Preliminary Stormwater Management Report
Matheson & Rosedale Subdivision

EFI Engineering Inc.

November 26, 2024
Revised: October 24, 2025

MONUMENT
GROUP

Mail: PO Box 1665 Belleville Stn. Main,
Location: 355 Highway 49,
Deseronto, ON K0K1X0
info@monumentge.com

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1 REVISIONS

This revised report was prepared in response to stakeholder comments provided on April 17, 2025. A brief description of the revised sections of the report is summarized below.

1. *Section 2.2 – Background Studies & Plans* – Section updated with revised reference studies and Plans.
2. *Section 4.2 – Pre-development Conditions* – Descriptions of outfalls and inclusion of external catchment areas were updated. As a result of the inclusion of the external areas, the resulting 100-yr pre-development peak flows to each outfall were updated.
3. *Section 5.2.5 – Enhanced Grass Swales* – Verbiage added to acknowledge high groundwater and bedrock elevations should be considered in the detailed design stage.
4. *Section 5.2.7 – Runoff Reduction* – Section added to provide verbiage to support water balance calculations in the Hydrogeological Assessment Report.
5. *Section 5.3.3 – Pond Outlet* – Was been updated to describe the proposed ditching and culvert system from the SWM facility down Matheson Drive directly to Rosedale Creek.

2 BACKGROUND

Monument Group ("Monument") was retained by EFI Engineering Inc. to prepare a Preliminary Stormwater Management Report for the proposed Matheson & Rosedale Subdivision. The 23.54ha site is located in the Township of Montague situated in Lanark County and the jurisdiction of Rideau Valley Conservation Authority (RVCA). The subject property has frontage both on Matheson Drive and Rosedale Road South zoned as Rural Residential. Figure 1 illustrates the site location.

Figure 2-1: Proposed Development Location

The site is generally bound by rural residential properties fronting Matheson Drive and Rosedale Rd S. Various pastures make up the subject property with some small, treed area in the northeast corner. The primary usage of the fields is pasture and wheat harvesting.

The topography of the land slopes towards the adjacent roadways in generally a west to southwest direction. The majority of property drains towards Matheson Drive with relatively low slopes of 0.5% - 1%, while the remainder of the property slopes southwest 3 – 4% towards Rosedale Road S.

The closest watercourse is the main channel of Rosedale Creek which intersects properties south of Rosedale Rd S. The creek is a tributary to the Rideau River System. No other regulated features (i.e., wetland, watercourses) are located within or immediately adjacent to the property.

2.1 Proposed Development

The proposed development will consist of 41 detached single-family homes on 1 acre lots with a minimum frontage of 46m (150ft). Potable water will be drawn at each lot from private wells and disposed of in individual septic systems. The roadway will be a typical rural right-of-way, 20m wide, with a 7m driving surface, and 1.0m wide bottom ditches to convey runoff from the site to a stormwater management (SWM) facility.

2.2 Background Studies & Plans

The following reports in relation to the development were referenced here within:

- *Issued for Draft Plan Approval Drawing Set,* prepared by EFI Engineering, November 22, 2024.
- *Rosedale Subdivision, East of Smiths Falls, ON Geotechnical Subsurface Investigation Report No. 23C258,* prepared by St. Lawrence Testing & Inspection Co. Ltd., October 31, 2023.
- *Hydrogeological Assessment Report – Matheson and Rosedale Subdivision, Part Lot 20 Concession,* Cambium Inc., June 25, 2025.

3 DESIGN OBJECTIVES

The SWM design was prepared to meet the following objectives:

1) **Quantity Control** – The objective is to ensure that post-development peak flows do not exceed the pre-development levels for all storm events up to the 100-yr return period. This ensures that there are no negative impacts due to flooding on any lands downstream of the subject property.

2) **Quality Control** – Quality control target will be to meet a Level 1 Total Suspended Solids (TSS) removal efficiency at the *Enhanced* level of 80% long-term TSS removal rate, in accordance with the *MOE SWM Planning and Design Manual (2003).*

Following Draft Plan approval, objectives 3) and 4) should be detailed further according to the detailed design of the development.

3) **Sediment and Erosion Control Measures** – Prepare a sediment and erosion control plan to control and mitigate release of sediment throughout the construction stage.

4) **Operation and Maintenance Plan** – Recommend an operation and maintenance plan for the proposed SWM devices to be implemented by the Municipality.

4 HYDROLOGY

Monument used PCSWMM version 7.4.1. to model the pre- and post- development conditions of the site. PCSWMM is a powerful modeling platform that provides various hydrologic and hydraulic modelling capabilities. The user can analyze several SWM components such as stormwater and watershed modelling. Rainfall/runoff modelling was used to determine pre- and post-development peak flows and pond storage requirements.

Three models were created; a “Pre-Development” model, “Post-Development Uncontrolled” model, and a “Post-Development Controlled” model to simulate peak runoff rates for all return period events. Catchment areas are delineated based on pre- and post-development drainage patterns. These catchments are then modelled in PCSWMM using the following parameters:

- Area (ha) – Total area of each catchment
- Width (m) – Width of overland flow path; this is automatically determined based on the flow length
- Length (m) – Longest flow path of overland sheet flow
- Slope (%) – Average surface slope; Monument determines the average slope using the 85/10 Method
- Imperv. (%) – Percent of Impervious area
- N Imperv. (unitless) – Manning’s coefficient for impervious area
- N Perv. (unitless) – Manning’s coefficient for pervious area
- Dstore Imperv. (mm) – Depth of depression storage on impervious area; Monument selected 1mm
- Dstore Perv. (mm) – Depth of depression storage on pervious area; Monument selected 5mm
- Zero Imperv (%) – Percent of impervious area with no depression storage; Monument selected 25%
- Curve Number (unitless) – SCS runoff curve number
- Runoff Coefficient (unitless) – used in the Nash IUH method for calculating time of concentration in PCSWMM using the Airport method

A Hydrologic Methodology Section in **Appendix A** provides a description of the hydrologic inputs and parameters used within the PCSWMM models.

4.1 Precipitation Data

Precipitation data was derived for the area using the Ministry of Transportation (MTO) online IDF lookup curve. An excerpt of this data is presented below.

MTO IDF Curve (IDF Data from 2010)
Matheson Drive @ County Road 23

IDF Parameters from MTO IDF Curve Lookup						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Coefficient (A)	20.3	27	31.4	36.9	41	45.1
Exponent (B)	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Uses Equation: Intensity = $A*(T_c\hat{}B)$

Rain Fall Depths from Table 2a						
Duration	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
5 mins	9.6	12.8	14.9	17.5	19.4	21.3
10 mins	11.8	15.7	18.3	21.5	23.9	26.3
15 mins	13.4	17.8	20.7	24.3	27	29.7
30 mins	16.5	21.9	25.5	30	33.3	36.6
60 mins	20.3	27	31.4	36.9	41	45.1
2 hrs	25	33.3	38.7	45.5	50.5	55.6
6 hrs	34.8	46.3	53.8	63.3	70.3	77.3
12 hrs	42.9	57	66.3	78	86.6	95.3
24 hrs	52.8	70.3	81.7	96	106.7	117.4

Figure 4-1: IDF Lookup Curve Precipitation Data

The precipitation data was simulated within each model using the rainfall distribution and durations recommended in the City of Ottawa's October 2012 Storm Design Guidelines identified in **Table 3-1.**

Table 3-1: Rainstorm Distributions & Timesteps

Duration	Timestep
SCS Type II	
24hr	1 hour
12hr	30 minutes
Chicago	
24hr	1 hour
6hr	15 minutes
3hr	5 minutes
AES 30%	
12hr	1 hour

4.2 Pre-development Conditions

Catchment areas were delineated in the "pre-development" conditions to determine outlet locations. Four (4) onsite catchment areas were delineated draining to three separate outlet locations. The location of these outlets and their contributing catchment areas are illustrated in the Pre-development Catchment Area (ST1) drawing in **Appendix B.**

OF1 – is a 600mm diameter corrugated steel pipe (CSP) cross culvert under Rosedale Road S. adjacent to the driveway entrance 876. Thereafter, a small ditch along the north property boundary of house 877 outlets directly into Rosedale Creek. Catchment Areas EX-3 and EX-4 onsite and EXT-1, EXT-2, EXT-3, and EXT-6 offsite contribute to this culvert crossing for a total drainage area of 32.9ha.

EXT-2 and EXT-6 is the contributing area from the field located north of the Matheson and Rosedale intersection that is conveyed under the roadway to OF#1 via two existing 450mm Ø CSP culvert. To accurately model this conveyance route, the cross culvert and roadway geometry was modelled in PCSWMM. The upstream node in the north quadrant was selected as a storage node with an assigned storge volume of 1851m^3 up to the spill elevation of 117.09m, which was derived from Land Information Ontario LiDAR contours. Similarly, the 600mm diameter CSP under Rosedale Road S. was also modelled with the roadway geometry modelled to account for major storms overtopping the right-of-way.

OF2 – Is a 500mm diameter CSP cross culvert under Rosedale Road S. near the existing farm entrance at the south portion of the subject property. Like OF1, the cross-culvert outlets downstream to a small ditch extending between property boundaries of house 795 and 805 to outlet directly into Rosedale Creek.

OF3 – Is the south roadside ditch of Matheson Drive located just north of house 969. The contributing area to this outlet is 22.9ha delineated as Catchment EX-1 and external Catchment Area EXT-4 & EXT-5. Currently, there is no confirmed outlet for this roadside ditch as it appears that runoff is blocked by the roadway which results in the development land storing the volume of water and presumably infiltrates into the underlying soil overtime.

The model was simulated using the storm distributions and timesteps described in the Precipitation Section above for the 100-yr return period event. Results of this simulation at each outfall are provided in **Table 3-2.**

Table 3-2: 100-yr Pre-development Peak Flows

Duration	OF1*	OF2	OF3
SCS Type II (m^3/s)			
24hr	0.874	0.244	1.071
12hr	0.458	0.146	0.615
Chicago (m^3/s)			
24hr	0.754	0.208	0.908
6hr	0.511	0.169	0.707
3hr	0.437	0.144	0.607
AES – 30% (m^3/s)			
12hr	0.537	0.111	0.545

4.3 Post-Development Conditions

The Post-development Catchment Area Drawing is provided in **Appendix B** and illustrates the drainage patterns that reflect proposed grading for the development. Majority of the development will drain to the west side of the site through the developers privately owned land at house 987 and into the south ditch of Matheson Drive S. This main outlet location was selected based on the following conditions:

1) Matheson Drive ditch south of House 969 offers a connection point to the municipal right of way and thereafter Rosedale Creek.
2) Lowest elevation of the subject property that will allow for the greatest area captured in the post-development conditions that will minimize earthworks.

The imperviousness of the site was calculated to reflect the increase in hardened surfaces (i.e. roofs, roads and driveways). Based on the concept plan, the average house size is 240m2 (2580sq.ft) including the garage. An impervious area buffer was also added to consider larger house footprints and outbuildings such as detached garages and sheds that may be considered at full build out. This impervious buffer was calculated by adding an additional 95m2 (~1000sq.ft) to the percentage house in each catchment. Overall, the total imperviousness was calculated to be 14% for the total development. **Table 3-3** shows a comparison of the pre-development peak flows and post-uncontrolled peak flows for all the storm distributions under the 100-yr return period event. Hydrologic values used as inputs for the catchments in the “Post-development” models are provided in **Appendix C.**

Table 3-3: 100-yr Peak flows for the Pre- and Post-Development Uncontrolled Conditions

Duration	OF1 (m^3/s)		OF2 (m^3/s)		OF3 (m^3/s)	
	Pre	Post Uncontrolled	Pre	Post Uncontrolled	Pre	Post Uncontrolled
SCS Type II						
24hr	0.874	1.998	0.244	0.142	0.701	0.372
12hr	0.458	1.172	0.146	0.117	0.402	0.215
Chicago						
24hr	0.754	1.679	0.209	0.124	0.593	0.315
6hr	0.511	1.303	0.169	0.156	0.461	0.248
3hr	0.437	1.093	0.144	0.133	0.395	0.213
AES 30%						
12hr	0.537	1.104	0.111	0.051	0.362	0.190

As illustrated in **Table 3-3**, the uncontrolled peak flows at OF2 and OF3 are less than the pre-development flows. This is a result of the contributing area being reduced in the proposed conditions. Therefore, quantity control measures are not required for these outfalls. However, peak flows for OF1 are greater than the pre and **therefore, quantity control is required**.

As identified above, evidence of a culvert was not found at OF-3 leaving a portion of the development lands and the external lands to the north without an outlet. Since the quantity control is not required at this outlet location, it is recommended that two options be explored at time of detailed design to establish a outlet that can be supported by the Municipality:

Option 1) – Install a new 675mm HDPE Smooth-Walled Culvert Crossing as illustrated on the Post-development Catchment Area Drawing.

Option 2) – Divert runoff from the roadway ditch southwest down Matheson Drive directly to Rosedale Creek.

Since Option 2 does not relay on permission from adjacent property owners it will be selected as the preferred option as discussed further in the conveyance section of this report.

To determine the storage requirements at OF1, a storage node was added to the post-development model to formulate the "Post-development Controlled" Model. Iteratively, the storage node was tested to determine the 100-yr storm that generated the greatest storage requirement. **Table 3-4** below identifies the storage requirements determined for each storm.

Table 3-4: Storage Requirements for each Return Period Event

Duration	Storage (m^3)
SCS Type II	
24hr	6870
12hr	4672
Chicago	
24hr	6171
6hr	5312
3hr	4217
AES 30%	
12hr	6387

Based on the results, it was determined that the SCS Type II 24hr storm requires the greatest volume of storage. **Therefore, the 24hr SCS Type II storm distribution was selected as the design storm for sizing the quantity control facility**.

4.4 Target Peak Flows

The 2-100yr SCS Type II storm was run in the pre-development PCSWMM model to determine the controlled target flows at OF1 for post-development conditions as illustrated in **Table 3-5.**

Table 3-5: Pre-Development Flows at OF1 For 2-100yr Return Period

Duration	OF1 (m^3/s)
2-yr	0.232
5-yr	0.368
10-yr	0.472
25-yr	0.634
50-yr	0.754
100-yr	0.874

5 STORMWATER MANAGEMENT

5.1 Quantity Control

An extended detention wet pond is proposed to meet both the quality and quantity control objective as outlined in **Section 2.0.** The basin will be in the west corner of the site as illustrated on the Conceptual SWM facility provided in **Appendix D.**

5.1.1 Detention Basin Design

The facility has been designed to have two cells – a forebay and main cell. A permanent pool is incorporated to provide7 quality treatment and limit the need for slope through the facility. The elevation of the facility has been set below the inlet of the ditches from the roadway to ensure active storage does not occupy the roadside ditches in the event of the 100-yr storm. The proposed basin has the following design specifications:

- 1.0m Permanent Pool depth
- 0.90m Active storage (inc. extended detention)
- 0.30m freeboard
- 3H:1V side slopes from bottom of pond to permanent pool
- 4H:1V side slopes from permanent pool to top of berm
- Forebay Berm
- Two stage outlet structure
- Emergency spillway
- 3.0m maintenance path
- 4 length to 1 width ratio
- 6m minimum bottom width

Elevations within the facility are as follows:

Bottom Pond Elevation	= 118.15 m
Top Permanent Pool	= 119.15 m
Top Active Storage	= 120.45 m
Top of Berm	= 120.75 m

The above design will provide a total active storage volume of 7656m^3 from the bottom of the pond to freeboard (0.30 m below top of berm). The storage node in the Post-development Controlled Model was supplied with a stage-discharge curve which was extracted from 0.10m intervals from Civil3D based on the conceptual layout in **Appendix D**. These incremental areas were also used to develop the stage-storage-discharge relationship provided in **Appendix E**. The stage-discharge relationship is then supplied to PCSWMM and assigned to an outlet curve from the storage node to the outlet channel downstream.

5.1.2 Outlet Structure

A 1500mm concrete maintenance hole is proposed as the main outlet structure. The inlet to the structure will be a 675mm diameter concrete pipe. Within this structure a concrete wall will contain two control outlets. Outlet 1 is a 150mm diameter orifice set at the invert of the permanent pool of 119.15m. Outlet 2 a 400mm wide concrete broad-crested weir set to an invert elevation of 119.85m.

An emergency spillway will also be constructed in the top of the berm set to the active storage elevation of 120.45m. The spillway is trapezoidal in shape with a 5000mm bottom width, 3H:1V side slopes and 300mm depth. The purpose of the spillway is to act an emergency conveyance route to ensure the banks are not washed out in the case Outlets 1 and 2 become blocked or storm events greater than the 100-yr occur. The spillway is designed to safely control the greater of the 100-yr return period event peak flow.

5.1.3 Overview of Detention Facility

Table 4-1 provides a hydraulic overview of the SWM facility under each return period event. The storage volumes show that each storm event is less than the active storage of 7656m^3 provided. The water surface elevations also illustrate that the 2-yr storm event will be conveyed through the orifice structure, while the 5-100-yr events will crest over the weir invert of 119.85m utilizing both outlets.

Table 4-1: Detention Facility Overview Under Each Return Period Event

Duration	Inflow (m^3/s)	Pond Outflow (m^3/s)	Depth (m)	WSEL (m)	Storage (m^3)
2-yr	0.287	0.032	0.49	119.64	2494
5-yr	0.563	0.053	0.76	119.91	4055
10-yr	0.778	0.097	0.87	120.02	4754
25-yr	1.071	0.168	1.00	120.15	5577
50-yr	1.300	0.232	1.09	120.24	6222
100-yr	1.533	0.304	1.19	120.34	6870
WSEL stands for water surface elevation.					

As mentioned in the Conveyance Section below, the pond outlets into a grassed swale and into the roadside ditch of Matheson Drive down to OF#1. Based on the pond outflow and runoff from the external catchment areas EXT-1 & EXT-2, the post-development peak flows compared to the pre-development peak flows are presented in **Table 4-2**. This also illustrates the success of the SWM facility controlling peak flows to ensure the target release rates are met at OF#1. **Therefore, the quantity control objective has been satisfied.**

Table 4-2: Comparison of Controlled Peak Flows to Pre-Development Peak Flows

Duration	OF1 (m^3/s)	
	Pre	Post-controlled
2-yr	0.232	0.114
5-yr	0.368	0.226
10-yr	0.472	0.303
25-yr	0.634	0.418
50-yr	0.754	0.520
100-yr	0.874	0.625

5.2 Quality Control

As outlined in the MOE *Stormwater Management, Planning and Design Manual (2003),* (here after referred too: "*Design Manual*") SWM ponds are suitable quality treatment facilities for drainage areas of 10ha or larger. The proposed development that will be serviced by the SWM pond is approximately 18.75ha in size and will meet the *Enhanced* quality target of 80% long term TSS removal. This includes post-development catchment areas ST1 – ST6. The remaining catchments PR1 – PR4, will be treated using the low impact development (LID) devices such as vegetated filter strips combined with enhanced grass swales.

5.2.1 Quality Storage

Table 3.2 of the *Design Manual* provides permanent pool and extended detention volume requirements for wet ponds serving sites with varying imperviousness levels. Values less than the prescribed imperviousness (i.e., < 35%) can be extrapolated to determine the necessary storage volumes. Based on the Table, a site having an imperviousness of 14% will require 70m^3/ha of water quality storage. From this value, the extended detention shall make up 40m^3/ha. The MOE minimum storage requirements and provided storage volumes for the SWM pond are Provided in **Table 4-3.**

Table 4-3: Wet Pond Storage Requirements for Quality Control

Pond Feature	MOE Requirement (m^3)	Provided (m^3)
Permanent Pool	562	3750
Extended Detention	750	1982
Total Storage	1312	5732

5.2.2 Erosion Control

Increased runoff from changes in land use may result in greater sediment loading to watercourses due to erosion. SWM ponds help sustain a stable fluvial system downstream by reducing the velocity of outflows. Erosion control is applied according to the *Design Manual* requirement of storing the 25mm 4-hour Chicago "quality storm" and extending its detention period to over a 24-hour period. At the Pond's outlet, an orifice attenuates inflow from the 25mm event in order to achieve the detention time within the allotted extended detention volume. The quality storm event was modelled to determine the drawdown time of the pond. **Figure 4-1** below illustrates the drawdown time of the quality event from PCSWMM.

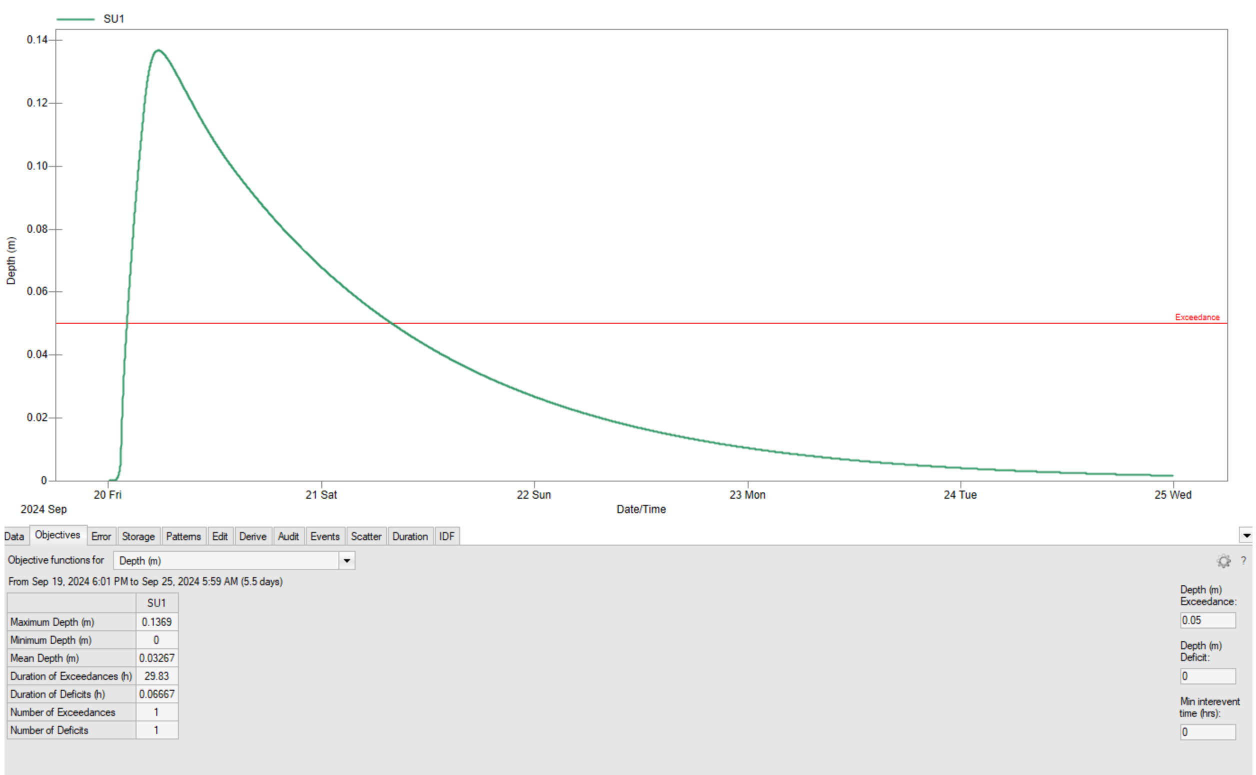


Figure 5-1: Quality Event Drawdown Time

The 25mm event requires a volume of 670m^3 which can be stored within the extended detention of 1982m^3. In addition, after 30 hours the extended detention is drained to a depth of 0.05m at which point the orifice is no longer submerged and the pond is effectively drained. This meets the MOE requirement of a 24-hour minimum drawdown time.

5.2.3 Forebay Sizing

The purpose of a wet pond forebay is to trap larger particles near the inlet of the pond to reduce the frequency of sediment removal within the main cell. The forebay should be deep enough to minimize resuspension of settled material and long enough to ensure that sediment will settle before entering the main body of the pond. The proposed pond forebay has a depth of 1.0m and a length of ~60m. Several forebay design guidelines have been established by the MOE which are outlined as follows.

Forebay length should at a minimum be long enough to facilitate settling of particles. The minimum forebay length to facilitate settling is 40.66m and was calculated using Equation 1:

(Eqn. 1)

$$Distance_{settle} = \sqrt{\frac{rQ_p}{V_s}}$$

where:
r = length-to-width ratio of forebay
Q_p = peak flow rate from the pond during design quality storm (m^3/s)
Vs = settling velocity of particles (m/s)

The forebay should also provide adequate distance to slow the discharge before it enters the main body of the pond. The minimum length of dispersion required to dissipate flows from 100-yr inflow is 24.54m and was calculated using Equation 2:

(Eqn. 2)

$$Distance_{dispersion} = \frac{8Q}{dV_f}$$

where:
Q = inlet flow rate (m^3/s)
d = depth of permanent pool in the forebay (m)
V_f = desired velocity in the forebay (m/s)
**The maximum desired velocity in the forebay recommended by the MOE is 0.5m/s*

The *Design Manual* also provides a target average velocity of 0.15m/s across the entire forebay cross-section to reduce the potential for erosion. The average velocity calculated was 0.15m/s which was calculated using Equation 3:

(Eqn. 3)

$$V_{avg} = \frac{Q}{A}$$

where:
V_{avg} = average velocity across forebay
Q = maximum pipe discharge
A = cross-sectional area of forebay

The forebay design provides adequate room for particle settling while keeping anticipated flow velocity within recommended limits. As such, there is little risk for particle resuspension. **Table 4-4** provides a comparison of required MOE forebay characteristics to the design configuration of the forebay.

Table 4-4: Forebay Sizing Requirements and Provided

Forebay Parameter	Required	Provided
Depth (m)	1.0 < depth > 1.5m	1.0
Settling Length (m)	>40.7	60
Dispersion Length (m)	>24.5	60
Width (m)	>5.1	6
Length: Width Ratio	Minimum 2:1	4:1
Average Velocity (m/s)	< 0.15	0.15

The Design Manual also provides a recommended permanent pool area and volume for the forebay to optimize treatment. The maximum criteria for the forebay area is 33% of the total permanent pool over the total wetted area and a preferred volume of 20%. As per the proposed pond configuration, the forebay area accounts for 30% corresponding 27% volume of the entire pond. As the maximum forebay area criteria is not exceeded, the SWM facility meets these design standards as well.

5.2.4 Vegetated Filter Strip

Vegetated filter strips are gently sloping vegetated areas that treat runoff from adjacent impervious areas in the form of sheet flow. These areas reduce runoff velocities and infiltrate runoff into the underlying soils. Filter strips also provide quality treatment by filtering out sediment and other pollutants such as chloride and sodium from de-icing salts. Vegetated filter strips are ideal as pre-treatment to other LID practices and will be located adjacent to driveways at the lot level and alongside slopes of roadside swales to provide pre-treatment. Design of the vegetated filter strips will include the following design guidance for optimal TSS removal efficiency:

- A maximum flow path length across adjacent impervious surfaces of 25m.
- A minimum flow path length across the filter strip of 5m.
- A slope of 1 to 5%.

Based on available performance studies, pollutant removal efficiencies of vegetated filter strips are highly variable with reported removal efficiencies between 20 to 80% (CVC LID SWM Planning and Design Guide) and therefore should be used in combination with other treatment technologies. Runoff from the vegetated filter strips will be captured in roadside grass swales for secondary treatment.

5.2.5 Enhanced Grass Swales

Grass swales are vegetated open channels that convey and treat stormwater runoff by filtration through vegetation and infiltration through the underlying native soils. Water quality benefits can be enhanced by design features such as modified geometry, check dams, and vegetation.

Grass swales are proposed along the roads to provide quality treatment as well as for conveyance. The Credit Valley Conservation (CVC) 2010 Low Impact Development Stormwater Management Planning and Design Guideline offers the following design guidance for grass swales:

- Bottom width between 0.75 and 3.0m
- Maximum side slopes of 2.5H:1V
- *Maximum velocity of 1m/s for the 4hr 25mm Chicago (Quality) storm
- Length when treating road runoff should be equal to or greater than the contributing roadway length.
- **Pre-treatment with vegetated filter strips

<u>Notes:</u>

*The CVC SWM Planning Design Guideline states that under the desired conditions, an enhanced grass swale’s TSS removal efficiency is greater when the swale maintains a velocity target of 0.50m/s under the quality storm event.
**Pre-treatment for side slope between the edge of pavement and toe of swale are accredited as pre-treatment for road surfaces.

As illustrated on the typical road cross-section, the proposed grassed swales will have a bottom width of 1.0m, a channel depth of 0.66m and side slopes of 3:1.

The PCSWMM "Post-Development" model was simulated under the quality event as mentioned under **Section 4.2.2**. The largest peak runoff was calculated from Catchment PR2 of 0.02m^3/s. Manning's Open Channel Flow Equation was manipulated to determine the corresponding velocity and flow depth. Based on a maximum longitudinal slope of 2% and ditch configuration for the typical road cross section, the channel flow depth would be 0.04m with a velocity of 0.446m/s. Therefore, a TSS Removal efficiency of 70% can be assigned, which is less than the 76% TSS removal efficiency recorded for these swale types in optimum configurations.

It is noted from the geotechnical investigation that high ground water and bedrock elevations are present onsite. Through detailed design, consideration of these features should be considered in the design of the LID devices.

5.2.6 Total TSS Removal

The total volume of precipitation to be treated is determined by applying the 25mm, 4hr storm to the area of driveways and roadways within each catchment. Runoff from rooftops is considered clean and does not require treatment given it is discharged from roof leaders to the pervious surfaces. PR4 also does not require quality treatment as this is the backslope of the pond berm only.

TSS removal efficiencies are based on median removal rates from available performance studies provided in the CVC LID SWM Planning and Design Guide. The proposed treatment train approach for runoff from driveways and roadways is summarised in **Table 4-5** below.

Table 4-5: Treatment Train Approach for Driveways & Roadways

Quality Treatment Train			
Catchment	**Treatment Type**	**LID**	**Assumed TSS Removal Rate**
PR1 – PR3	Pre-treatment (Roadway)	Vegetated Filter Strip	30%
PR1 – PR3	Pre-treatment (Driveway)	Vegetated Filter Strip	50%
PR1 – PR3	Final treatment	Enhanced Grass Swale	70%

Note: TSS removal rates of vegetated filter strips treating roadways are assumed to be less than for driveways because filter strips treating roadways are located between the edge of pavement and toe of the roadside grass swales and thus, they have a comparably shorter flow path and greater slope. Therefore, to be conservative a 30% removal efficiency is assigned for vegetated filter strips.

The quality treatment matrix below, identifies the performance of the LIDs each catchment. As can be seen, the TSS removal rate provided by each LID is applied to the volume of remaining "untreated" runoff after each treatment device.

Volume Untreated	Volume Treated

<table>
<tr><td colspan="6">PR1 & PR3</td></tr>
<tr><td colspan="6">56m^3 Precipitation (2,260 m^2 x 0.025m)</td></tr>
<tr><td colspan="3">Driveway (669m^2)</td><td colspan="3">Roadway (1591m^2)</td></tr>
<tr><td colspan="3">16m^3</td><td colspan="3">40m^3</td></tr>
<tr><td colspan="3">Vegetated Buffer Strip 50% TSS Removal</td><td colspan="3">Vegetated Filter Strip 30% TSS Removal</td></tr>
<tr><td colspan="2">8m^3</td><td>8m^3</td><td>12m^3</td><td colspan="2">28m^3</td></tr>
<tr><td colspan="3">Enhanced Grass Swale 70% TSS Removal</td><td colspan="3">Enhanced Grass Swale 70% TSS Removal</td></tr>
<tr><td>2m^3</td><td colspan="2">14m^3</td><td colspan="2">32m^3</td><td>8m^3</td></tr>
</table>

<table>
<tr><td colspan="6">PR2</td></tr>
<tr><td colspan="6">32m^3 Precipitation (1,288m^2 x 0.025m)</td></tr>
<tr><td colspan="3">Driveway (439 m^2)</td><td colspan="3">Roadway (848 m^2)</td></tr>
<tr><td colspan="3">11m^3</td><td colspan="3">21m^3</td></tr>
<tr><td colspan="3">Vegetated Buffer Strip 50% TSS Removal</td><td colspan="3">Vegetated Filter Strip 30% TSS Removal</td></tr>
<tr><td colspan="2">5.5m^3</td><td>5.5m^3</td><td>6m^3</td><td colspan="2">15m^3</td></tr>
<tr><td colspan="3">Enhanced Grass Swale 70% TSS Removal</td><td colspan="3">Grass Swale 70% TSS Removal</td></tr>
<tr><td>1.5m^3</td><td colspan="2">9.5m^3</td><td colspan="2">17m^3</td><td>4m^3</td></tr>
</table>

Figure 5-2: Quality Treatment Matrix

The proposed treatment train approach provides a total TSS removal of 82% for all three catchment areas. **Therefore, the SWM facility accompanied by the LID measures provides the necessary quality control across the site to satisfy the quality control objective outlined in Section 2.**

5.2.7 Runoff Reduction

LIDs can also be accounted for assistance in promoting infiltration for the purpose of diluting nitrate loading from onsite septic systems. As these features are natural interaction between runoff and vegetated devices, infiltration can be directly correlated to runoff reduction.

A detailed water balance calculation was conducted Cambium in the Hydrogeological Assessment Report (June 2025), which accounted for infiltration from impervious to pervious surfaces. Specifically, runoff from rooftops to grassed areas adjacent to house and runoff entering the roadside ditches.

As identified in the CVC SWM Planning and Design Guide, rooftop disconnect will promote runoff reduction of up to 50% based on the HSG type B, provided a minimum flow path length of 2 – 5m are available. Which correlates to the function of the vegetated filter strips described above. Similar to vegetated filter strips, grassed swales also provide a runoff reduction and have been observed to conservatively provide a reduction rate of 50% for HSG Type A & B soils. However, a 25% reduction was recommended as longitudinal slopes for the proposed development do not fall within the optimal design as discussed above. Additional measures such as rock check dams can also be reviewed at the time of detailed design to aid in further infiltration if necessary.

5.3 Conveyance

This Section provides an overview of the storm water conveyance through the development. Minor flows (<5-yr storm) will be conveyed within the roadside ditches and culvert network. Major flows (10- to 100-yr storms) will be safely conveyed within the right-of-way to the intended outfalls.

5.3.1 Roadside Ditches

As per the typical road cross-section these ditches will have a 1.0m bottom, 3:1 side slopes, an overall depth of 0.66m, and longitudinal slopes ranging from 1% to 2%. At the minimum slope, the ditches will contain a full flow capacity 2.949m^3/s which far exceeds the 100-yr inlet flow of 1.53m^3/s into the SWM facility.

5.3.2 Culvert Sizing

Culverts at specified locations within the site will need to be sized to ensure they contain adequate flow capacity. This analysis will be completed at time of detailed design.

5.3.3 Pond Outlet

The County and Municipality have identified that the existing ditch on Rosedale Road S to Outfall #1 has experienced flooding in the past. Additionally, there is no drainage easement from Rosedale Road S for the existing swale than convenes through private property outletting into Rosedale Creek. Although this is an existing condition and realistically the SWM facility is controlling post-development flows down to the pre-development conditions before leaving the site, there is a concern that a greater volume of water will be conveyed through the existing swale and cannot be relied upon without a formal easement established.

As there is no option to obtain an easement, The County and Municipality have requested that flow be diverted to Rosedale Creek via Matheson Drive right of way approximately 290m southwest of Rosedale Road S. This will not only provide a sufficient outlet for the subdivision but also improve the overall drainage conditions for the existing right-of-way and fronting residents.

Also, since there is an opportunity to convey runoff from OF#3 down Matheson Drive the proposed ditch and culvert sizing will take into account the additional flow from this outfall location. The layout modelled within PCSWMM is illustrated in the image below for reference.

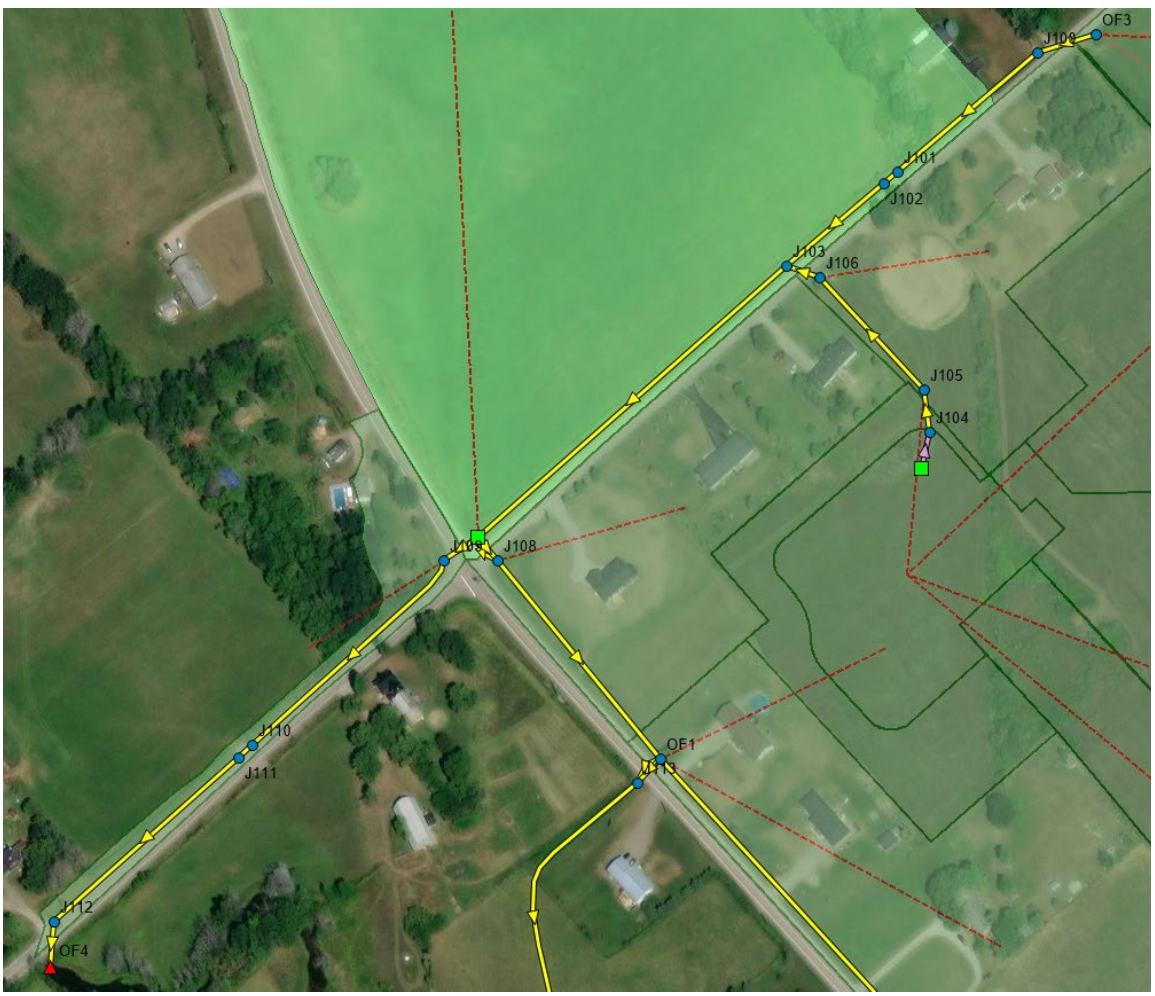


Figure 5-3: PCSWMM Model - Plan View

As illustrated in the image above, the runoff would be convey from OF#3 and the SWM facility to the north side of Matheson Drive down to Rosedale Road S. From there, the existing culvert crossing across Rosedale Road S in the northwest quadrant would be reversed and ditching completed in the north ditch line down to Rosedale Creek. Preliminary plan and profile drawings prepared by EFI illustrating this proposed works are provided in Appendix ##.

As this will be the major overland flow route from the SWM facility, the proposed ditching and culvert system will need to be sized to convey the following two scenarios:

1) The 100-yr controlled outflow from the SWM facility and all other external contributing areas with a minimum of 0.15m freeboard.

2) Convey the 100-yr uncontrolled flow from the SWM and all other external contributing areas in the event the SWM facility outlet becomes obstructed.

The resulting 100-yr peak flow at specified junctions illustrated in the image above for Scenario 1 are listed below. In this scenario, the pond is controlling the 100-yr flow from the development lands and releasing it as calculated in the Quantity Section above.

OF#3	– 0.388m^3/s
J105	– 0.304m^3/s
J106	– 0.330m^3/s
J103	– 0.606m^3/s
J107	– 1.388m^3/s
J108	– 0.304m^3/s
J109	– 1.078m^3/s
J110 - J112	– 1.078m^3/s

The 100-yr peak flow at specified junctions under Scenario 2 are listed below. Under this scenario the model is set to by-passes the SWM facility and route runoff from the pond block directly to Junction 105.

OF#3	– 0.388m^3/s
J105	– 1.533m^3/s
J106	– 1.627m^3/s
J103	– 1.976m^3/s
J107	– 2.521m^3/s
J108	– 0.376m^3/s

J109 – 1.642m^3/s
J110 - J112 – 1.642m^3/s

Preliminary sizes for each link (conduit) are modelled as such.

Scenario 1:

OF#3 to J100 – Culvert
Preliminary Size: 675mm Ø HDPE Smooth-walled Culvert
Slope = 0.50%
Flow = 0.388m^3/s
Head Upstream = 0.49m
Flow capacity used = 64%

J100 to J101 – Swale
Preliminary Size: V-shaped; 0.60m depth; 3H:1V side slopes; slope = 0.50%
Flow =0.388m^3/s
Flow Depth = 0.45m
Freeboard = 0.15m

J101 to J102 – Culvert
Preliminary Size: 450mm Ø HDPE Smooth-walled Culvert; 2.1% slope
Flow = 0.388m^3/s
Head Upstream = 0.44m
Flow capacity used = 71%

J102 to J103 – Swale
Preliminary Size: V-shaped; 0.60m depth; 3H:1V side slopes; slope = 3.95%
Flow =0.388m^3/s
Flow Depth = 0.29m
Freeboard = 0.31m

J105 to J106 – Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 3H:1V side slopes; slope = 1.0%
Flow =0.304m^3/s
Flow Depth = 0.22m
Freeboard = 0.78m

J106 to J103 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled Culvert; 1.0% slope
Flow = 0.331m^3/s
Head Upstream = 0.36m
Flow capacity used = 14%

J103 to J107 – Swale
Preliminary Size: V-ditch 1.0 m depth; 2H:1V side slopes; slope = 0.675%
Flow =0.605m^3/s
Flow Depth = 0.48m
Freeboard = 0.52m

J107 to J109 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.067m^3/s
Head Upstream = 0.65m
Flow capacity used = 44%

J109 to J110 - Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 1.5H:1V side slopes; slope= 0.58%
Flow =1.078m^3/s
Flow Depth = 0.82m
Freeboard = 0.18m

J110 to J111 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.078m^3/s
Head Upstream = 0.64m
Flow capacity used = 57%

J111 to J112 – Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 1.5H:1V side slopes; slope= 1.01%
Flow =1.078m^3/s
Flow Depth = 0.74m
Freeboard = 0.26m

J112 to J113 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.078m^3/s
Head Upstream = 0.29m
Flow capacity used = 19%

<u>Scenario #2</u>

As the second scenario only impacts the pond outfall, the results from OF#3 to J103 will remain the same.

J105 to J106 – Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 3H:1V side slopes; slope = 1.0%
Flow =0.1.533m^3/s
Flow Depth = 0.49m
Freeboard = 0.51m

J106 to J103 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled Culvert; 1.0% slope
Flow = 1.609m^3/s
Head Upstream = 0.97m
Flow capacity used = 91%

J103 to J107 – Swale
Preliminary Size: V-ditch 1.0 m depth; 2H:1V side slopes; slope = 0.675%
Flow =1.995m^3/s
Flow Depth = 0.67m
Freeboard = 0.33m

J107 to J109 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.626m^3/s
Head Upstream = 0.88m
Flow capacity used = 80%

J109 to J110 - Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 1.5H:1V side slopes; slope= 0.58%
Flow =1.642m^3/s

Flow Depth = 0.69m
Freeboard = 0.31m

J110 to J111 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.642m^3/s
Head Upstream = 0.83m
Flow capacity used = 69%

J111 to J112 – Swale
Preliminary Size: Trapezoid; 1.0m depth; 1.0m bottom width; 1.5H:1V side slopes; slope= 1.01%
Flow =1.642m^3/s
Flow Depth = 0.60m
Freeboard = 0.40m

J112 to J113 – Culvert
Preliminary Size: 1000mm Ø HDPE Smooth-walled culvert; 1.0% slope
Flow = 1.642m^3/s
Head Upstream = 0.37m
Flow capacity used = 29%

Under each scenario, each link in the system conveys the 100-yr controlled and uncontrolled flow from the SWM facility. A snippet of each scenario from J105 to J113 of the PCSWMM model is provided in the image below. It is also noted that the hydraulic grade line (HGL) at J107 in the post-development conditions is 0.09m below the HGL at this location in the pre-development conditions. **Therefore, the proposed ditching and culvert system for the outlet of the SWM facility will safely convey the required flow under each scenario**.

Note: The preliminary drawings are to illustrate the feasibility of the option. Various other options (i.e., piped system) or a combination of options can be explored at time of detailed design.

Scenario 1

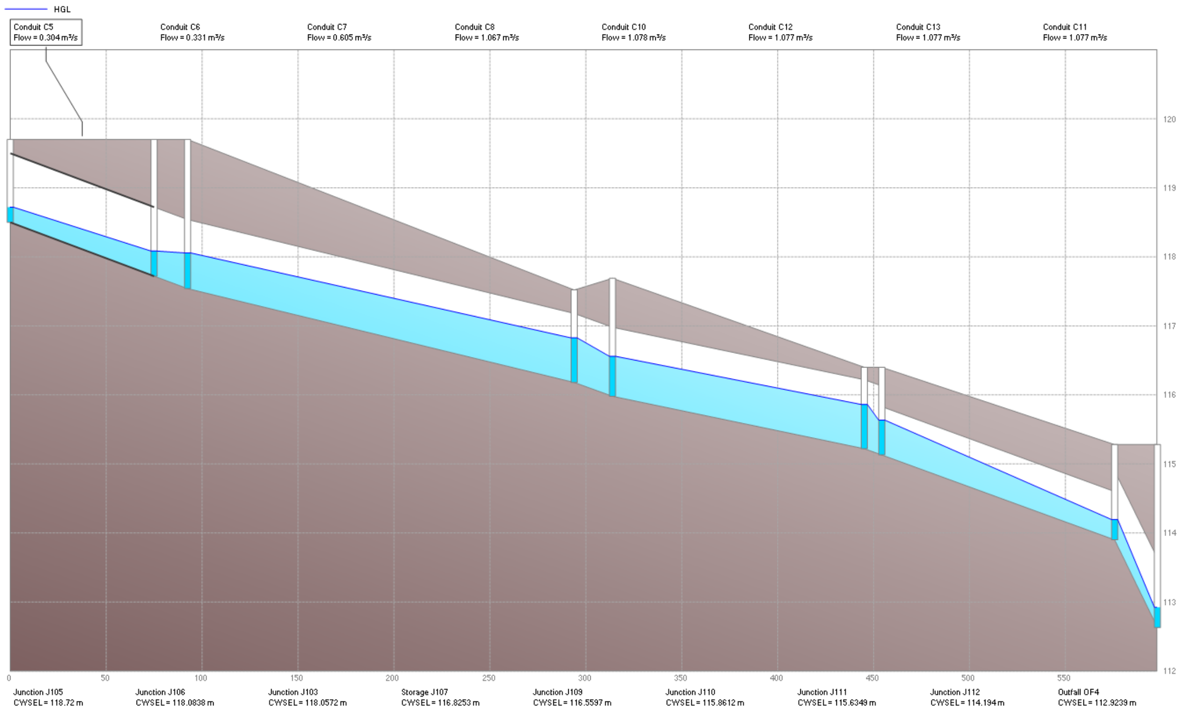


Figure 5-4:Scenario 1 - PCSWMM Model - Profile View

Scenario 2

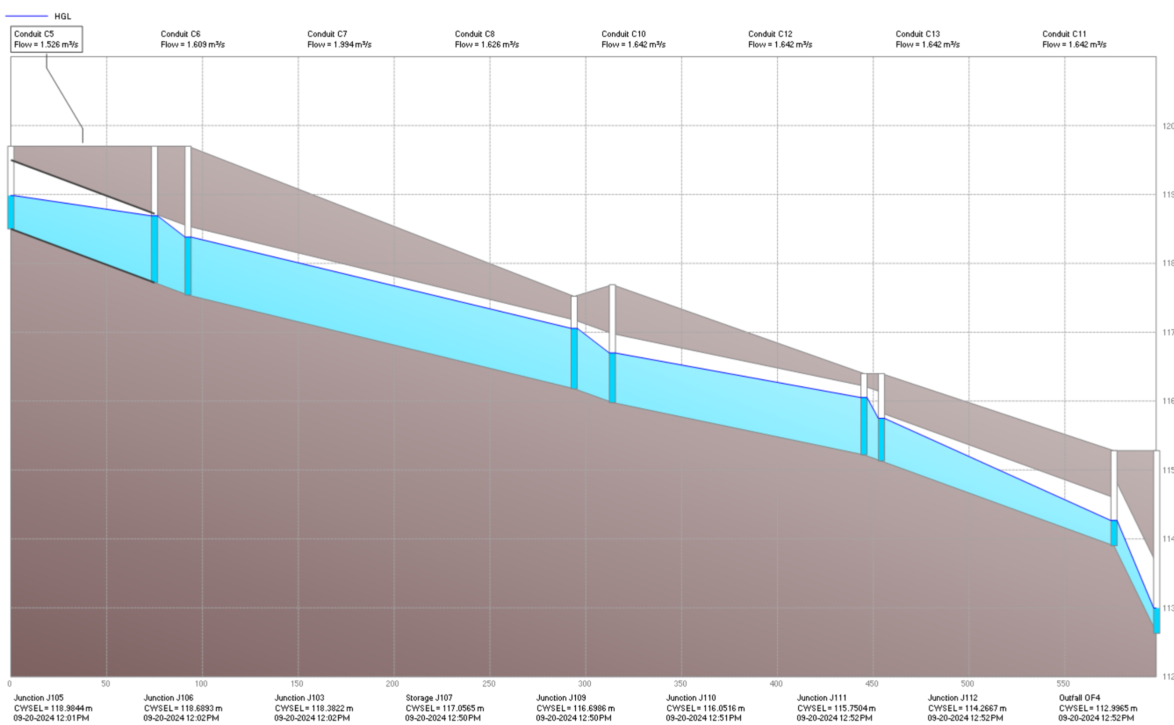


Figure 5-5: Scenario 2 - PCSWMM Model - Profile View

6 EROSION AND SEDIMENT CONTROL

Erosion and sediment control measures are required during the construction phase to limit the amount of sediment leaving the site. Areas that are disturbed due to construction activity create potential for washing out of exposed areas resulting in blockages in downstream infrastructure such as culverts and ditches. To mitigate this exposure, erosion and sediment control measures should be installed and maintained throughout construction and until vegetation has been re-established.

These measures should be tailored to the final design of the development in order ensure proper protection. Therefore, these details will be prepared at the time of detailed design.

7 MAINTENANCE & OPERATIONS

The transfer of a SWM facility from the developer to the future owner shifts responsibility of the long-term operation and maintenance of the facility to the new owner. Similar to other municipal infrastructure such as sewers and treatment plants, the effective long-term operation of a SWM facility relies upon effective and consistent maintenance practices. Maintenance of the quality control facilities should be in accordance with Toronto Region Conservation Authority's *Inspection and Maintenance Guide for Stormwater Management Ponds and Constructed Wetlands (2018)* guide.

These measures should be tailored to the final design of the facility in order ensure proper maintenance & Operation. Therefore, these details will be prepared at the time of detailed design.

8 CONCLUSION

Monument Group ("Monument") was retained by EFI Engineering Inc. to prepare a Preliminary Stormwater Management Report for the proposed Matheson & Rosedale Subdivision. The 23.54ha site is located in the Township of Montague situated in Lanark County and the jurisdiction of Rideau Valley Conservation Authority (RVCA). The intent of the stormwater management (SWM) design is to satisfy the following design objectives:

1) **Quantity Control** – The objective is to ensure that post-development peak flows do not exceed the pre-development levels for all storm events up to the 100-yr return period. This ensures that there are no negative impacts due to flooding on any lands downstream of the subject property.

2) **Quality Control** – Quality control target will be to meet a Level 1 Total Suspended Solids (TSS) removal efficiency at the *Enhanced* level of 80% long-term TSS removal rate, in accordance with the *MOE SWM Planning and Design Manual (2003).*

The proposed development will consist of 41 detached single-family homes on 1 acre lots with a minimum frontage of 46m (150ft). Potable water will be drawn at each lot from private wells and disposed of in individual septic systems. The roadway will be a typical rural right-of-way, 20m wide, with a 7m driving surface, and 1.0m wide bottom ditches to convey runoff from the site to a stormwater management (SWM) facility.

The proposed SWM facility was selected as an extended detention wet pond to provide both quality and quantity control. The pond has been designed to provide Level 1 TSS removal efficiency equipped with a forebay and permanent pool to treat the 25mm – 4hr Chicago storm. For those areas of the development that cannot feasibly drain to the SWM facility, low impact devices such as vegetated buffers and enhanced grassed swales will provide Level 1 TSS removal efficiency as well. An active storage volume of 7656m^3 within the pond will provide quantity control, sized to attenuate the 2 – 100yr return period events in the post-development conditions. The active storage will overcontrol flows, to ensure that pre-development release rates are met at each outlet location from the site. **Therefore, satisfying the design objectives identified above.**

Additional objectives such as a sediment and erosion control plan and owners' operation and maintenance manual for the SWM facility will be prepared at detailed design.

Reviewed by:

Submitted by:

Patrick Quinn, P.Eng.
Project Engineer

9 REFERENCES

Ministry of Environment (2003). *Stormwater Management Planning and Design Manual.*

Ministry of Transportation of Ontario (1997) *MTO Drainage Management Manual*

DRAFT – Stormwater Management Facility Design Guidelines & Standards (2012) *City of Ottawa Environmental Services Department, Surface Water Management Services Branch, Stormwater & Municipal Drainage Unit*

Sewer Design Guidelines, Second Edition (2012) SDG002 *City of Ottawa*

Thornthwaite, C.W., and Mather, J.R., The Water Balance. *The Water Balance.* Publications in Climatology, vol 10, No. 3 Drexel Institute of Technology, Laboratory of Climatology, Centreton, New Jersey, 1055.

Toronto and Region Conservation Authority, Credit Valley Conservation Authority (2010) *Low Impact Development Stormwater Management Planning and Design Guide.*

Appendix A – Hydrology Methodology

This Section provides explanation on the hydrologic parameter selected for the each PCSWMM models.

Soil Conditions

Soil conditions for rural and undeveloped sites in Ontario are catalogued in the AG Ontario Soil Survey Complex by the Ministry of Agriculture, Food, and Rural Affairs and was used to determine the Hydrologic Soils Group (HSG) for the area. The site is comprised of Type B Farmington Grenville Loam; moderately drained fine sandy loam and silt loam overlying loam till with moderate infiltration capacity.

Curve Number

Curve Number is a system developed by the U.S. Soil Conservation Service for estimating the volume of rainfall runoff in agricultural lands. Curve numbers are heavily dependent on the hydrologic soils group and land use type. A weighted curve number was calculated for the site by assigning curve numbers by land area based on Design Chart 1.09 in the MTO Drainage Management Manual (2008) for B soils with pastures and woodland forests.

Runoff Coefficient

Monument determined an arithmetic weighted runoff coefficient, R.C., to account for different land uses and soil types. Runoff coefficients were assigned based on Chart 1.07 in the MTO Drainage Management Manual (2008) HSG B (Loam) reflective of the overland slope. The runoff coefficient is used for calculating the time of concentration using the NashIUH method for the pre-development model.

Imperviousness

Imperviousness is a measure of surface hardness in a catchment area. Increased surface hardening due to building of water-resistant structures such as roads and rooftops reduces the infiltration capacity of ground surfaces and increases runoff. Imperviousness can be expressed as a percentage of hardened surfaces in the entire catchment area. The site imperviousness in post-development conditions was calculated for each catchment area.

Slope and Flow Length

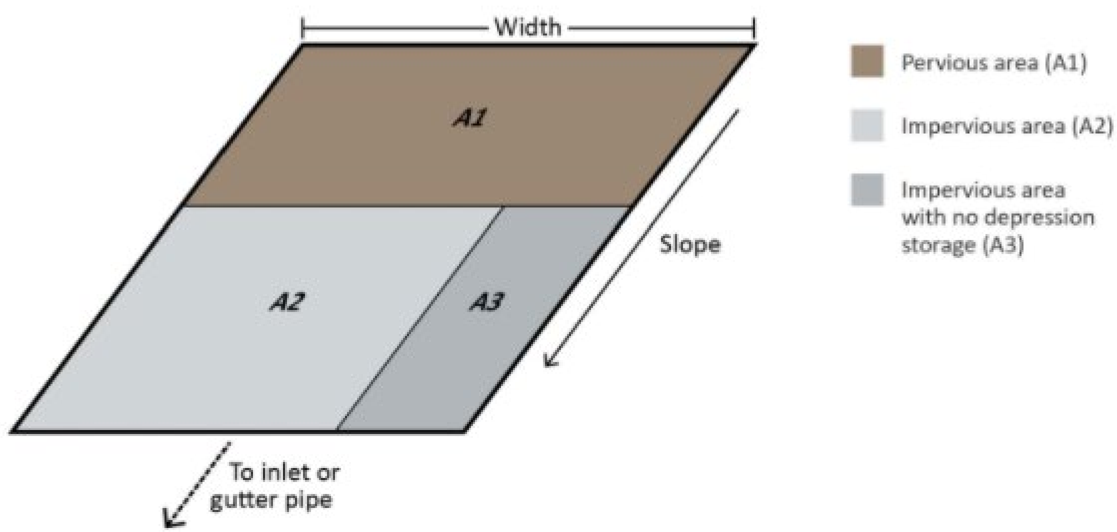
Slope and flow length influence the time it takes for runoff to reach an outlet. Flow lengths for each catchment were measured as the longest overland flow route to either the channelized route or outfall of the contributing area. Slope for each catchment was determined using the 85/10 method which is generally recommended for normal use.

Time of Concentration

The time of concentration describes the time required for water travel from the most remote point in a catchment area to the outfall. The time of concentration is used for the NashIUH method in PCSWMM and is calculated internally. The Airport method was selected in PCSWMM for calculating the time of concentration since the runoff coefficient for each catchment area is below 0.4.

Subarea Routing

Post-development catchments are divided into pervious and impervious subareas. Surface runoff can infiltrate in pervious surfaces represented by the curve number, whereas impervious areas will directly runoff. Overland flow is then generated from each subarea by approximating them as non-linear reservoirs (see image below).



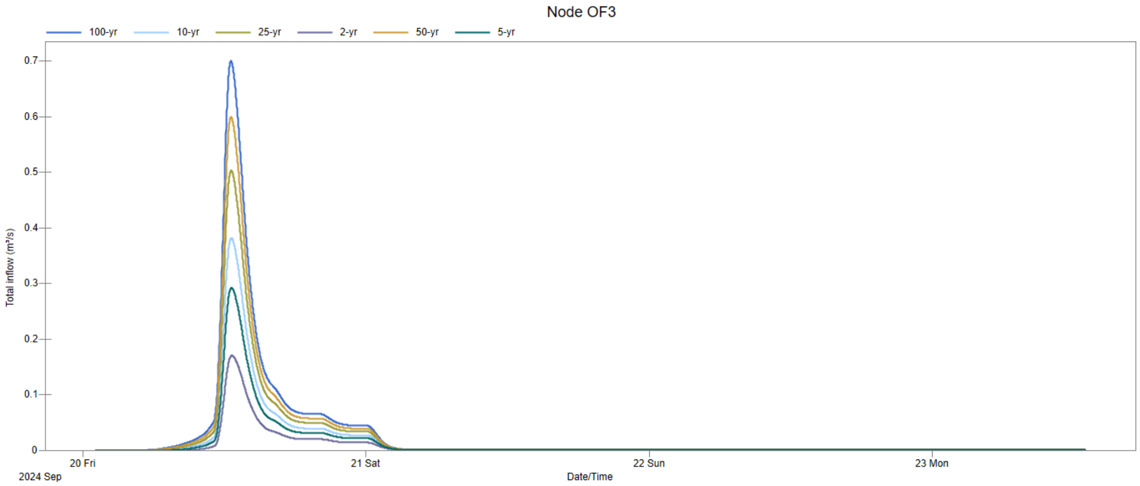
PCSWMM Support Excerpt describing Subarea Routing
(https://support.chiwater.com/80217/subarea-routing)

Typically, the overland flow from each subarea is independently routed to the outlet, however, PCSWMM allows the user to further subdivide runoff between subareas using the subarea routing tool. This creates internal routing between pervious and impervious surfaces. (e.g. roofs onto lawn surfaces). There are three selection options for the subarea routing tool:

- **IMPERV:** some percentage of the runoff from the pervious area is directed to the impervious area and then to outlet,
- **PERV:** some percentage of the runoff from the impervious area is directed on the pervious area and then to outlet,
- **OUTLET**: runoff from each subarea is routed directly to the outlet. (e.g. no subarea routing)

Monument selected to use the PERV command for the on-site catchments to route runoff from rooftops onto the grassed areas due to roof leader disconnection. This is represented by expressing a percent routed for each catchment (i.e. area of rooftop over the impervious area). The percent routed for each catchment are provided in the Hydrologic Input Parameters in **Appendix C**.

OF3 Pre-development



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for Total inflow (m³/s)

From Sep 19, 2024 8:49 PM to Sep 23, 2024 5:11 PM (3.85 days)

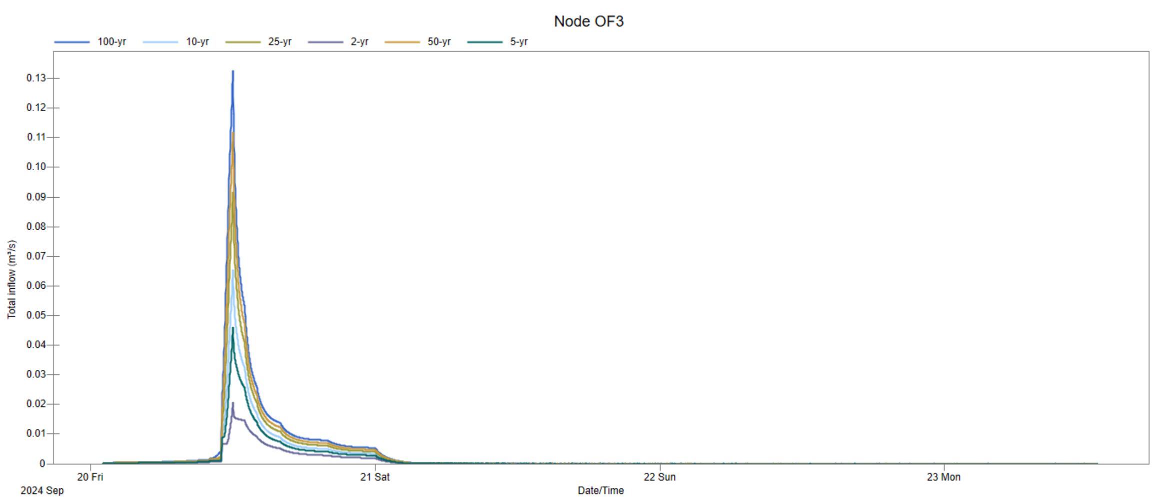
	OF3 100-yr	OF3 10-yr	OF3 25-yr	OF3 2-yr	OF3 50-yr	OF3 5-yr
Maximum Total inflow (m³/s)	0.7009	0.3817	0.5033	0.1704	0.5998	0.2921
Minimum Total inflow (m³/s)	0	0	0	0	0	0
Mean Total inflow (m³/s)	0.02807	0.01564	0.0204	0.007219	0.02416	0.01209
Duration of Exceedances (h)	83.98	83.98	83.98	83.98	83.98	83.98
Duration of Deficits (h)	4.517	5.9	5.25	7.817	4.85	6.55
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	1	1	1	1	1	1
Volume of Exceedances (m³)	8486	4727	6169	2182	7306	3656
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	8486	4727	6169	2182	7306	3656

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Min interevent time (hrs): 0

OF3 Post-development



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for Total inflow (m³/s)

From Sep 19, 2024 8:49 PM to Sep 23, 2024 5:11 PM (3.85 days)

	OF3 100-yr	OF3 10-yr	OF3 25-yr	OF3 2-yr	OF3 50-yr	OF3 5-yr
Maximum Total inflow (m³/s)	0.1327	0.06543	0.09149	0.02056	0.1118	0.04612
Minimum Total inflow (m³/s)	0	0	0	0	0	0
Mean Total inflow (m³/s)	0.003449	0.001956	0.002534	0.0009033	0.002985	0.001519
Duration of Exceedances (h)	83.98	83.98	83.98	83.98	83.98	83.98
Duration of Deficits (h)	52.68	52.77	52.48	52.55	52.77	52.62
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	85	91	92	104	92	98
Volume of Exceedances (m³)	1043	591.2	766	273.1	902.3	459.3
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	1043	591.2	766	273.1	902.3	459.3

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Min interevent time (hrs): 0