# PROPOSED

# THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE

**PART OF LOT B**

**CONCESSION 11**

**GEOGRAPHIC TOWNSHIP OF CUMBERLAND**

**1670 TENTH LINE ROAD**

**CITY OF OTTAWA**

## STORM DRAINAGE REPORT

**REPORT R-825-8**

**T.L. MAK ENGINEERING CONSULTANTS LTD.**

**JULY 2025**

**REFERENCE FILE NUMBER 825-8**

## Introduction

The proposed three (3) storey residential apartment building site is located on the west side of Tenth Line Road and situated south of Amiens Street and north of Charlemagne Boulevard. Its legal property description is Part of Lot B Concession 11 Geographic Township of Cumberland in City of Ottawa (Ward 1 – Orleans East - Cumberland). Presently, the residential development site under consideration houses a 1-storey brick dwelling in which the house is located near the front of the lot. At the rear of the site, there is a 1 ½ - storey siding building which fronts Duvernay Drive and is connected to the municipal roadway by a gravel driveway. The municipal address of the property is referenced as 1670 Tenth Line Road.

The lot under consideration is approximately 1,857.76 square metres. This property is proposed for the development of a (3) storey residential apartment building plus a basement. The total gross floor area at each floor will cover an area of approximately 6,602.0 $ft^2$ (613.0 $m^2$) and overall the proposed building is approximately 20,021.4 square feet (1,859.0 $m^2$) excluding the basement level.

The building will house a total of 30 apartment units, including twelve (12) 2-bedroom units, nine (9) 1-bedroom and den units, seven (7) 1-bedroom units and two (2) bachelor units. The stormwater outlet for this site is the existing 375 mm diameter storm sewer located within the Duvernay Drive road right-of-way.

From storm-drainage criteria set by the staff at the City of Ottawa's Engineering Department, the allowable post-development runoff release rates shall not exceed the five (5)-Year pre-development conditions. The allowable pre-development runoff coefficient is the lesser of the calculated "C" existing value = 0.61 or $C_{allow}$ = 0.5 maximum. If the uncontrolled stormwater runoff exceeds the specified requirements, the on-site stormwater management (SWM) control measures are necessary. The post-development run-off coefficient for this site is estimated at C = 0.80, which exceeds the calculated pre-development allowable $C_{allow}$ = 0.5 criteria for the Duvernay Drive storm sewer without on-site SWM control. Therefore, SWM measures are required. Refer to the attached Drainage Area Plan (Figure 1) as detailed in Appendix A. For Pre and Post site development characteristics, refer to the Storm Drainage Area Plan Dwg. No. 825-8 D-1 for details.

This report will address and detail the grading, drainage and stormwater management control measures required to develop this property. Based on the Proposed Site Grading and Servicing Plan (Dwg. No. 825-8 G-1), and on the Proposed Stormwater Management Plan (Dwg. No. 825-8 SWM-1), the storm water of this lot will be regulated and stored on-site by the building's flat rooftop c/w controlled roof drains plus the asphalt parking lot surface with installation of an ICD in CB/MH#1.

The stormwater management calculations that follow will detail the extent of on-site SWM control to be implemented and the storage volume required on-site to attain where possible, the appropriate rumoff release that will conform to the City's established drainage criteria and review requirements.

Because the site will be connecting to and outletting into a separated Duvernay Drive storm sewer, therefore, the approval exemption under Ontario Regulations 525/98 would apply since storm water discharges from this site will outlet flow into a downstream storm sewer. Thus, an Environmental Compliance Application (ECA) will not be required to be submitted to the Ministry.

## Site Data

1. Development Property Area

Post-Development Site Area Characteristics

| | |
|---|---|
| Development Lot Area | = 1,857.76 $m^2$ |
| Roof Surface Area | = 655.19 $m^2$ |
| Asphalt Area | = 770.81 $m^2$ |
| Interlock/Concrete Area | = 151.98 $m^2$ |
| Grass Area | = 279.78 $m^2$ |

$$C = \frac{(655.19 \times 0.9) + (770.81 \times 0.9) + (151.98 \times 0.9) + (279.78 \times 0.2)}{1{,}857.76}$$

$$C = \frac{1{,}475.138}{1{,}857.76}$$

$$C = 0.794$$

Say "C" $= 0.80$

Therefore, the average post-development "C" for this site is 0.80.

2. Controlled Area Data (NODE No. 1 to NODE No. 6)

| | |
|---|---|
| Roof Surface Area | = 655.19 $m^2$ |
| Asphalt Area | = 761.68 $m^2$ |
| Interlock/Concrete Area | = 135.92 $m^2$ |
| Grass Area | = 117.07 $m^2$ |
| Total Storm-water Controlled Area | = 1,669.86 $m^2$ |

$$C = \frac{(655.19 \times 0.9) + (761.68 \times 0.9) + (117.07 \times 0.2) + (135.92 \times 0.9)}{1{,}669.86}$$

$$C = \frac{1{,}420.925}{1{,}669.86}$$

$$C = 0.8509$$

Say "C" $= 0.85$

Therefore, the post-development "C" for the controlled storm-water drainage area is 0.85.

3. Uncontrolled Area Data (NODE No. 7 and NODE No. 8)

| | |
|---|---|
| Grass Area | = 162.71 $m^2$ |
| Interlock/Concrete Area | = 16.06 $m^2$ |
| Asphalt Area | = 9.13 $m^2$ |
| Total Storm-water Uncontrolled Area | = 187.90 $m^2$ |

$$C_5 = \frac{(16.06 \times 0.9) + (9.13 \times 0.9) + (162.71 \times 0.2)}{187.90}$$

$$C_5 = \frac{55.213}{187.90}$$

$$C_5 = 0.294$$

$$\text{Say "}C_5\text{"} = 0.30$$

Therefore, the post-development "$C_5$" for the uncontrolled stormwater drainage area of the site is $C_5 = 0.30$.

$$C_{100} = \frac{(16.06 \times 1.0) + (9.13 \times 1.0) + (162.71 \times 0.2 \times 1.25)}{187.90}$$

$$C_{100} = \frac{65.868}{187.90}$$

$$C_{100} = 0.351$$

$$\text{Say "}C_{100}\text{"} = 0.35$$

Therefore, the post-development "$C_{100}$" for the uncontrolled stormwater drainage area of the site is $C_{100} = 0.35$.

- Tributary Area consisting of approximately 187.90 $m^2$ will be outletting off-site uncontrolled from the residential apartment building site.

- The SWM Area to be controlled is 1,669.86 $m^2$. Refer to the attached Figure 1 entitled "Drainage Area Plan" in Appendix "A" for details.

- The site SWM storage area excluding the apartment building rooftop area shall be controlled by an ICD in CB/MH#1 is 1,669.86 $m^2$ – 655.19 $m^2$ = 1,014.67 $m^2$ or 0.1015 ha.

The ICD type recommended is Hydrovex Model No. 125-VHV-2 or equivalent. See Appendix "C" for details.

## Pre-Development Flow Estimation

Maximum Allowable Off-Site Flow: Five (5)-Year Storm

Node #101

Pre-Development Site Area Characteristics

| | |
|---|---|
| Development Lot Area | = 1,857.76 $m^2$ |
| Asphalt Area | = 395.62 $m^2$ |
| Concrete/Pool Area | = 85.74 $m^2$ |
| Roof Area | = 489.03$m^2$ |
| Grass Area | = 748.37 $m^2$ |
| Gravel Area | = 139.0 $m^2$ |

$$C_{5pre} = \frac{(489.03 \times 0.9) + (395.62 \times 0.9) + (85.74 \times 0.9) + (748.37 \times 0.2) + (139.0 \times 0.8)}{1{,}857.76}$$

$$C_{5pre} = \frac{1{,}134.225}{1{,}857.76}$$

$$C_{5pre} = 0.611$$

Say "$C_{5pre}$" $= 0.61 > C_{5allow} = 0.5$

$\therefore$ Use $C_{pre} = 0.50$ allowable for re-development

$T_c$ = D/V where D = 66.5 m, ΔH = 0.46 m, S = 0.7 %, and V = 1.80 feet/second = 0.55 m/s

Therefore,

$$T_c = \frac{66.5\text{m}}{0.55\ \text{m/s}}$$

$T_c = 2.02$ minutes

Use $T_c = 10$ minutes

$I_5$ = 104.2 mm/hr [City of Ottawa, five (5)-Year storm]

Using the Rational Method

Q = 2.78 (0.5) (104.2) (0.1858)

Q = 26.91 L/s

Therefore, the total allowable flow off-site is 26.91 L/s.

The pre-development flow of the five (5)-Year and 100-year storm event draining **off-site** from this lot is as follows:

Where, Tc = 10 min.

$$Q_{5pre} = 2.78\ \text{CIA}$$

$$C_{5pre} = \frac{1{,}134.225}{1{,}857.76}$$

$$C_{5pre} = 0.611$$

Say, $C_{5pre} = 0.61$

$$Q_{5pre} = 2.78\ (0.61)\ (104.2)\ (0.1858)$$

$$= 32.83\ \text{L/s}$$

$$C_{100pre} = \frac{(489.03\ \times 1.0) + (395.62 \times 1.0) + (85.74 \times 1.0) + (748.37 \times 0.2 \times 1.25) + (139.0 \times 1.0)}{1{,}857.76}$$

$$C_{100pre} = \frac{1{,}296.48}{1{,}857.76}$$

$$C_{100pre} = 0.698$$

Say, $C_{100pre} = 0.70$

$$Q_{100pre} = 2.78\ (0.70)\ (178.6)\ (0.1858)$$

$= 64.58$ L/s draining off-site

Therefore under current site conditions the 5-Year pre-development flow is estimated at 32.83 L/s and the 100 year pre-development flow is estimated at 64.58 L/s.

A coloured Google image and aerial photography of these current pre-development conditions of the site is provided in Appendix "B" of this report for reference.

Since 187.90 $m^2$ is drained uncontrolled off-site, therefore, accordingly the net allowable discharge for this site into the existing Duvernay Drive storm sewer is Q = [2.78 (0.5) (104.2) (0.1858)] – [2.78 (0.35) (178.6) (0.0188)] = 26.91 L/s – 3.27 L/s = 23.64 L/s.

## Stormwater Management Analysis

The calculated flow rate of 23.64 L/s for on-site stormwater management detention volume storage will be used for this SWM analysis. Since a total of four (4) controlled roof drains are proposed to restrict flow from the building at a rate of 3.80 L/s (4 × 0.95 L/s) into the Duvernay Drive storm sewer, therefore, the remainder of the site allowable release rate from the ICD in CB/MH#1 is 23.64 L/s – 3.80 L/s = 19.84 L/s.

Therefore, the total allowable 5-Year release rate of 19.84 L/s will be entering into the existing 375 mm dia. Duvernay Drive storm sewer from the site drainage system controlled by ICD in CB/MH#1. Runoff greater than the allowable release rate will be stored on-site in the proposed stormwater management system consisting of parking lot surface, underground storm pipes and drainage structures and the flat rooftop of the proposed residential apartment building will be used for stormwater attenuation purposes.

The inflow rate during the 5-Year and 100-Year storm from the parking lot surface area, underground drainage system and rooftop areas can now be calculated as follows:

## Design Discharge Computation

1. Proposed Parking Lot Surface and Underground Pipe Drainage System (NODE No. 5 to NODE No. 6)

   The Rational Method was used to estimate peak flows.

   Q = 2.78 CIA

   Inflow rate $Q_{ACTUAL}$ for the site is:

   C = (AVG "C" value of controlled area excluding office building roof area)

| | | |
|---|---|---|
| Asphalt Area | = 761.68 $m^2$ | |
| Interlock/Concrete Area | = 135.92 $m^2$ | |
| Grass Area | = 117.07 $m^2$ | |
| Total Storm-water Controlled Area | = 1,014.67 $m^2$ | *(excluding roof area of main building)* |

5-Year Event

$$C_5 = \frac{(761.68 \times 0.9) + (135.92 \times 0.9) + (117.07 \times 0.2)}{1{,}014.67}$$

$$C_5 = \frac{831.254}{1{,}014.67}$$

$C_5 = 0.819$ Say "$C_5$" $= 0.82$

A = 0.1015 ha.

Inflow Rate $(Q_A)_5$ = 2.78 CIA

= 2.78 (0.82) (0.1015) I

= 0.2314 I I = (mm/hr)

100-Year Event

$$C_{100} = \frac{(761.68 \times 1.0) + (135.92 \times 1.0) + (117.07 \times 0.2 \times 1.25)}{1{,}014.67}$$

$$C_{100} = \frac{926.868}{1{,}014.67}$$

$$C_{100} = 0.914$$

Say "$C_{100}$" $= 0.92$

A = 0.1015 ha.

Inflow Rate $(Q_A)_5$ = 2.78 CIA

= 2.78 (0.92) (0.1015) I

= 0.2596 I    I = (mm/hr)

The 100-Year inflow rate for the controlled site tributary area can be calculated as follows:

$Q_{100}$ = 0.2596 I

This now can be used to determine the storage volume for the site using the Modified Rational Method.

- Actual Flow ($Q_{ACTUAL}$) is calculated as:

Q = 2.78 CIA

- $Q_{STORED}$ is calculated as:

$Q_S = Q_A - Q_{ALLOW}$

2. To Calculate Roof Storage Requirements (NODE No. 1, NODE No. 2, NODE No. 3 and NODE No. 4)

The proposed flat roof of the residential apartment building on this property will consist of four (4) rooftop areas for stormwater attenuation in which each area will incorporate one (1) roof drain per area to control flow off-site. The specified roof drain maximum flow rate per each drain is 0.95 L/s (15 U.S. gal/min.) under a maximum head of 150mm. Therefore, the stormwater flow that can be controlled from this rooftop and outletted off-site is 3.80 L/s (4 × 0.95 L/s).

C = 0.9 will be used for sizing roof storage volume in this case for the 5-Year event and C = 1.0 will be used for the 100-Year event.

Inflow rate ($Q_A$) = 2.78 CIA

Where $C = 0.9$

A = Surface are of roof

$I$ = (mm/hr)

5-Year Event

For Roof Area 1 (NODE No. 1)

$Q_{A1} = 2.78$ CIA

$C = 0.90$

$A = 163.76\ m^2$

$I$ = mm/hr

$= 2.78\ (0.90)\ (0.0164)\ I$

$= 0.0411\ I$

For Roof Area 2 (NODE No. 2)

$Q_{A2} = 2.78$ CIA

$C = 0.90$

$A = 162.58\ m^2$

$I$ = mm/hr

$= 2.78\ (0.90)\ (0.0163)\ I$

$= 0.0408\ I$

For Roof Area 3 (NODE No. 3)

$Q_{A3} = 2.78$ CIA

$C = 0.90$

$A = 162.53\ m^2$

$I$ = mm/hr

$= 2.78\ (0.90)\ (0.0163)\ I$

$= 0.0408\ I$

For Roof Area 4 (NODE No. 4)

$Q_{A4} = 2.78$ CIA

$C = 0.90$

$A = 166.32\ m^2$

$I$ = mm/hr

$= 2.78\ (0.90)\ (0.01663)\ I$

$= 0.0416\ I$

100-Year Event

For Roof Area 1 (NODE No. 1)

$Q_{A1} = 2.78$ CIA

$C = 1.0$

$A = 163.76\ m^2$

$I =$ mm/hr

$= 2.78\ (1.0)\ (0.0164)\ I$

$= 0.0456\ I$

For Roof Area 2 (NODE No. 2)

$Q_{A2} = 2.78$ CIA

$C = 1.0$

$A = 162.58\ m^2$

$I =$ mm/hr

$= 2.78\ (1.0)\ (0.0163)\ I$

$= 0.0454\ I$

For Roof Area 3 (NODE No. 3)

$Q_{A3} = 2.78$ CIA

$C = 0.90$

$A = 162.53\ m^2$

$I =$ mm/hr

$= 2.78\ (1.0)\ (0.0163)\ I$

$= 0.0454\ I$

For Roof Area 4 (NODE No. 4)

$Q_{A4} = 2.78$ CIA

$C = 0.90$

$A = 166.32\ m^2$

$I =$ mm/hr

$= 2.78\ (1.0)\ (0.01663)\ I$

$= 0.0463\ I$

Summary results of the calculated inflow and the required storage volume of the site and the building's flat rooftop to store the 5-Year and 100-Year storm events are shown on the Tables 1 to 4 and 6 to 9 inclusive.

**Table 11** summarizes the post-development design flows from the building roof top area as well as the type of roof drains, the maximum anticipated ponding depths, storage volumes required, and storage volumes provided for the five (5)-year and 100-year design events.

**Table 11 :** Design Flow and Roof Drain Table

| Roof Area ID & Drainage Area (ha) | Number of Roof Drains | Watts Roof Drain Model ID (Weir Opening) | Controlled Flow per Drain (L/s) | | Approximate Ponding Depth Above Drains (m) | | Storage Volume Required ($m^3$) | | Max. Storage Available ($m^3$) |
|---|---|---|---|---|---|---|---|---|---|
| | | | 5 YR | 100 YR | 5 YR | 100 YR | 5 YR | 100 YR | |
| No. 1 (0.0164 ha) | 1 | RD-100-A-ADJ (1/4 OPENING EXPOSED) | 0.83 | 0.95 | 0.11 | 0.15 | 2.51 | 5.94 | 7.70 |
| No. 2 (0.0163 ha) | 1 | RD-100-A-ADJ (1/4 OPENING EXPOSED) | 0.83 | 0.95 | 0.11 | 0.15 | 2.49 | 5.91 | 7.70 |
| No. 3 (0.0163 ha) | 1 | RD-100-A-ADJ (1/4 OPENING EXPOSED) | 0.83 | 0.95 | 0.11 | 0.15 | 2.49 | 5.91 | 7.70 |
| No. 4 (0.0167 ha) | 1 | RD-100-A-ADJ (1/4 OPENING EXPOSED) | 0.83 | 0.95 | 0.11 | 0.15 | 2.55 | 6.08 | 7.54 |
| Total Roof (0.655 ha) | 4 | - | **3.32** | **3.80** | - | - | **10.04** | **23.84** | **30.64** |

## Water Quality

Storm water quality treatment is required for this proposed development.

For this site, based on the City of Ottawa's drainage criteria and on typical recommendations set out by Rideau Valley Conservation Authority (RVCA), water quality treatment for 80 percent (min.) removal of total suspended solids (TSS) is required for development of this property.

The said property is in the watershed area where the existing 375 mm diameter storm sewer fronting on Duvernay Drive outlets to a water course where no municipal treatment for water quality is provided. Therefore, a Stormceptor system is proposed to support the water quality improvement objective. Stormceptor (Model EFO-4) was selected to provide the water quality removal of TSS at a level above 80 percent, which is above the minimum requirement of 80 percent TSS removal. In addition to TSS removal, the Stormceptor system is also an oil and sediment separator. Refer to Appendix "D" for the Stormceptor sizing details from the manufacturer.

## Erosion and Sediment Control

The contractor shall implement Best Management Practices to provide for protection of the receiving storm sewer during construction activities. These practices are required to ensure no sediment and/or associated pollutants are released to the receiving watercourse. These practices include installation of a "siltsack" catch basin sediment control device or equal in catch basins as recommended by manufacturer on-site and off-site within the Duvernay Drive and Tenth Line Road road right of way adjacent to this property. Siltsack shall be inspected every 2 to 3 weeks and after every major storm. The deposits will be disposed of as per the requirements of the contract. See Dwg. No. 825-8 ESC-1 for details. Additionally, silt sacs shall be placed on all storm sewer maintenance hole openings during construction. A mud mat is proposed to be installed at the construction site access in order to protect the public road right of way from potential construction traffic damages.

Additionally, silt sacs shall be placed on all storm sewer maintenance hole openings during construction. A mud mat is proposed to be installed at the construction site access in order to protect the public road right-of-way from potential construction traffic damages.

## Conclusion

To develop this site (±0.1858 ha. in size) and in controlling the 5-Year stormwater release rate off-site to an allowable rate of 26.91 L/s, a calculated site storage volume of approximately 13.88 $m^3$ (min.) is required during the 5-Year event, See Table No. 1 to 10 inclusive. We estimate that the required storage volume is 10.04 $m^3$ (min.) from rooftop storage and 3.85 $m^3$ (min.) from the site asphalt parking lot surface area are necessary to attenuate the 5-Year storm event. See Table No. 1 to 5 inclusive.

During the 5-Year storm event for the flat rooftop storage, the ponding depth on this rooftop is estimated at 110 mm at Drain No. 1, 2, 3 and 4 and 0 mm at the roof perimeter assuming a 1.7% (min.) roof pitch to the drains. The rooftop storage available at Roof Area No. 1 is 3.19 $m^3$, Roof Area No. 2 is 3.19 $m^3$, Roof Area No. 3 is 3.19 $m^3$ and Roof Area No. 4 is 2.96 $m^3$ for a total of 12.53 $m^3$ which is greater than the required volume of 10.04 $m^3$.

As for the remaining storage volume of 3.85 $m^3$ (min.) required from the site development area for the 5-Year storm event, the estimated H.W.L. of 87.91 m will provide a total available asphalt surface storage volume of 4.83 $m^3$. In total, the 5-Year available site storage volume (roof and parking lot) is approximately 17.36 $m^3$ which is greater than the required site storage volume of 13.88 $m^3$. See Appendix "E" for details.

In order to control the 100-Year stormwater release rate off-site to an allowable rate of 26.91 L/s, a calculated site storage volume of approximately 39.76 $m^3$ (min.) is required during the 100-Year event. We estimate that the required storage volume of 23.84 $m^3$ (min.) of rooftop storage and 15.92 $m^3$ (min.) from the site asphalt parking lot surface area are necessary to attenuate the 100-Year storm event. See Table No. 6 to 10 inclusive.

During the 100-year storm event for the flat rooftop storage, the ponding depth on this rooftop is estimated at 150 mm at Drain No. 1, 2, 3 and 4 and 0 mm at the roof perimeter assuming a 1.7% (min.) roof pitch to the drains. The rooftop storage available at Roof Area No. 1 is 7.70 $m^3$, Roof Area No. 2 is 7.70 $m^3$, Roof Area No. 3 is 7.70 $m^3$ and Roof Area No. 4 is 7.54 $m^3$ for a total of 30.64 $m^3$ which is greater than the required volume of 23.84 $m^3$.

As for the remaining storage volume of 15.92 $m^3$ (min.) required from the asphalt parking area for the 100-Year storm event, the estimated H.W.L. of 87.96 m will provide a total available asphalt surface storage volume of 16.24 $m^3$. In total, the 100-Year available site storage volume (roof and parking lot) is 46.88 $m^3$ which is greater than the required site storage volume of 39.76 $m^3$. See Appendix "E" for details.

Therefore, by means of flat building rooftop storage, grading the site to the proposed grades and constructing the proposed parking lot area and drainage system as shown on the Proposed Site Grading and Servicing Plan (Dwg. No. 825-8, G-1), the desirable 5-Year and 100-Year storm event attenuation volume of 17.36 $m^3$ and 46.88 $m^3$ respectively will be available on-site.

In order to control the release flow rate off-site from the controlled drainage area of the lot, an inlet control device (ICD) will be installed at the outlet of CB/MH#1 in the 250 mm diameter storm pipe (outlet pipe) with Q = 19.84 L/s under a head of 2.34 m. A rooftop drain with a release rate of 0.95 L/s (under a maximum head of 150 mm) will be installed at Roof Drain #1, #2, #3 and #4 of the proposed residential apartment building flat rooftop as depicted on (Dwg. No. 825-8, G-1). The 5-Year and 100-Year flow off-site is restricted to 26.91 L/s.

An inlet control device (ICD) will be installed at the outlet of CB/MH#1 in the 250 mm diameter storm pipe (outlet pipe) with Q = 19.84 L/s under a head of 2.34 m. The ICD type recommended is a Hydrovex Regulator (125-VHV-2) or equivalent. See Appendix "C" for ICD details.

The building weeping tile drainage will outlet via its separate 150 mm diameter PVC storm lateral. The roof drains will be outletted also via a separate 150 mm diameter PVC storm lateral from the residential apartment building which "wye" into the proposed 150 mm dia. weeping tile storm lateral, where upon both laterals are outletting to the existing Duvernay Drive 375 mm diameter storm sewer with only one (1) connection. The City of Ottawa recommends that pressurized drain pipe material be used in the building for the roof drain leader pipe in the event of surcharging in the City storm sewer system. Refer to the proposed site grading and servicing plan (Dwg. No. 825-8, G-1) for details.

To achieve a minimum of 80 percent TSS removal, a Stormceptor structure (Model EFO-4) is proposed to be installed for the site development of this property. This Stormceptor structure shall be located downstream of the proposed CB/MH#1, which houses the site's inlet control device (ICD). Based on the Stormceptor system that is proposed for this site, size of the lot, and impervious ratio, a greater than 80 percent TSS removal is estimated for all rainfall events including large storms. (See Appendix "D" for details).

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PROVINCE OF ONTARIO

PROPOSED 1670 TENTH LINE ROAD RESIDENTAIL APARTMENT BUILDING DEVELOPMENT SITE

**TABLE 1**

**FIVE (5)-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 1 STORAGE VOLUME**

**ROOF DRAIN No. 1**

| $t_c$ **TIME** (minutes) | **I** **5-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 104.20 | 4.28 | 0.83 | 3.45 | 2.07 |
| 15 | 83.50 | 3.43 | 0.83 | 2.60 | 2.34 |
| 20 | 70.30 | 2.89 | 0.83 | 2.06 | 2.47 |
| 25 | 60.90 | 2.50 | 0.83 | 1.67 | <u>2.51</u> |
| 30 | 53.93 | 2.22 | 0.83 | 1.39 | 2.50 |
| 35 | 48.50 | 1.99 | 0.83 | 1.16 | 2.44 |
| 40 | 44.20 | 1.82 | 0.83 | 0.99 | 2.38 |

Therefore, the required rooftop storage volume is 2.51 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #1
- Area = 0.0164 ha.
- $C_5 = 0.9$

**TABLE 2**

**FIVE (5)-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 2 STORAGE VOLUME**

**ROOF DRAIN No. 2**

| $t_c$ **TIME** (minutes) | **I 5-YEAR** (mm/hr) | **Q ACTUAL** (L/s) | **Q ALLOW** (L/s) | **Q STORED** (L/s) | **VOLUME STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 104.20 | 4.25 | 0.83 | 3.42 | 2.05 |
| 15 | 83.50 | 3.41 | 0.83 | 2.58 | 2.32 |
| 20 | 70.30 | 2.87 | 0.83 | 2.04 | 2.45 |
| 25 | 60.90 | 2.49 | 0.83 | 1.66 | 2.49 |
| 30 | 53.93 | 2.20 | 0.83 | 1.37 | 2.47 |
| 35 | 48.50 | 1.98 | 0.83 | 1.15 | 2.42 |
| 40 | 44.20 | 1.80 | 0.83 | 0.97 | 2.33 |

Therefore, the required rooftop storage volume is 2.49 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #2
- Area = 0.0163 ha.
- $C_5 = 0.9$

**TABLE 3**

**FIVE (5)-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 3 STORAGE VOLUME**

**ROOF DRAIN No. 3**

| $t_c$ **TIME** (minutes) | **I** **5-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME** **STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 104.20 | 4.25 | 0.83 | 3.42 | 2.05 |
| 15 | 83.50 | 3.41 | 0.83 | 2.58 | 2.32 |
| 20 | 70.30 | 2.87 | 0.83 | 2.04 | 2.45 |
| 25 | 60.90 | 2.49 | 0.83 | 1.66 | 2.49 |
| 30 | 53.93 | 2.20 | 0.83 | 1.37 | 2.47 |
| 35 | 48.50 | 1.98 | 0.83 | 1.15 | 2.42 |
| 40 | 44.20 | 1.80 | 0.83 | 0.97 | 2.33 |

Therefore, the required storage volume is 2.49 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #3
- Area = 0.0163 ha.
- $C_5 = 0.9$

**TABLE 4**

**FIVE (5)-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 4 STORAGE VOLUME**

**ROOF DRAIN No. 4**

| $t_c$ **TIME** (minutes) | **I** **5-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME** **STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 104.20 | 4.34 | 0.83 | 3.51 | 2.11 |
| 15 | 83.50 | 3.47 | 0.83 | 2.64 | 2.38 |
| 20 | 70.30 | 2.92 | 0.83 | 2.09 | 2.51 |
| 25 | 60.90 | 2.53 | 0.83 | 1.70 | 2.55 |
| 30 | 53.93 | 2.24 | 0.83 | 1.41 | 2.54 |
| 35 | 48.50 | 2.02 | 0.83 | 1.19 | 2.50 |
| 40 | 44.20 | 1.84 | 0.83 | 1.01 | 2.42 |
| 45 | 40.60 | 1.69 | 0.83 | 0.86 | 2.32 |

Therefore, the required rooftop storage volume is 2.55 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #4
- Area = 0.01663 ha.
- $C_5 = 0.9$

**TABLE 5**

**FIVE (5)-YEAR EVENT**

**SITE REQUIRED STORAGE VOLUME**

**ASPHALT PARKING LOT AREA**

| $t_c$ **TIME** (minutes) | **I** **5-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME STORED** ($m^3$) |
|---|---|---|---|---|---|
| 5 | 141.20 | 32.67 | 19.84 | 12.83 | 3.85 |
| 10 | 104.20 | 24.11 | 19.84 | 4.27 | 2.56 |
| 15 | 83.50 | 19.32 | 19.84 | 0 | 0 |

Therefore, the required storage volume is 3.85 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #5 and Node #6
- Area = 0.1015 ha.
- $C_5 = 0.82$

**TABLE 6**

**100-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 1 STORAGE VOLUME**

**ROOF DRAIN No. 1**

| $t_c$ **TIME** (minutes) | **I** **100-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME** **STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 178.6 | 8.14 | 0.95 | 7.19 | 4.31 |
| 15 | 142.9 | 6.52 | 0.95 | 5.57 | 5.01 |
| 20 | 120.0 | 5.47 | 0.95 | 4.52 | 5.42 |
| 25 | 103.9 | 4.74 | 0.95 | 3.79 | 5.69 |
| 30 | 91.90 | 4.19 | 0.95 | 3.24 | 5.83 |
| 35 | 82.60 | 3.77 | 0.95 | 2.82 | 5.92 |
| 40 | 75.10 | 3.43 | 0.95 | 2.48 | 5.93 |
| 45 | 69.10 | 3.15 | 0.95 | 2.20 | 5.94 |
| 50 | 63.90 | 2.92 | 0.95 | 1.97 | 5.91 |
| 55 | 59.62 | 2.72 | 0.95 | 1.77 | 5.84 |
| 60 | 55.90 | 2.55 | 0.95 | 1.60 | 5.76 |

Therefore, the required rooftop storage volume is 5.94 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #1
- Area = 0.0.164 ha.
- $C_{100}$ = 1.0

**TABLE 7**

**100-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 2 STORAGE VOLUME**

**ROOF DRAIN No. 2**

| $t_c$ TIME (minutes) | I 100-YEAR (mm/hr) | Q ACTUAL (L/s) | Q ALLOW (L/s) | Q STORED (L/s) | VOLUME STORED ($m^3$) |
|---|---|---|---|---|---|
| 10 | 178.6 | 8.11 | 0.95 | 7.16 | 4.30 |
| 15 | 142.9 | 6.49 | 0.95 | 5.54 | 4.99 |
| 20 | 120.0 | 5.45 | 0.95 | 4.50 | 5.40 |
| 25 | 103.9 | 4.72 | 0.95 | 3.77 | 5.66 |
| 30 | 91.90 | 4.17 | 0.95 | 3.22 | 5.80 |
| 35 | 82.60 | 3.75 | 0.95 | 2.80 | 5.88 |
| 40 | 75.10 | 3.41 | 0.95 | 2.46 | 5.90 |
| 45 | 69.10 | 3.14 | 0.95 | 2.19 | 5.91 |
| 50 | 63.90 | 2.90 | 0.95 | 1.95 | 5.85 |
| 55 | 59.62 | 2.71 | 0.95 | 1.76 | 5.81 |

Therefore, the required rooftop storage volume is 5.91 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #2
- Area = 0.0163 ha.
- $C_{100}$ = 1.0

**TABLE 8**

**100-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 3 STORAGE VOLUME**

**ROOF DRAIN No. 3**

| $t_c$ **TIME** (minutes) | **I** **100-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 178.6 | 8.11 | 0.95 | 7.16 | 4.30 |
| 15 | 142.9 | 6.49 | 0.95 | 5.54 | 4.99 |
| 20 | 120.0 | 5.45 | 0.95 | 4.50 | 5.40 |
| 25 | 103.9 | 4.72 | 0.95 | 3.77 | 5.66 |
| 30 | 91.90 | 4.17 | 0.95 | 3.22 | 5.80 |
| 35 | 82.60 | 3.75 | 0.95 | 2.80 | 5.88 |
| 40 | 75.10 | 3.41 | 0.95 | 2.46 | 5.90 |
| 45 | 69.10 | 3.14 | 0.95 | 2.19 | 5.91 |
| 50 | 63.90 | 2.90 | 0.95 | 1.95 | 5.85 |
| 55 | 59.62 | 2.71 | 0.95 | 1.76 | 5.81 |

Therefore, the required rooftop storage volume is 5.91 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #3
- Area = 0.0163 ha.
- $C_{100}$ = 1.0

**TABLE 9**

**100-YEAR EVENT**

**REQUIRED BUILDING ROOF AREA 4 STORAGE VOLUME**

**ROOF DRAIN No. 4**

| $t_c$ **TIME** (minutes) | **I** **100-YEAR** (mm/hr) | **Q** **ACTUAL** (L/s) | **Q** **ALLOW** (L/s) | **Q** **STORED** (L/s) | **VOLUME** **STORED** ($m^3$) |
|---|---|---|---|---|---|
| 10 | 178.6 | 8.27 | 0.95 | 7.32 | 4.39 |
| 15 | 142.9 | 6.62 | 0.95 | 5.67 | 5.10 |
| 20 | 120.0 | 5.56 | 0.95 | 4.61 | 5.53 |
| 25 | 103.9 | 4.81 | 0.95 | 3.86 | 5.79 |
| 30 | 91.90 | 4.26 | 0.95 | 3.31 | 5.96 |
| 35 | 82.60 | 3.82 | 0.95 | 2.87 | 6.03 |
| 40 | 75.10 | 3.48 | 0.95 | 2.53 | 6.07 |
| 45 | 69.10 | 3.20 | 0.95 | 2.25 | 6.08 |
| 50 | 63.90 | 2.96 | 0.95 | 2.01 | 6.03 |
| 55 | 59.62 | 2.76 | 0.95 | 1.81 | 5.97 |

Therefore, the required rooftop storage volume is 6.08 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #4
- Area = 0.01663 ha.
- $C_{100}$ = 1.0

**TABLE 10**

**100-YEAR EVENT**

**SITE REQUIRED STORAGE VOLUME**

**ASPHALT PARKING LOT AREA**

| $t_c$ TIME (minutes) | I 100-YEAR (mm/hr) | Q ACTUAL (L/s) | Q ALLOW (L/s) | Q STORED (L/s) | VOLUME STORED ($m^3$) |
|---|---|---|---|---|---|
| 10 | 178.6 | 46.37 | 19.84 | 26.53 | 15.92 |
| 15 | 142.9 | 37.10 | 19.84 | 17.26 | 15.53 |
| 20 | 120.0 | 37.15 | 19.84 | 11.31 | 13.57 |
| 25 | 103.9 | 26.97 | 19.84 | 7.13 | 10.69 |
| 30 | 91.90 | 23.86 | 19.84 | 4.02 | 7.24 |

Therefore, the required storage volume is 15.92 $m^3$.

**SITE DATA**

- Drainage Area ID = Node #5 and Node #6
- Area = 0.1015 ha.
- $C_{100}$ = 0.92

**PROPOSED**

**THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE**

**PART OF LOT B**

**CONCESSION 11**

**GEOGRAPHIC TOWNSHIP OF CUMBERLAND**

**1670 TENTH LINE ROAD**

**CITY OF OTTAWA**

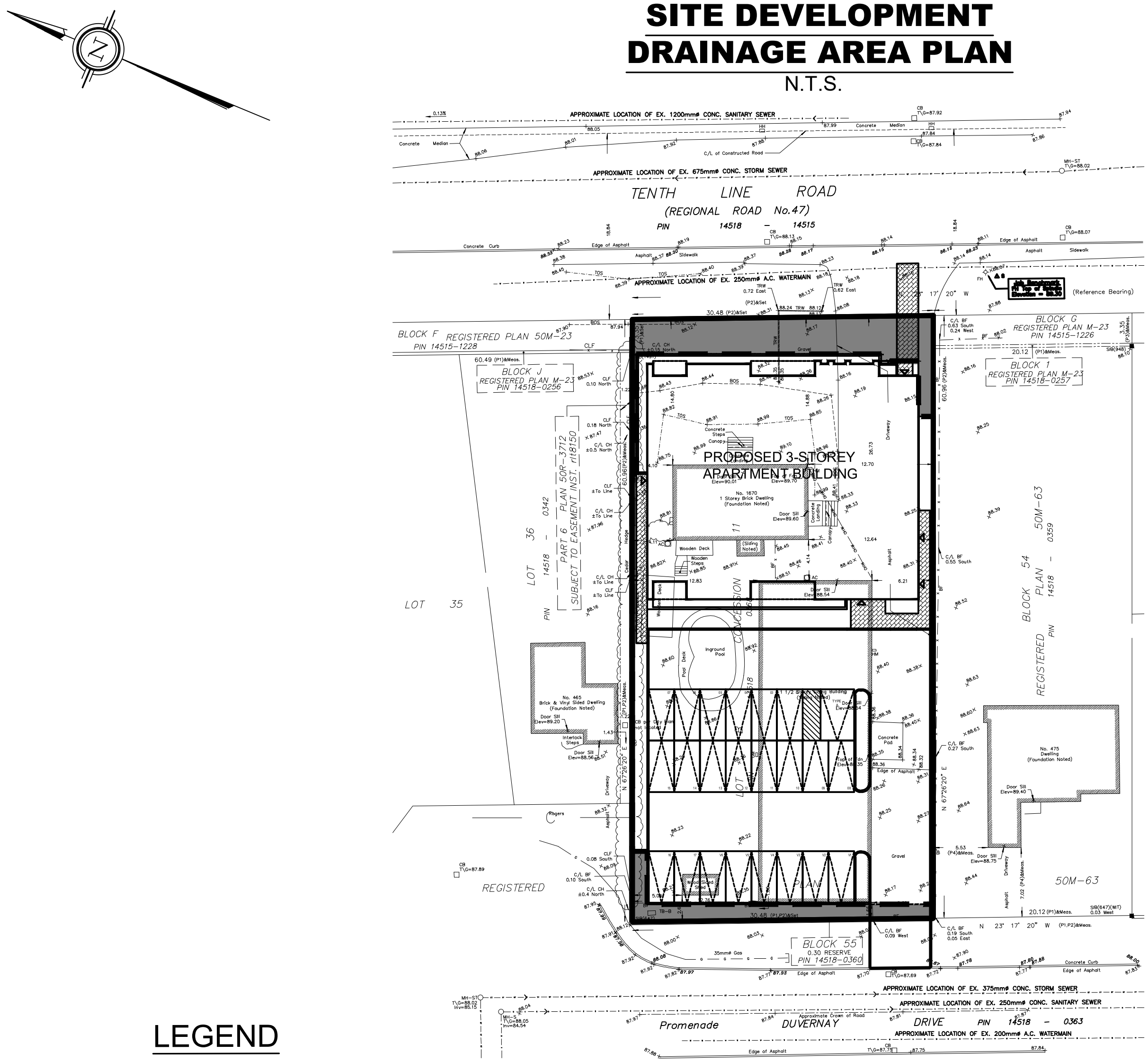
# APPENDIX A

## STORM DRAINAGE AREA PLAN

## FIGURE 1

# PROPOSED 1670 TENTH LINE ROAD
# SITE DEVELOPMENT
# DRAINAGE AREA PLAN

N.T.S.




## LEGEND

LIMIT OF CONTROLLED STORM
DRAINAGE AREA = 1,669.86 SQ. M

UNCONTROLLED STORM
DRAINAGE AREA = 187.90 SQ. M

TOTAL AREA = 1,857.76 SQ. M

POST-DEVELOPMENT SITE
AVERAGE "C" = 0.80

T.L. MAK ENGINEERING CONSULTANTS LTD.
CONSULTING ENGINEERS

| PROJECT No. | DATE | DRAWING No. |
|---|---|---|
| 825-8 | JULY 2025 | FIGURE 1 |

**PROPOSED**

**THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE**

**PART OF LOT B**

**CONCESSION 11**

**GEOGRAPHIC TOWNSHIP OF CUMBERLAND**

**1670 TENTH LINE ROAD**

**CITY OF OTTAWA**

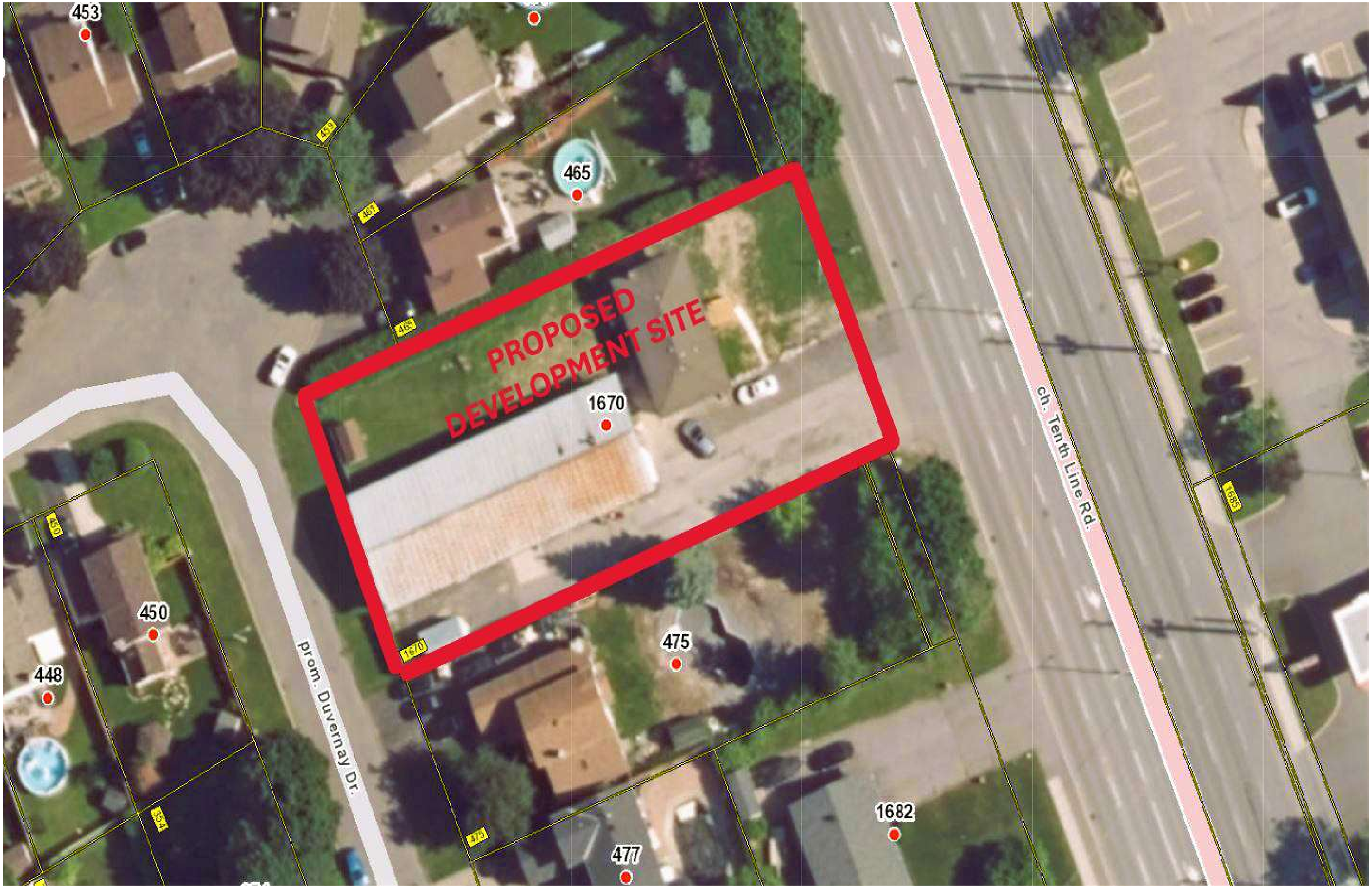
**APPENDIX B**

**SITE PRE-DEVELOPMENT CONDITION**

**GOOGLE IMAGE 2021**

**AND**

**AERIAL PHOTOGRAPY 2022 (GEOOTTAWA)**



PROPOSED
DEVELOPMENT SITE
1670
465
453
450
448
475
477
1682
ch. Tenth Line Rd.
prom. Duvernay Dr.


2022


PROPOSED DEVELOPMENT SITE
1670
465
475
477
1682
453
450
448
ch. Tenth Line Rd.
prom. Duvernay Dr.

2022

**1670** TENTH LINE ROAD

2021

**FRONTING DUVERNAY DRIVE**

2021

PROPOSED

THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE

PART OF LOT B

CONCESSION 11

GEOGRAPHIC TOWNSHIP OF CUMBERLAND

1670 TENTH LINE ROAD

CITY OF OTTAWA

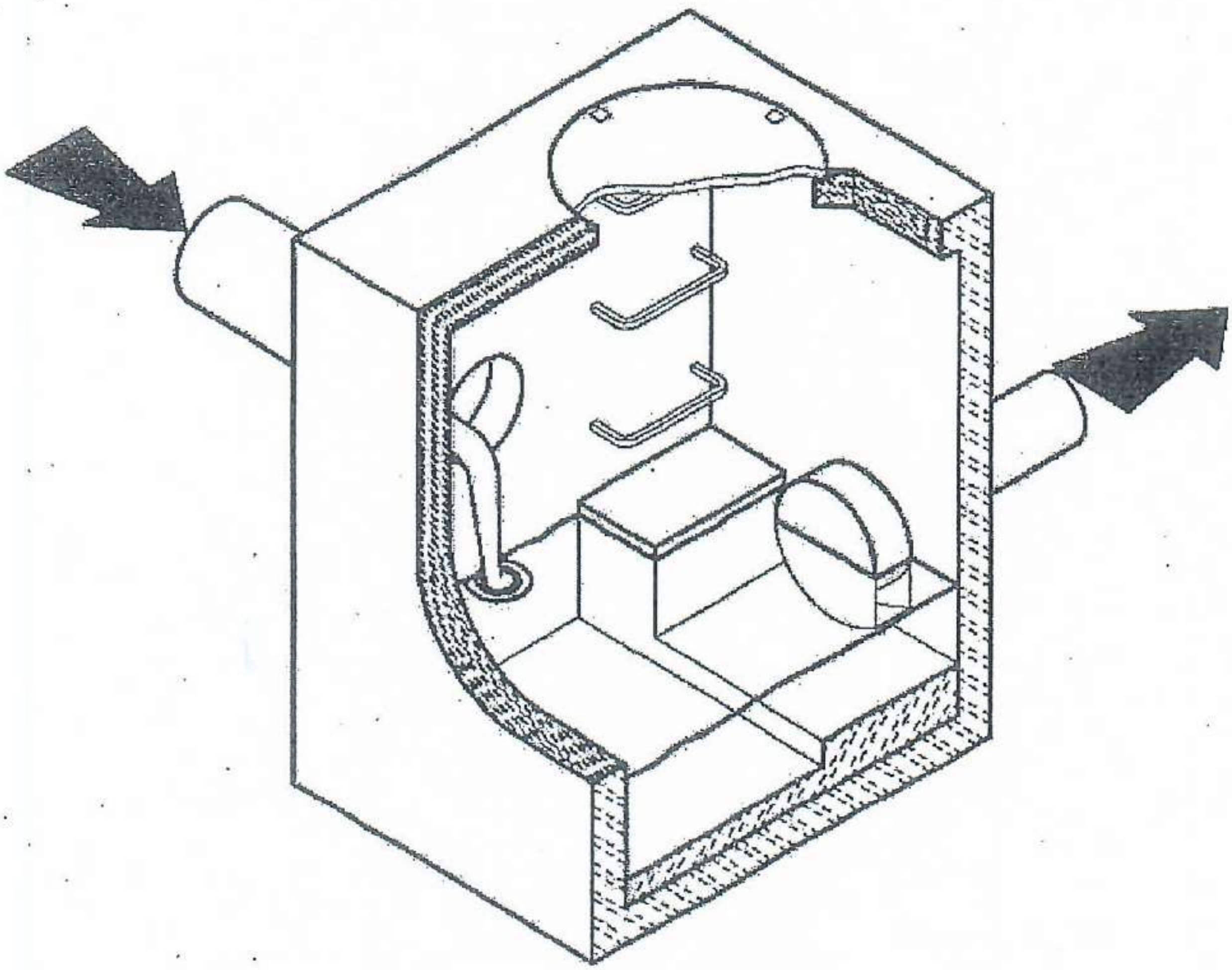
# APPENDIX C

# INLET CONTROL DEVICE (ICD) DETAILS

# HYDROVEX MODEL No. 125-VHV-2

# CSO/STORMWATER MANAGEMENT

## ® HYDROVEX® VHV / SVHV
## Vertical Vortex Flow Regulator



JOHN MEUNIER

# HYDROVEX® VHV / SVHV VERTICAL VORTEX FLOW REGULATOR

## APPLICATIONS

One of the major problems of urban wet weather flow management is the runoff generated after a heavy rainfall. During a storm event, uncontrolled flows may overload the drainage system and cause flooding. Sewer pipe wear and network deterioration are increased dramatically as a result of increased flow velocities. In a combined sewer system, the wastewater treatment plant will experience a significant increase in flows during storms, thereby losing its treatment efficiency.

A simple means of managing excessive water runoff is to control excessive flows at their point of origin, the manhole. **John Meunier Inc.** manufactures the **HYDROVEX® VHV / SVHV** line of vortex flow regulators for point source control of stormwater flows in sewer networks, as well as manholes, catch basins and other retention structures.

The **HYDROVEX® VHV / SVHV** design is based on the fluid mechanics principle of the forced vortex. The discharge is controlled by an air-filled vortex which reduces the effective water passage area without physically reducing orifice size. This effect grants precise flow regulation without the use of moving parts or electricity, thus minimizing maintenance. Although the concept is quite simple, over 12 years of research and testing have been invested in our vortex technology design in order to optimize its performance.

The **HYDROVEX® VHV / SVHV** Vertical Vortex Flow Regulators (refer to **Figure 1**) are manufactured entirely of stainless steel, and consist of a hollow body (1) (in which flow control takes place) and an outlet orifice (7). Two rubber "O" rings (3) seal and retain the unit inside the outlet pipe. Two stainless steel retaining rings (4) are welded on the outlet sleeve to ensure that there is no shifting of the "O" rings during installation and operation.

1. BODY
2. SLEEVE
3. O-RING
4. RETAINING RINGS (SQUARE BAR)
5. ANCHOR PLATE
6. INLET
7. OUTLET ORIFICE

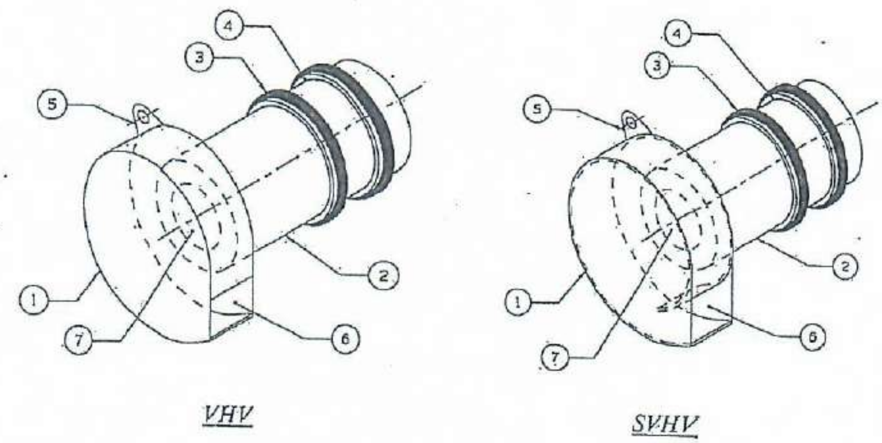
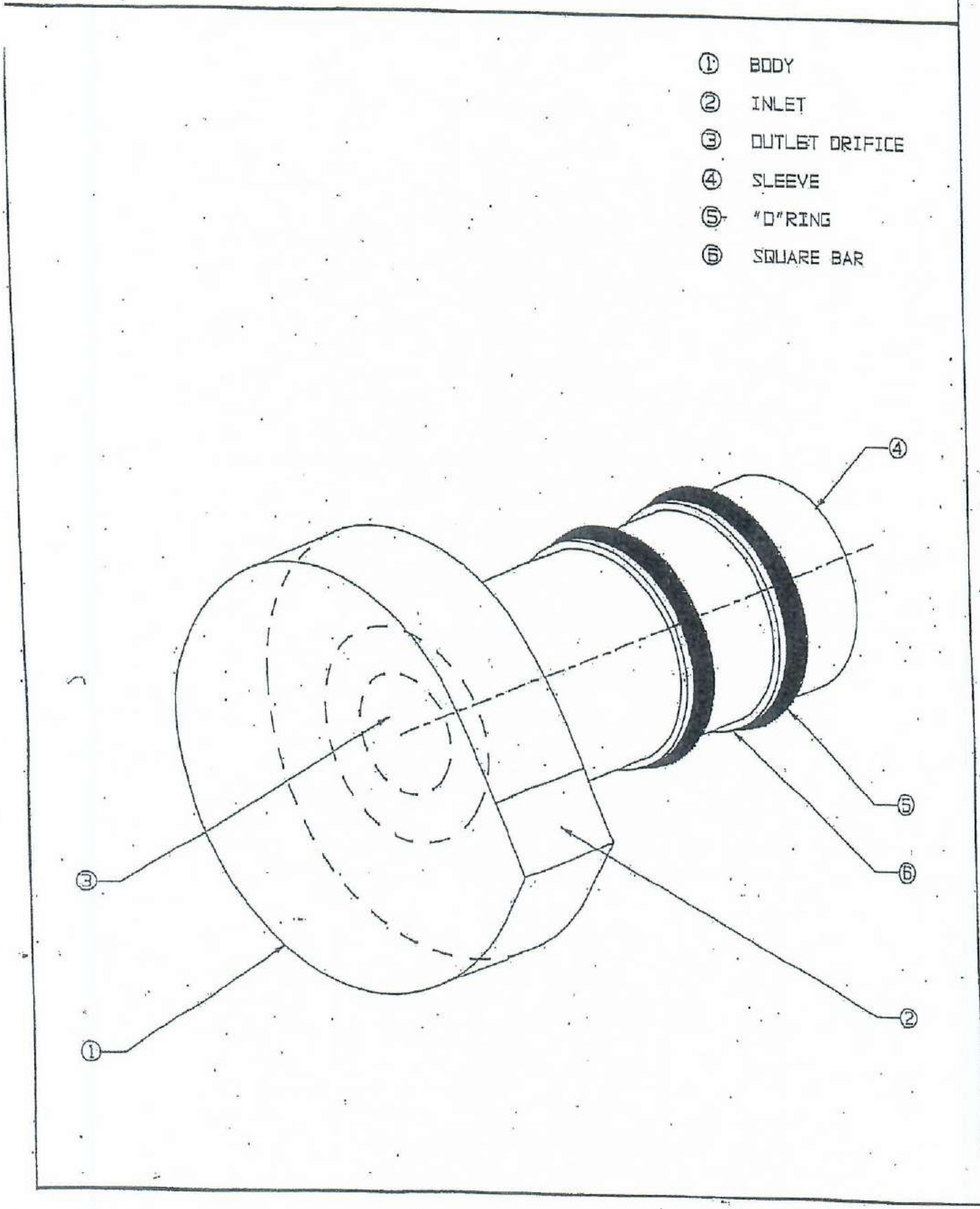



*FIGURE 1: HYDROVEX® VHV-SVHV VERTICAL VORTREX FLOW REGULATORS*

## ADVANTAGES

- As a result of the air-filled vortex, a **HYDROVEX®** VHV / SVHV flow regulator will typically have an opening 4 to 6 times larger than an orifice plate. Larger opening sizes decrease the chance of blockage caused by sediments and debris found in stormwater flows. **Figure 2** shows the discharge curve of a vortex regulator compared to an equally sized orifice plate. One can see that for the same height of water and same opening size, the vortex regulator controls a flow approximately four times smaller than the orifice plate.
- Having no moving parts, they require minimal maintenance.
- Submerged inlet for floatables control.
- The **HYDROVEX®** VHV / SVHV line of flow regulators are manufactured entirely of stainless steel, making them durable and corrosion resistant.
- Installation of the **HYDROVEX®** VHV / SVHV flow regulators is quick and straightforward and is performed after all civil works are completed.
- Installation requires no assembly, special tools or equipment and may be carried out by any contractor.

FIGURE 1-VHV



# HYDROVEX®
## VHV/SVHV Vortex Flow Regulator

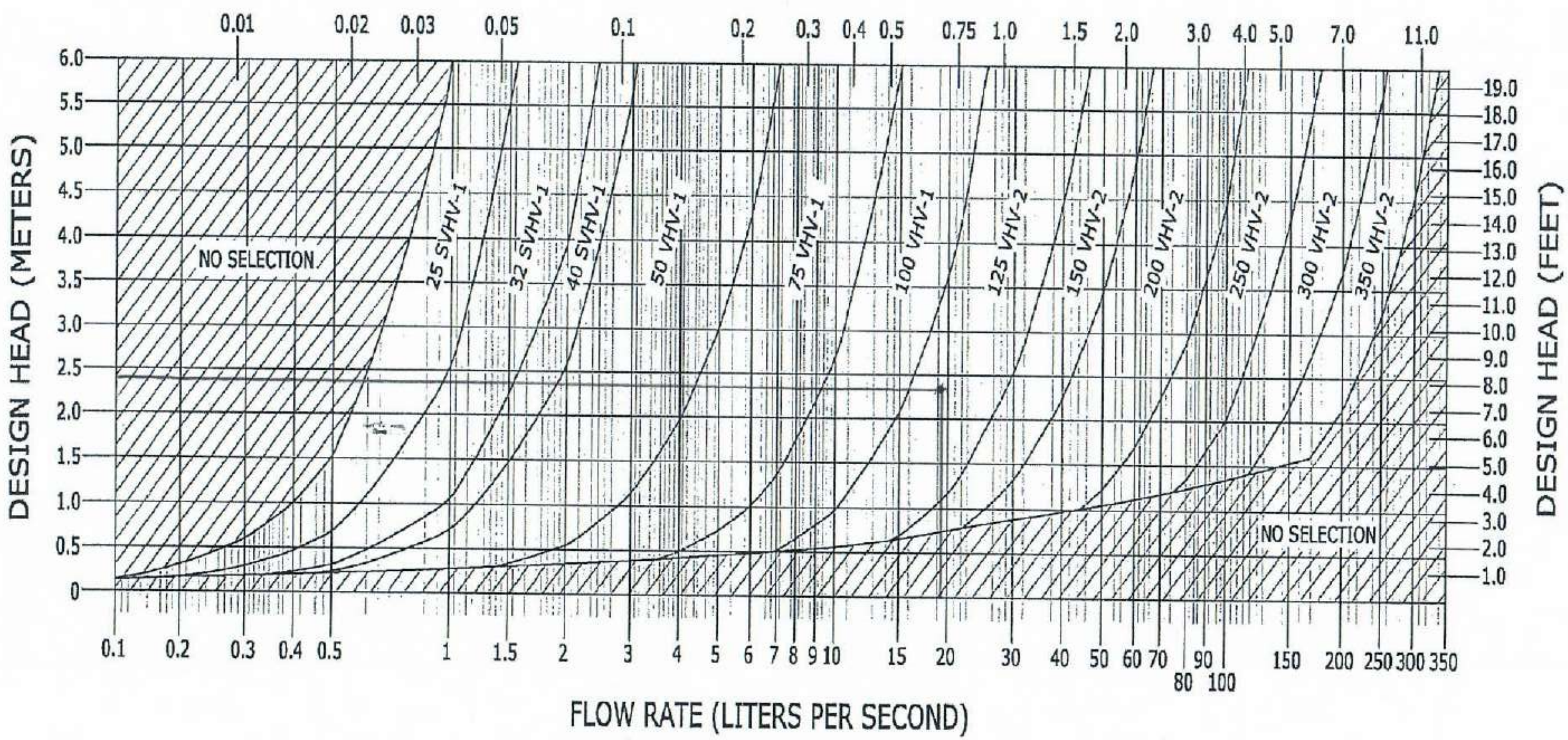


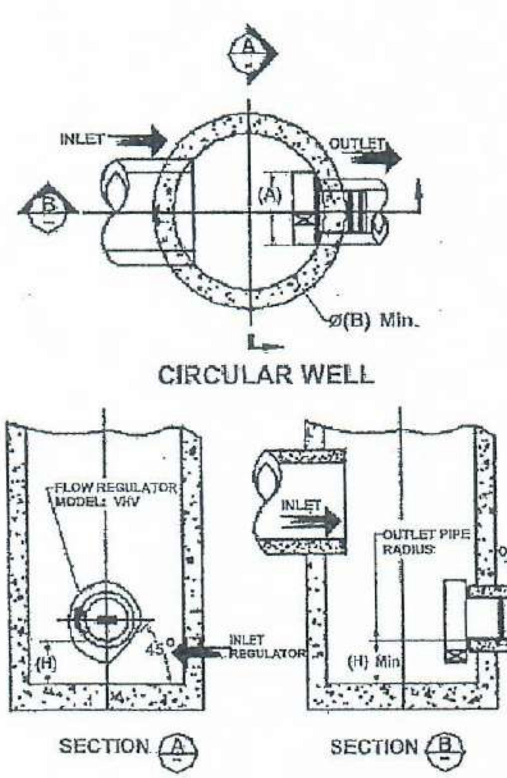

FIGURE 3

JOHN MEUNIER

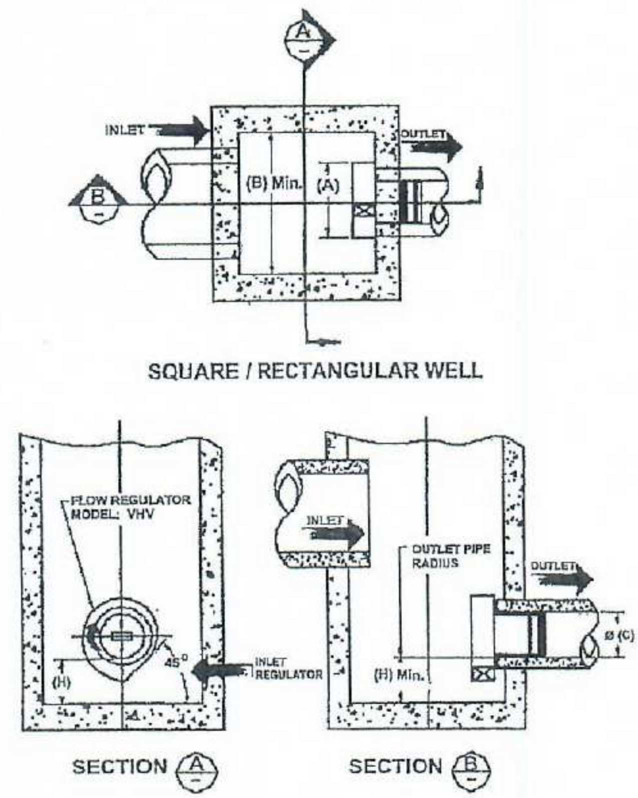
*TYPICAL INSTALLATION OF A VORTEX FLOW REGULATOR IN A CIRCULAR OR SQUARE/RECTANGULAR MANHOLE*
*FIGURE 4*

| Model | Regulator Diameter A (mm) [in] | CIRCULAR Minimum Manhole Diameter B (mm) [in] | SQUARE Minimum Chamber Width B (mm) [in] | Minimum Outlet Pipe Diameter C (mm) [in] | Minimum Clearance H (mm) [in] |
|---|---|---|---|---|---|
| 25 SVHV-1 | 125 [5] | 600 [24] | 600 [24] | 150 [6] | 150 [6] |
| 32 SVHV-1 | 150 [6] | 600 [24] | 600 [24] | 150 [6] | 150 [6] |
| 40 SVHV-1 | 200 [8] | 600 [24] | 600 [24] | 150 [6] | 150 [6] |
| 50 VHV-1 | 150 [6] | 600 [24] | 600 [24] | 150 [6] | 150 [6] |
| 75 VHV-1 | 250 [10] | 600 [24] | 600 [24] | 150 [6] | 150 [6] |
| 100 VHV-1 | 325 [13] | 900 [36] | 600 [24] | 150 [6] | 200 [8] |
| 125 VHV-2 | 275 [11] | 900 [36] | 600 [24] | 150 [6] | 200 [8] |
| 150 VHV-2 | 350 [14] | 900 [36] | 600 [24] | 150 [6] | 225 [9] |
| 200 VHV-2 | 450 [18] | 1200 [48] | 900 [36] | 200 [8] | 300 [12] |
| 250 VHV-2 | 575 [23] | 1200 [48] | 900 [36] | 250 [10] | 350 [14] |
| 300VHV-2 | 675 [27] | 1600 [64] | 1200 [48] | 250 [10] | 400 [16] |
| 350VHV-2 | 800 [32] | 1800 [72] | 1200 [48] | 300 [12] | 500 [20] |

**Circular Manhole**




**Square / Rectangular Manhole**




*NOTE:* *In the case of a square manhole, the outlet pipe must be centered on the wall to ensure that there is enough clearance for installation of the regulator.*

PROPOSED

THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE

PART OF LOT B

CONCESSION 11

GEOGRAPHIC TOWNSHIP OF CUMBERLAND

1670 TENTH LINE ROAD

CITY OF OTTAWA

# APPENDIX D

# STORMCEPTOR MODEL No. EFO-4

# SIZING AND DETAILS

JULY 18, 2025

## Tony Mak

**From:** Brandon O'Leary [brandon.oleary@RinkerPipe.com]
**Sent:** July 18, 2025 3:50 PM
**To:** Tony Mak
**Cc:** Jessica Steffler
**Subject:** RE: [EXTERNAL] 1670 Tenth Line Road
**Attachments:** image001.png; 250717 Stormceptor EFO Sizing Report, 1670 Tenth Line Rd. Ottawa, T.L. Mak, Tony Mak.pdf; Stormceptor EFO4.PDF; Stormceptor EFO - Oil Grit Separator specification (rev 12-23).doc; Stormceptor EFO - Oil Grit Separator Specification (rev 12-23).pdf

Hello Tony,

Great to hear from you as always. Attached is the requested Stormceptor EFO (ISO 14034/ETV verified) sizing report for 80% TSS removal of the MoE FINE PSD. Please note that if OGS units are not credited by the reviewing agency for 80% TSS removal, the report should be updated to use the ETV PSD. Based on the site parameters provided below, the EFO4 is recommended; standard drawing attached (DWG format also available). This unit has a budget cost of $22,377 including purchase, delivery to site, and the 5-year value-added Quality Assurance Program unique to Stormceptor. I have also attached the standard spec. for the EFO and provided the capacities of the recommended unit below, both for incorporation into the spec. If you need anything else at all, please let me know.

Stormceptor EFO4 Capacities:
Maximum Treatment Flow Rate: 10.4 L/s
Maintenance Sediment Volume: 270 L
Maximum Sediment Capacity: 1,190 L
Maximum Hydrocarbon Storage Capacity: 265 L
Total Storage Volume: 1,780 L

Best Regards,

Brandon O'Leary, P.Eng., B.A.Sc.
Stormwater Specialist
Bowmanville/Cambridge Plant
Cell: (905) 630-0359

Rinker
MATERIALS
*A QUIKRETE COMPANY*

**We are excited to announce that Forterra is now Rinker Materials**
***Stormceptor***
*Protecting the water for future generations*
*Our Online Sizing Tool for the Stormceptor EFO:* https://www.imbriumsystems.com/login?returnurl=%2flaunch-pcswmm-for-stormceptor

**From:** Tony Mak <tlmakecl@bellnet.ca>
**Sent:** Friday, July 18, 2025 3:09 PM
**To:** Brandon O'Leary <brandon.oleary@RinkerPipe.com>
**Cc:** Jessica Steffler <jessica.steffler@RinkerPipe.com>
**Subject:** [EXTERNAL] 1670 Tenth Line Road

**CAUTION: This email originated from outside of the organization. Exercise caution when opening attachments or clicking links, especially from *UNKNOWN* senders.**

Hi Brandon,

Currently we are working on a project in the east end of the City of Ottawa, Ontario. Regarding the above-noted site, we are requesting your assistance in sizing a Stormceptor structure for TSS removal of 80% (min.). Attached is a PDF of our Site Grading and Servicing Plan (Dwg. #825-8 G-1) for your reference.

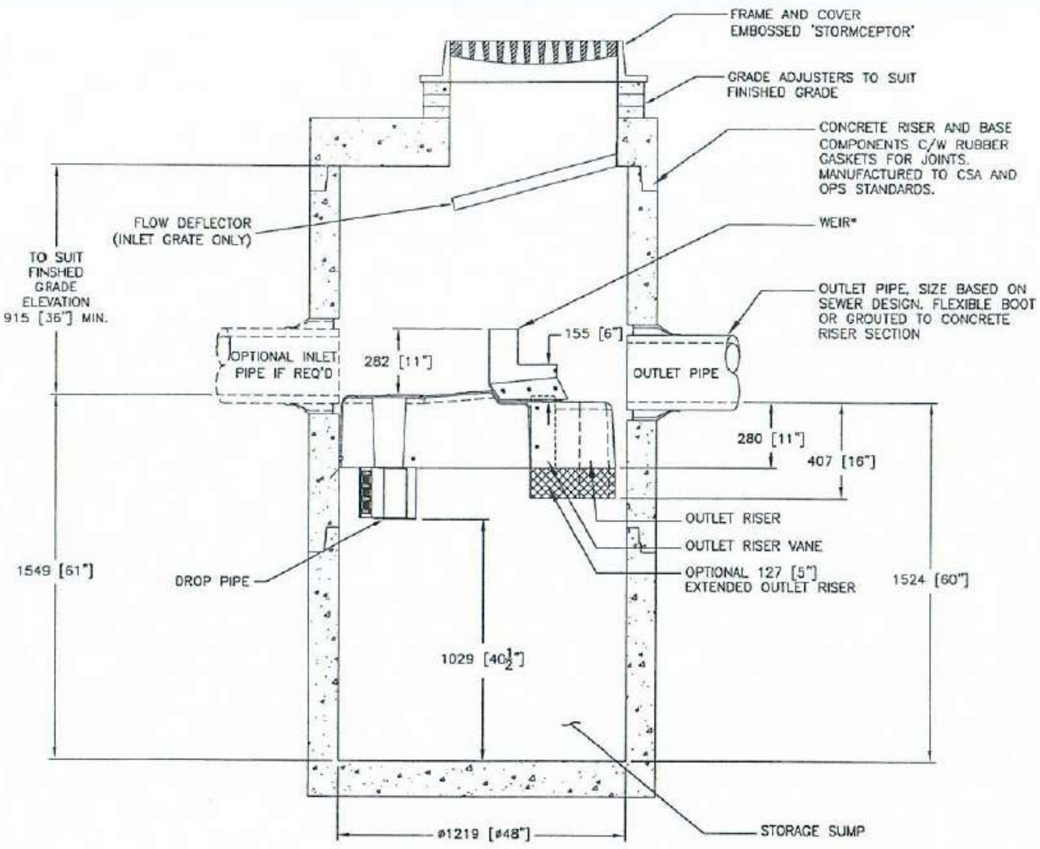
The total site area is ±1,857.76 $m^2$. The controlled area regulated by the Stormceptor is approximately ±0.1015 ha. The impervious area within the controlled area of the site is ±0.09 ha. and mainly comprised of the asphalt parking and concrete area. Please size the Stormceptor unit accordingly at your earliest convenience for our report. The "$C_5$" value is 0.82 and "$C_{100}$" value is 0.92 within the Stormceptor regulated/controlled area. If you need more information, please let me know.

Thank you for your help. Have a great day!

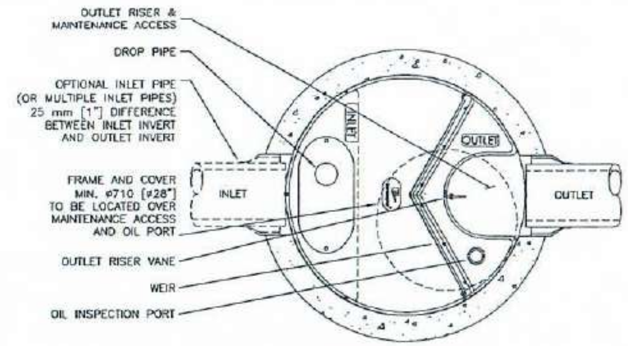
Regards,

Tony Mak

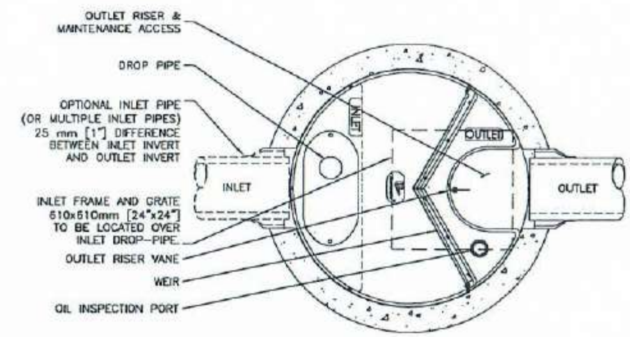
T.L. Mak Engineering Consultants Ltd.
1455 Youville Drive, Suite 218
Ottawa, ON. K1C 6Z7
Tel. 613-837-5516 | Fax: 613-837-5277
E-mail: tlmakecl@bellnet.ca




SECTION VIEW




PLAN VIEW (STANDARD)




PLAN VIEW (INLET TOP)

FOR SITE SPECIFIC DRAWINGS PLEASE CONTACT YOUR LOCAL STORMCEPTOR REPRESENTATIVE. SITE SPECIFIC DRAWINGS ARE BASED ON THE BEST AVAILABLE INFORMATION AT THE TIME. SOME FIELD REVISIONS TO THE SYSTEM LOCATION OR CONNECTION PIPING MAY BE NECESSARY BASED ON AVAILABLE SPACE OR SITE CONFIGURATION REVISIONS. ELEVATIONS SHOULD BE MAINTAINED EXCEPT WHERE NOTED ON BYPASS STRUCTURE (IF REQUIRED).

**GENERAL NOTES:**

* MAXIMUM SURFACE LOADING RATE (SLR) INTO LOWER CHAMBER THROUGH DROP PIPE IS 1135 L/min/m$^2$ (27.9 gpm/ft$^2$) FOR STORMCEPTOR EF4 AND 535 L/min/m$^2$ (13.1 gpm/ft$^2$) FOR STORMCEPTOR EFO4 (OIL CAPTURE CONFIGURATION). WEIR HEIGHT IS 150 mm (6 INCH) FOR EFO4.

1. ALL DIMENSIONS INDICATED ARE IN MILLIMETERS (INCHES) UNLESS OTHERWISE SPECIFIED.
2. STORMCEPTOR STRUCTURE INLET AND OUTLET PIPE SIZE AND ORIENTATION SHOWN FOR INFORMATIONAL PURPOSES ONLY.
3. UNLESS OTHERWISE NOTED, BYPASS INFRASTRUCTURE, SUCH AS ALL UPSTREAM DIVERSION STRUCTURES, CONNECTING STRUCTURES, OR PIPE CONDUITS CONNECTING TO COMPLETE THE STORMCEPTOR SYSTEM SHALL BE PROVIDED AND ADDRESSED SEPARATELY.
4. DRAWING FOR INFORMATION PURPOSES ONLY. REFER TO ENGINEER'S SITE/UTILITY PLAN FOR STRUCTURE ORIENTATION.
5. NO PRODUCT SUBSTITUTIONS SHALL BE ACCEPTED UNLESS SUBMITTED 10 DAYS PRIOR TO PROJECT BID DATE, OR AS DIRECTED BY THE ENGINEER OF RECORD.

**INSTALLATION NOTES**

A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STRUCTURE (LIFTING CLUTCHES PROVIDED)
C. CONTRACTOR WILL INSTALL AND LEVEL THE STRUCTURE, SEALING THE JOINTS, LINE ENTRY AND EXIT POINTS (NON-SHRINK GROUT WITH APPROVED WATERSTOP OR FLEXIBLE BOOT)
D. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT THE DEVICE FROM CONSTRUCTION-RELATED EROSION RUNOFF.
E. DEVICE ACTIVATION, BY CONTRACTOR, SHALL OCCUR ONLY AFTER SITE HAS BEEN STABILIZED AND THE STORMCEPTOR UNIT IS CLEAN AND FREE OF DEBRIS.

# STANDARD DETAIL
# NOT FOR CONSTRUCTION

| SITE SPECIFIC DATA REQUIREMENTS | | | | | |
|---|---|---|---|---|---|
| STORMCEPTOR MODEL | | EFO4 | | | |
| STRUCTURE ID | | | | | * |
| HYDROCARBON STORAGE REQ'D (L) | | | | | * |
| WATER QUALITY FLOW RATE (L/s) | | | | | * |
| PEAK FLOW RATE (L/s) | | | | | * |
| RETURN PERIOD OF PEAK FLOW (yrs) | | | | | * |
| DRAINAGE AREA (HA) | | | | | * |
| DRAINAGE AREA IMPERVIOUSNESS (%) | | | | | * |
| PIPE DATA: | I.E. | MAT'L | DIA | SLOPE % | HGL |
| INLET #1 | * | * | * | * | * |
| INLET #2 | * | * | * | * | * |
| OUTLET | * | * | * | * | * |
| * PER ENGINEER OF RECORD | | | | | |

| MARK | DATE | REVISION DESCRIPTION | BY |
|---|---|---|---|
| #### | #### | #### | #### |
| #### | #### | #### | #### |
| #### | #### | #### | #### |
| 1 | 6/8/18 | UPDATES | JSK |
| 0 | 5/26/17 | INITIAL RELEASE | JSK |

Stormceptor® EF

SCALE = NTS

imbrium

| | |
|---|---|
| DATE: 10/13/2017 | |
| DESIGNED: JSK | DRAWN: JSK |
| CHECKED: BSF | APPROVED: SP |
| PROJECT No.: EFO4 | SEQUENCE No.: * |
| SHEET: 1 OF 1 | |

# STANDARD SPECIFICATION FOR "OIL GRIT SEPARATOR" (OGS) STORMWATER QUALITY TREAMENT DEVICE WITH THIRD-PARTY VERIFIED LIGHT LIQUID RE-ENTRAINMENT SIMULATION PERFORMANCE TESTING RESULTS

## PART 1 – GENERAL

### 1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, designing, maintaining, and constructing an underground Oil Grit Separator (OGS) device for stormwater quality treatment, **specifically an OGS device that has been third-party tested for oil and fuel retention capability using a protocol for light liquid re-entrainment simulation testing, with testing results and a Statement of Verification in accordance with all the provisions of ISO 14034 Environmental Management – Environmental Technology Verification (ETV)**. Work includes supply and installation of concrete bases, precast sections, and the appropriate precast section with OGS internal components correctly installed within the system, watertight sealed to the precast concrete prior to arrival to the project site.

### 1.2 REFERENCE STANDARDS

1.2.1 For Canadian projects only, the following reference standards apply:

CAN/CSA-A257.4-14: Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections, and Fittings Using Rubber Gaskets
CAN/CSA-A257.4-14: Precast Reinforced Circular Concrete Manhole Sections, Catch Basins, and Fittings
CAN/CSA-S6-00: Canadian Highway Bridge Design Code

1.2.2 For ALL projects, the following reference standards apply:

| | |
|---|---|
| ASTM D-4097: | Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks |
| 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 C 891: | Standard Practice for Installation of Underground Precast Concrete Utility Structures |
| ASTM D2563: | Standard Practice for Classification of Visual Defects in Reinforced Plastics |

### 1.3 SHOP DRAWINGS

1.3.1 Shop drawings shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail the precast concrete components and OGS internal components prior to shipment, including the sequence for installation.

1.3.2 Unless directed otherwise by the Engineer of Record, OGS stormwater quality treatment product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record. Any and all changes to project cost estimates, bonding amounts, plan check fees for revision of approved documents, or design impacts due to regulatory requirements as a result of a product substitution shall be coordinated by the Contractor with the Engineer of Record.

### 1.4 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

1.4.1 OGS internal components supplied by the Manufacturer for attachment to the precast concrete vessel shall be pre-fabricated, bolted to the precast and watertight sealed to the precast vessel surface prior to site delivery to ensure Manufacturer's internal assembly process and quality control processes are fully adhered to, and to prevent materials damage on site.

1.4.2 Follow all instructions including the sequence for installation in the shop drawings during installation.

**PART 2 – PRODUCTS**

2.1 GENERAL

2.1.1 The OGS vessel shall be cylindrical and constructed from precast concrete riser and slab components.

2.1.2 The precast concrete OGS internal components shall include a fiberglass insert bolted and watertight sealed inside the precast concrete vessel, prior to site delivery. Primary internal components that are to be anchored and watertight sealed to the precast concrete vessel shall be done so only by the Manufacturer prior to arrival at the job site to ensure product quality.

2.1.3 The OGS shall be allowed to be specified and have the ability to function as a 240-degree bend structure in the stormwater drainage system, or as a junction structure.

2.1.4 The OGS to be specified shall have the capability to accept influent flow from an inlet grate and an inlet pipe.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be designed and manufactured to meet highway loading conditions per State/Provincial or local requirements.

2.3 GASKETS

Only profile neoprene or nitrile rubber gaskets that are oil resistant shall be accepted. For Canadian projects only, gaskets shall be in accordance to CSA A257.4-14. Mastic sealants, butyl tape/rope or Conseal CS-101 alone are not acceptable gasket materials.

2.4 JOINTS

The concrete joints shall be watertight and meet the design criteria according to ASTM C-990. For projects where joints require gaskets, the concrete joints shall be watertight and oil resistant and meet the design criteria according to ASTM C-443. Mastic sealants or butyl tape/rope alone are not an acceptable alternative.

2.5 FRAMES AND COVERS

Frames and covers shall be manufactured in accordance with State/Provincial or local requirements for inspection and maintenance access purposes. A minimum of one cover, at least 22-inch (560 mm) in diameter, shall be clearly embossed with the OGS manufacturer's product name to properly identify this asset's purpose is for stormwater quality treatment.

2.6 PRECAST CONCRETE

All precast concrete components shall conform to the appropriate CSA or ASTM specifications.

2.7 FIBERGLASS

The fiberglass portion of the OGS device shall be constructed in accordance with ASTM D2563, and in accordance with the PS15-69 manufacturing standard, and shall only be installed, bolted and watertight sealed to the precast concrete by the Manufacturer prior to arrival at the project site to ensure product quality.

2.8 OGS POLLUTANT STORAGE

The OGS device shall include a sump for sediment storage, and a fiberglass insert for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants. The total sediment storage capacity shall be a minimum 40 $ft^3$ (1.1 $m^3$). The total petroleum hydrocarbon storage capacity shall be a minimum 50 gallons (189 liters). The access opening to the sump of the OGS device for periodic inspection and maintenance purposes shall be a minimum 16 inches (406 mm) in diameter.

2.9 LADDERS

Ladder rungs shall be provided upon request or to comply with State/Provincial or local requirements.

2.10 INSPECTION

All precast concrete sections shall be level and inspected to ensure dimensions, appearance, integrity of internal components, and quality of the product meets State/Provincial or local specifications and associated standards.

**PART 3 – PERFORMANCE & DESIGN**

3.1 GENERAL

The OGS stormwater quality treatment device shall be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV). The OGS stormwater quality treatment device shall remove oil, sediment and gross pollutants from stormwater runoff during frequent wet weather events, and retain these pollutants during less frequent high flow wet weather events below the insert within the OGS for later removal during maintenance. The Manufacturer shall have at least ten (10) years of local experience, history and success in engineering design, manufacturing and production and supply of OGS stormwater quality treatment device systems, acceptable to the Engineer of Record.

3.2 HYDROLOGY AND RUNOFF VOLUME

The OGS device shall be engineered, designed and sized to treat a minimum of 90 percent of the average annual runoff volume, unless otherwise stated by the Engineer of Record, using historical rainfall data. Rainfall data sets should be comprised of a minimum 15-years of rainfall data or a longer continuous period if available for a given location, but in all cases a minimum 5-year period of rainfall data.

3.3 ANNUAL (TSS) SEDIMIMENT LOAD AND STORAGE CAPACITY

The OGS device shall be capable of removing and have sufficient storage capacity for the calculated annual total suspended solids (TSS) mass load and volume without scouring previously captured pollutants prior to maintenance being required. The annual (TSS) sediment load and volume transported from the drainage area should be calculated and compared to the OGS device's available storage capacity by the specifying Engineer to ensure adequate capacity between maintenance cycles. Sediment loadings shall be determined by land use and defined as a minimum of 450 kg (992 lb) of sediment (TSS) per impervious hectare of drainage area per year, or greater based on land use, as noted in Table 1 below.

Annual sediment volume calculations shall be performed using the projected average annual treated runoff volume, a typical sediment bulk density of 1602 $kg/m^3$ (100 $lbs/ft^3$) and an assumed Event Mean Concentration (EMC) of 125 mg/L TSS in the runoff, or as otherwise determined by the Engineer of Record.

Example calculation for a 1.3-hectares parking lot site:

- 1.28 meters of rainfall depth, per year
- 1.3 hectares of 100% impervious drainage area
- EMC of 125 mg/L TSS in runoff
- Treatment of 90% of the average annual runoff volume
- Target average annual TSS removal rate of 60% by OGS

Annual Runoff Volume:

- 1.28 m rain depth x 1.3 ha x 10,000 $m^2$/ha= 16,640 $m^3$ of runoff volume
- 16,640 $m^3$ x 1000 L/$m^3$ = 16,640,000 L of runoff volume
- 16,640,000 L x 0.90 = 14,976,000 L to be treated by OGS unit

Annual Sediment Mass and Sediment Volume Load Calculation:

- 14,976,000 L x 125 mg/L x kg/1,000,000 mg = 1,872 kg annual sediment mass
- 1,872 kg x $m^3$/1602 kg = 1.17 $m^3$ annual sediment volume
- 1.17 $m^3$ x 60% TSS removal rate by OGS = 0.70 $m^3$ minimum expected annual storage requirement in OGS

As a guideline, the U.S. EPA has determined typical annual sediment loads per drainage area for various sites by land use (see Table 1). Certain States, Provinces and local jurisdictions have also established such guidelines.

**Table 1 – Annual Mass Sediment Loading by Land Use**

| | Commercial | Parking Lot | Residential | | | Highways | Industrial | Shopping Center |
|---|---|---|---|---|---|---|---|---|
| | | | High | Med. | Low | | | |
| (lbs/acre/yr) | 1,000 | 400 | 420 | 250 | 10 | 880 | 500 | 440 |
| (kg/hectare/yr) | 1,124 | 450 | 472 | 281 | 11 | 989 | 562 | 494 |

**Source:** U.S. EPA Stormwater Best Management Practice Design Guide Volume 1, Appendix D, Table D-1, Burton and Pitt 2002

## 3.4 SIZING METHODOLOGY

The OGS device shall be engineered, designed and sized to provide stormwater quality treatment based on treating a minimum of 90 percent of the average annual runoff volume and a minimum removal of an annual average 60% of the sediment (TSS) load based on the Particle Size Distribution (PSD) specified in Table 2, Section 3.5, and based on third-party performance testing conducted in accordance with the Canadian Environmental Technology Verification (ETV) Program's **Procedure for Laboratory Testing of Oil-Grit Separators**. Sizing of the OGS shall be determined by use of a minimum ten (10) years of local historical rainfall data provided by Environment Canada. Sizing shall also be determined by use of the sediment removal performance data derived from the ISO 14034 ETV third-party verified laboratory testing data from testing conducted in accordance with the Canadian ETV protocol *Procedure for Laboratory Testing of Oil-Grit Separators*, as follows:

3.4.1 Sediment removal efficiency for a given surface loading rate and its associated flow rate shall be based on sediment removal efficiency demonstrated at the seven (7) tested surface loading rates specified in the protocol, ranging 40 L/min/$m^2$ to 1400 L/min/$m^2$, and as stated in the ISO 14034 ETV Verification Statement for the OGS device.

3.4.2 Sediment removal efficiency for surface loading rates between 40 L/min/$m^2$ and 1400 L/min/$m^2$ shall be based on linear interpolation of data between consecutive tested surface loading rates.

3.4.3 Sediment removal efficiency for surface loading rates less than the lowest tested surface loading rate of 40 L/min/$m^2$ shall be assumed to be identical to the sediment removal efficiency at 40 L/min/$m^2$. No extrapolation shall be allowed that results in a sediment removal efficiency that is greater than that demonstrated at 40 L/min/$m^2$.

3.4.4 Sediment removal efficiency for surface loading rates greater than the highest tested surface loading rate of 1400 L/min/$m^2$ shall assume zero sediment removal for the portion of flow that exceeds 1400 L/min/$m^2$, and shall be calculated using a simple proportioning formula, with 1400 L/min/$m^2$ in the numerator and the higher surface loading rate in the denominator, and multiplying the resulting fraction times the sediment removal efficiency at 1400 L/min/$m^2$.

The OGS device shall also have sufficient annual sediment storage capacity as specified and calculated in Section 3.3.

3.4.5 The Peclet Number is not an approved method or model for calculating TSS removal, sizing, or scaling OGS devices.

3.4.6 If an alternate OGS device is proposed, supporting documentation shall be submitted that demonstrates:

- Canadian ETV or ISO 14034 ETV Verification Statement which verifies third-party performance testing conducted in accordance with the **Procedure for Laboratory Testing of Oil-Grit Separators,** including the Light Liquid Re-entrainment Simulation Testing.
- Equal or better sediment (TSS) removal of the PSD specified in Table 2 at equivalent surface loading rates, as compared to the OGS device specified herein.
- Equal or better Light Liquid Re-entrainment Simulation Test results (using low-density polyethylene beads as a surrogate for light liquids such as oil and fuel) at equivalent surface loading rates, as compared to the OGS device specified herein. However, an alternative OGS device shall not be allowed as a substitute if the Light Liquid Re-entrainment Simulation Test was performed with screening components within the OGS device that are effective at retaining the low-density polyethylene beads, but would not be expected to retain light liquids such as oil and fuel.
- Equal or greater sediment storage capacity, as compared to the OGS device specified herein.
- Supporting documentation shall be signed and sealed by a local registered Professional Engineer. All costs associated with preparing and certifying this documentation shall be born solely by the Contractor.

## 3.5 PARTICLE SIZE DISTRIBUTION (PSD) FOR SIZING

The OGS device shall be sized to achieve the Engineer-specified average annual percent sediment (TSS) removal based solely on the test sediment used in the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.** This test sediment is comprised of inorganic ground silica with a specific gravity of 2.65, uniformly mixed, and containing a broad range of particle sizes as specified in Table 2. No alternative PSDs or deviations from Table 2 shall be accepted.

**Table 2**
**Canadian ETV Program Procedure for Laboratory Testing of Oil-Grit Separators**
**Particle Size Distribution (PSD) of Test Sediment**

| Particle Diameter (Microns) | % by Mass of All Particles | Specific Gravity |
|---|---|---|
| 1000 | 5% | 2.65 |
| 500 | 5% | 2.65 |
| 250 | 15% | 2.65 |
| 150 | 15% | 2.65 |
| 100 | 10% | 2.65 |
| 75 | 5% | 2.65 |
| 50 | 10% | 2.65 |
| 20 | 15% | 2.65 |
| 8 | 10% | 2.65 |
| 5 | 5% | 2.65 |
| 2 | 5% | 2.65 |

3.6 CANADIAN ETV or ISO 14034 ETV VERIFICATION OF SCOUR TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of completed third-party scour testing conducted and have in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators**. This scour testing is conducted with the device pre-loaded with test sediment comprised of the particle size distribution (PSD) illustrated in Table 2.

3.6.1 To be acceptable for on-line installation, the OGS device must demonstrate an average scour test effluent concentration less than 10 mg/L at each surface loading rate tested, up to and including 2600 L/min/$m^2$.

Data generated from laboratory scour testing performed with an OGS device pre-loaded with a coarser PSD than in Table 2 (i.e. the coarser PSD has no particles in the 1-micron to 50-micron size range, or the $D_{50}$ of the test sediment exceeds 75 microns) shall not be acceptable for the determination of the device's suitability for on-line installation.

3.7 DESIGN ACCOUNTING FOR BYPASS

3.7.1 The OGS device shall be specified to achieve the TSS removal performance and water quality objectives without washout of previously captured pollutants. The OGS device shall also have sufficient hydraulic conveyance capacity to convey the peak storm event, in accordance with hydraulic conditions per the Engineer of Record. To ensure this is achieved, there are two design options with associated requirements:

3.7.1.1 The OGS device shall be placed **off-line** with an upstream diversion structure (typically in an upstream manhole) that only allows the water quality volume to be diverted to the OGS device, and excessive flows diverted downstream around the OGS device to prevent high flow washout of pollutants previously captured. This design typically incorporates a triangular layout including an upstream bypass manhole with an appropriately engineered weir wall, the OGS device, and a downstream junction manhole, which is connected to both the OGS device and bypass structure. In this case with an external bypass required, the OGS device manufacturer must provide calculations and designs for all structures, piping and any other required material applicable to the proper functioning of the system, stamped by a Professional Engineer.

3.7.1.2 Alternatively, OGS devices in compliance with Section 3.6 shall be acceptable for an **on-line** design configuration, thereby eliminating the requirement for an upstream bypass manhole and downstream junction manhole.

3.7.2 The OGS device shall also have sufficient hydraulic conveyance capacity to convey the peak storm event, in accordance with hydraulic conditions per the Engineer of Record. If an alternate OGS device is proposed, supporting documentation shall be submitted that demonstrates equal or better hydraulic conveyance capacity as compared to the OGS device specified herein. This documentation shall be signed and sealed by a local registered Professional Engineer. All costs associated with preparing and certifying this documentation shall be born solely by the Contractor.

3.8 LIGHT LIQUID RE-ENTRAINMENT SIMULATION TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of completed third-party Light Liquid Re-entrainment Simulation Testing in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators**, with results reported within the Canadian ETV or ISO 14034 ETV verification. This re-entrainment testing is conducted with the device pre-loaded with low density polyethylene (LDPE) plastic beads as a surrogate for light liquids such as oil and fuel. Testing is conducted on the same OGS unit tested for sediment removal to assess whether light liquids captured after a spill are effectively retained at high flow rates.

3.8.1 For an OGS device to be an acceptable stormwater treatment device on a site where vehicular traffic occurs and the potential for an oil or fuel spill exists, the OGS device must have reported verified performance results of greater than 99% cumulative retention of LDPE plastic beads for the five specified surface loading rates (ranging 200 L/min/$m^2$ to 2600 L/min/$m^2$) in accordance with the Light Liquid Re-entrainment Simulation Testing within the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators**. However, an OGS device shall not be allowed if the Light Liquid Re-entrainment Simulation Testing was performed with screening components within the OGS device that are effective at retaining the LDPE plastic beads, but would not be expected to retain light liquids such as oil and fuel.

3.9 PETROLEUM HYDROCARBONS AND FLOATABLES STORAGE CAPACITY

Petroleum hydrocarbons and floatables storage capacity in the OGS device shall be a minimum 50 gallons (189 Liters), or more as specified.

3.9.1 The OGS device shall have gasketed precast concrete joints that are watertight, and oil resistant and meet the design criteria according to ASTM C-443 to provide safe oil and other hydrocarbon materials storage and ground water protection. Mastic sealants or butyl tape/rope alone are not an acceptable alternative.

3.10 SURFACE LOADING RATE SCALING OF DIFFERENT MODEL SIZES

The reference device for scaling shall be an OGS device that has been third-party tested in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators**. Other model sizes of the tested device shall only be scaled such that the claimed TSS removal efficiency of the scaled device shall be no greater than the TSS removal efficiency of the tested device at identical **surface loading rates** (flow rate divided by settling surface area). The depth of other model sizes of the tested device shall be scaled in accordance with the depth scaling provisions within Section 6.0 of the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators**.

3.10.1 The Peclet Number and volumetric scaling are not approved methods for scaling OGS devices.

**PART 4 – INSPECTION & MAINTENANCE**

The OGS manufacturer shall provide an Owner's Manual upon request.
Maintenance shall be performed by a professional service provider who has experience in cleaning OGS devices and has been trained and certified in applicable health and safety practices, including confined space entry procedures.

4.1 A Quality Assurance Plan that provides inspection for a minimum of 5 years shall be included with the OGS stormwater quality device, and written into the Environmental Compliance Approval (ECA) or the appropriate State/Provincial or local approval document.

4.2 OGS device inspection shall include determination of sediment depth and presence of petroleum hydrocarbons below the insert. Inspection shall be easily conducted from finished grade through a frame and cover of at least 22 inch (560 mm) in diameter.

4.3 Inspection and pollutant removal shall be conducted periodically. For routine maintenance cleaning activities, pollutant removal shall typically utilize a truck equipped with vacuum apparatus, and shall be easily conducted from finished grade through a frame and cover of at least 22-inches (560 mm) in diameter.

4.4 Diameter of the maintenance access opening to the lower chamber and sump shall be scaled consistently across all model sizes, and shall be 1/3 the inside diameter of the OGS structure, or larger.

4.5 No confined space entry shall be required for routine inspection and maintenance cleaning activities.

4.6 For OGS model sizes of diameter 72 inches (1828 mm) and greater, the access opening to the OGS device's lower chamber and sump shall be large enough to allow a maintenance worker to enter the lower chamber to facilitate non-routine maintenance cleaning activities and repairs, as needed.

4.7 The orifice-containing component (i.e. drop pipe, duct, chute, etc.) of the OGS device used to control flow rate into the lower chamber shall be removable from the insert to facilitate cleaning, repair, or replacement of the orifice-containing component, as needed.

## PART 5 – EXECUTION

### 5.1 PRECAST CONCRETE INSTALLATION

The installation of the precast concrete OGS stormwater quality treatment device shall conform to ASTM C 891, ASTM C 478, ASTM C 443, CAN/CSA-A257.4-14, CAN/CSA-A257.4-14, CAN/CSA-S6-00 and all highway, State/Provincial, or local specifications for the construction of manholes. Selected sections of a general specification that are applicable are summarized below. The Contractor shall furnish all labor, equipment and materials necessary to offload, assemble as needed the OGS internal components as specified in the Shop Drawings.

### 5.2 EXCAVATION

5.2.1 Excavation for the installation of the OGS stormwater quality treatment device shall conform to highway, State/Provincial or local specifications. Topsoil that is removed during the excavation for the OGS stormwater quality treatment device shall be stockpiled in designated areas and not be mixed with subsoil or other materials. Topsoil stockpiles and the general site preparation for the installation of the OGS stormwater quality device shall conform to highway, State/Provincial or local specifications.

5.2.2 The OGS device shall not be installed on frozen ground. Excavation shall extend a minimum of 12 inch (300 mm) from the precast concrete surfaces plus an allowance for shoring and bracing where required. If the bottom of the excavation provides an unsuitable foundation additional excavation may be required.

5.2.3 In areas with a high water table, continuous dewatering shall be provided to ensure that the excavation is stable and free of water.

### 5.3 BACKFILLING

Backfill material shall conform to highway, State/Provincial or local specifications. Backfill material shall be placed in uniform layers not exceeding 12 inches (300 mm) in depth and compacted to highway, State/Provincial or local specifications.

### 5.4 OGS WATER QUALITY DEVICE CONSTRUCTION SEQUENCE

5.4.1 The precast concrete OGS stormwater quality treatment device is installed and leveled in sections in the following sequence:

- aggregate base
- base slab, or base
- riser section(s) (if required)
- riser section w/ pre-installed fiberglass insert
- upper riser section(s)
- internal OGS device components
- connect inlet and outlet pipes
- riser section, top slab and/or transition (if required)
- frame and access cover

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

5.4.3 Adjustment of the OGS 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 shall be repaired or replaced as necessary. Once the OGS stormwater quality treatment device has been constructed, any lift holes must be plugged with mortar.

5.5 <u>DROP PIPE AND OIL INSPECTION PIPE</u>

Once the upper precast concrete riser has been attached to the lower precast concrete riser section, the OGS device Drop Pipe and Oil Inspection Pipe must be attached, and watertight sealed to the fiberglass insert using Sikaflex 1a. Installation instructions and required materials shall be provided by the OGS manufacturer.

5.6 <u>INLET AND OUTLET PIPES</u>

Inlet and outlet pipes shall be securely set using grout or approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight. Non-secure inlets and outlets will result in improper performance.

5.7 <u>FRAME AND COVER OR FRAME AND GRATE INSTALLATION</u>

Precast concrete adjustment units shall be installed to set the frame and cover/grate at the required elevation. The adjustment units shall be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover/grate should be set in a full bed of mortar at the elevation specified.

5.7.1 A minimum of one cover, at least 22-inch (560 mm) in diameter, shall be clearly embossed with the OGS device brand or product name to properly identify this asset's purpose is for stormwater quality treatment.

PROPOSED

THREE STOREY RESIDENTIAL APARTMENT BUILDING SITE

PART OF LOT B

CONCESSION 11

GEOGRAPHIC TOWNSHIP OF CUMBERLAND

1670 TENTH LINE ROAD

CITY OF OTTAWA

# APPENDIX E

# DETAILED CALCULATIONS

# FOR FIVE (5)-YEAR AND 100-YEAR

# AVAILABLE STORAGE VOLUME

## AVAILABLE STORAGE VOLUME CALCULATIONS

**Five (5)-Year Event**

**Roof Storage at Flat Roof Building and Asphalt Parking Lot**

Roof Area 1 to Roof Area 4 will be used for storm-water detention. Each roof area will be drained by one (1) controlled drain. Each roof drain will have a maximum release rate of 15.0 U.S.gal./min. or 0.95 L/s under a head of 150mm. Thus from the flat roof area of this building the controlled flow off-site is 3.80 L/s (4 × 0.95 L/s). Therefore, the (4) roof drains specified is the Watts model Adjustable Accutrol Weir (Model No. RD-100A-ADJ) with 1/4 opening as specified. Refer to Dwg. No. 825-8 SWM-1 for details. As for the remainder of the site, the proposed asphalt parking lot surface area within this site will be designed to provide stormwater detention in order to control the allowable site release rate of 19.84 L/s by incorporating a specified ICD in CB/MH#1.

Roof Storage Area No. 1 (NODE #1)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.11\text{m})[83.61 + 4\,(22.60) + 0]}{6}$$
$$V = \frac{(0.11\text{m})(174.01)}{6}$$
$$V = 3.19\ \text{m}^3$$

The available Roof Area 1 storage volume of 3.19 $m^3$ > required five (5)-Year storage volume of 2.51 $m^3$ from Table 1.

Therefore, the ponding depth at the proposed Roof Drain No. 1 location is approximately 0.11 m (110 mm) and the five (5)-Year level is estimated not to reach the roof perimeter of the building.

Roof Storage Area No. 2 (NODE #2)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.11\text{m})[83.61 + 4\,(22.60) + 0]}{6}$$
$$V = \frac{(0.11\text{m})(174.01)}{6}$$
$$V = 3.19\ \text{m}^3$$

The available Roof Area 2 storage volume of 3.19 $m^3$ > required five (5)-year storage volume of 2.49 $m^3$ from Table 2.

Therefore, the ponding depth at the proposed Roof Drain No. 2 location is approximately 0.11 m (110 mm) and the five (5)-year level is estimated not to reach the roof perimeter of the building.

Roof Storage Area No. 3 (NODE #3)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.11\text{m})[83.61 + 4\ (22.60) + 0]}{6}$$
$$V = \frac{(0.11\text{m})(174.01)}{6}$$
$$V = 3.19\ \text{m}^3$$

The available Roof Area 3 storage volume of 3.19 $m^3$ > required five (5)-Year storage volume of 2.49 $m^3$ from Table 3.

Therefore, the ponding depth at the proposed Roof Drain No. 3 location is approximately 0.11 m (110 mm) and the five (5)-Year level is estimated not to reach the roof perimeter of the building.

Roof Storage Area No. 4 (NODE #4)

Available flat roof area for storage = 149.66 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.11\text{m})[79.61 + 4\ (20.52) + 0]}{6}$$
$$V = \frac{(0.11\text{m})(161.69)}{6}$$
$$V = 2.96\ \text{m}^3$$

The available Roof Area 4 storage volume of 2.96 $m^3$ > required five (5)-Year storage volume of 2.55 $m^3$ from Table 4.

Therefore, the ponding depth at the proposed Roof Drain No. 4 location is approximately 0.11 m (110 mm) and the five (5)-Year level is estimated not to reach the roof perimeter of the building.

During the 5-Year event the required rooftop storage volume is estimated at 10.04 $m^3$ (min.) and the available storage volume is 12.53 $m^3$ at the 0.11m ponding depth specified above at each of the 4 drains.

Parking Lot Surface Storage Volume (NODE #5 and NODE #6)

Assume 5-Year H.W.L. = 87.91 m (see attached Proposed Stormwater Management Plan Dwg. No. 825-8 SWM-1 with the ponding limit shown)

- CB/MH#1

Available Storage Volume $= \frac{d}{6}(A_1 + 4A_2 + A_3)$

$$= \frac{0.10}{6}[\,89.74 + 4(22.26) + 0]$$

$$= \frac{0.10}{6}(178.78)$$

$$= 2.98\ m^3$$

- CB/MH#2

Available Storage Volume $= \frac{d}{6}(A_1 + 4A_2 + A_3)$

$$= \frac{0.10}{6}[\,56.50 + 4(13.69) + 0]$$

$$= \frac{0.10}{6}(111.26)$$

$$= 1.85\ m^3$$

Therefore at the estimated 5-Year H.W.L. = 87.91 m, the available parking lot area and underground drainage system volume is estimated at 4.83 $m^3$ > 3.85 $m^3$ from Table No. 5.

Thus the 5-Year available <u>site</u> storage volume (roof + parking lot) is estimated at 17.36 $m^3$ which is greater than the required storage volume of 13.88 $m^3$.

# AVAILABLE STORAGE VOLUME CALCULATIONS

## 100-Year Event

### Roof Storage at Flat Roof Building and Asphalt Parking Lot

Roof Area 1 to Roof Area 4 will be used for storm-water detention. Each roof area will be drained by one (1) controlled drain. Each roof drain will have a maximum release rate of 15.0 U.S.gal./min. or 0.95 L/s under a head of 150mm. Thus for each roof area the controlled flow off-site is 3.80 L/s (4 × 0.95 L/s). Therefore, the (4) roof drains specified is the Watts model Adjustable Accutrol Weir (Model No. RD-100A-ADJ) with 1/4 opening as specified. Refer to Dwg. No. 825-8 SWM-1 for details. As for the remainder of the site, the proposed oversized underground drainage pipes and structures within this site will be designed to provide stormwater detention in order to control the allowable site release rate of 19.84 L/s.

Roof Storage Area No. 1 (NODE #1)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.15\text{m})[156.04 + 4\,(38.01) + 0]}{6}$$

$$V = \frac{(0.15\text{m})(308.08)}{6}$$

$$V = 7.70 \text{ m}^3$$

The available Roof Area 1 storage volume of 7.70 $m^3$ > required 100-Year storage volume of 5.94 $m^3$ from Table 6.

Therefore, the ponding depth at the proposed Roof Drain No. 1 location is approximately 0.15 m (150 mm) and 0mm above the roof perimeter surface. Therefore, it is recommended that roof scuppers be installed at an elevation to match the top of roof drain elevation for emergency overflow purposes in case of blockage from debris build up at the roof drains as per City's requirements.

Roof Storage Area No. 2 (NODE #2)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.15\text{m})[156.04 + 4\,(38.01) + 0]}{6}$$

$$V = \frac{(0.15\text{m})(308.08)}{6}$$

$$V = 7.70 \text{ m}^3$$

The available Roof Area 2 storage volume of 7.70 $m^3$ > required five 100-Year storage volume of 5.91 $m^3$ from Table 7.

Therefore, the ponding depth at the proposed Roof Drain No. 2 location is approximately 0.15 m (150 mm) and 0mm above the roof perimeter surface. Therefore, it is recommended that roof scuppers be installed at an elevation to match the top of roof drain elevation for emergency overflow purposes in case of blockage from debris build up at the roof drains as per City's requirements.

Roof Storage Area No. 3 (NODE #3)

Available flat roof area for storage = 156.04 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.15\text{m})[156.04 + 4\,(38.01) + 0]}{6}$$

$$V = \frac{(0.15\text{m})(308.08)}{6}$$

$$V = 7.70\ \text{m}^3$$

The available Roof Area 3 storage volume of 7.70 $m^3$ > required 100-Year storage volume of 5.91 $m^3$ from Table 8.

Therefore, the ponding depth at the proposed Roof Drain No. 3 location is approximately 0.15 m (150 mm) and 0mm above the roof perimeter surface. Therefore, it is recommended that roof scuppers be installed at an elevation to match the top of roof drain elevation for emergency overflow purposes in case of blockage from debris build up at the roof drains as per City's requirements.

Roof Storage Area No. 4 (NODE #4)

Available flat roof area for storage = 149.66 $m^2$, @ roof slope of 1.7% (min.). Therefore, the available roof area regulated by one (1) controlled roof drain will store a volume as shown below using the reservoir volume equation.

$$V = \frac{(0.15\text{m})[149.66 + 4\,(38.01) + 0]}{6}$$

$$V = \frac{(0.15\text{m})(301.70)}{6}$$

$$V = 7.54\ \text{m}^3$$

The available Roof Area 4 storage volume of 7.54 $m^3$ > required 100-Year storage volume of 6.08 $m^3$ from Table 9.

Therefore, the ponding depth at the proposed Roof Drain No. 4 location is approximately 0.15 m (150 mm) and 0mm above the roof perimeter surface. Therefore, it is recommended that roof scuppers be installed at an elevation to match the top of roof drain elevation for emergency overflow purposes in case of blockage from debris build up at the roof drains as per City's requirements.

During the 100-Year event the required rooftop storage volume is estimated at 23.84 $m^3$ (min.) and the available storage volume is 30.64 $m^3$ at the 0.15m ponding depth specified above at each of the 4 drains.

Parking Lot Surface Storage Volume (NODE #5 and NODE #6)

Assume 100-Year H.W.L. = 87.96 m (see attached Proposed Stormwater Management Plan Dwg. No. 825-8 SWM-1 with the ponding limit shown)

- CB/MH#1

Available Storage Volume $=\frac{d}{6}(A_1 + 4A_2 + A_3)$

$$=\frac{0.15}{6}[\,205.74 + 4(50.22) + 0]$$

$$=\frac{0.15}{6}(406.62)$$

$$= 10.17\ \mathrm{m}^3$$

- CB/MH#2

Available Storage Volume $=\frac{d}{6}(A_1 + 4A_2 + A_3)$

$$=\frac{0.15}{6}[\,121.89 + 4(30.24) + 0]$$

$$=\frac{0.15}{6}(242.85)$$

$$= 6.07\mathrm{m}^3$$

Therefore at the estimated 100-Year H.W.L. = 87.96 m, the available parking lot area and underground drainage system volume is estimated at 16.24 $m^3$ > 15.92 $m^3$ from Table No. 10.

Thus the 100-Year available site storage volume (roof + parking lot) is estimated at 46.88 $m^3$ which is greater than the required storage volume of 39.76 $m^3$.