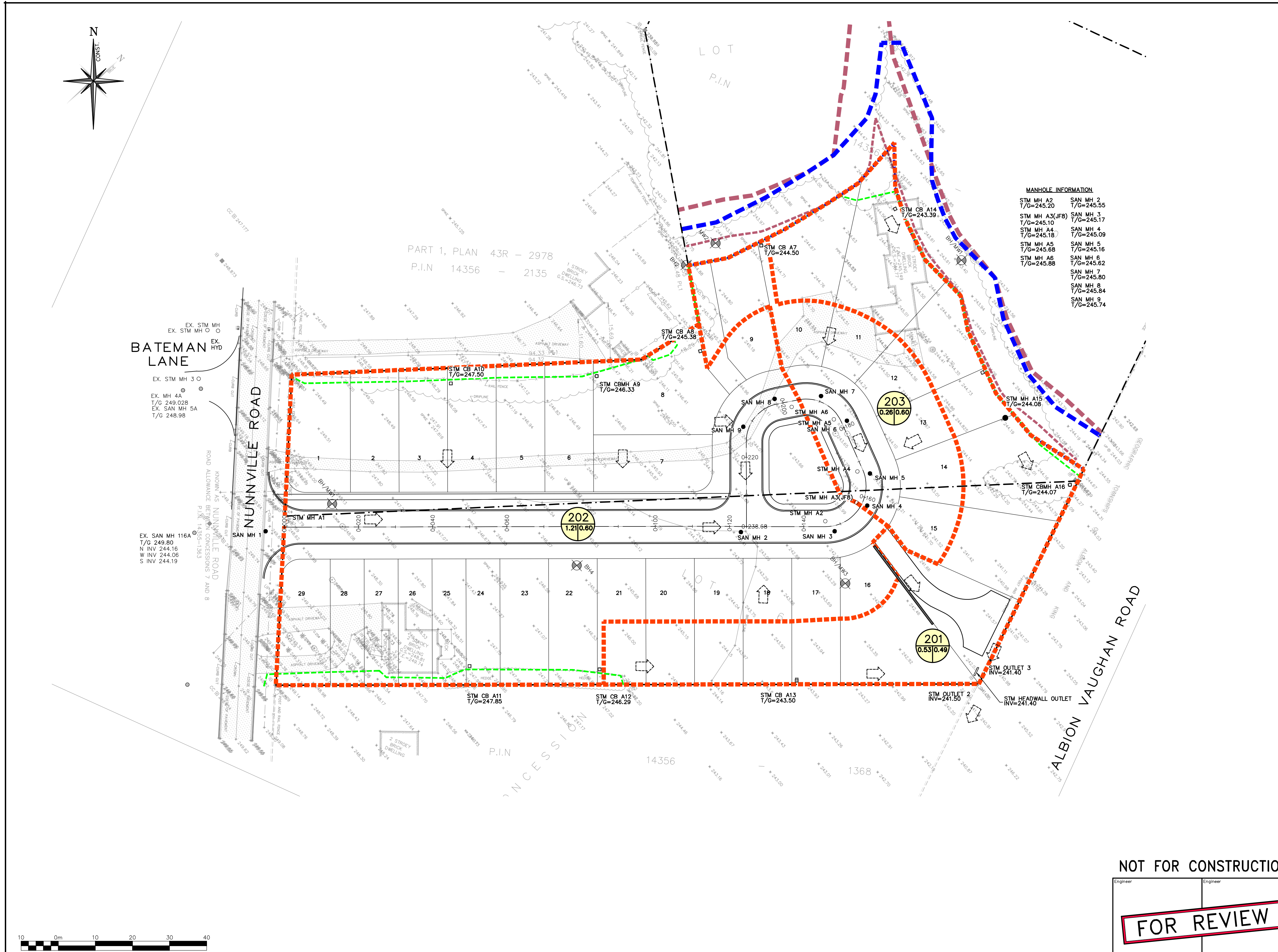
FOR REVIEW

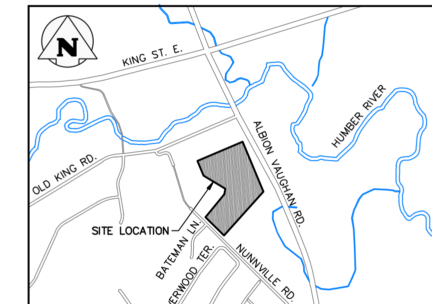
CROZIER
CONSULTING ENGINEERS

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.	
P.S.	P.S./N.A.S.	649–5291	
Check S.C.S.	Check N.A.S./K.J.F	Scale 1:500	Dwg. FIG 1

10 0m 10 20 30 40
SCALE: 1:500





KEY PLAN

SCALE: N.T.S.

LEGEND

- PROPERTY LINE
- EXISTING CONTOUR (0.5m)
- EXISTING CONTOUR (1.0m)
- EXISTING DITCH
- × 215.00 EXISTING GRADE
- PROPOSED MAJOR FLOW DIRECTION
- PROPOSED DRAINAGE CATCHMENT
- ID / A | RC — CATCHMENT I.D. / AREA (ha) | RUNOFF COEFFICIENT
- TRCA STAKED DRIPLINE
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- TREE PRESERVATION LIMITS

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Project

13247 & 13233 NUNNVILLE ROAD
BOLTON
TOWN OF CALEDON

Drawing

POST-DEVELOPMENT DRAINAGE PLAN

NOT FOR CONSTRUCTION

Engineer | Engineer

FOR REVIEW

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.	
P.S.	P.S./N.A.S.	649-5291	
Check S.C.S.	Check N.A.S./K.J.F	Scale 1:500	Dwg. FIG 2