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CONCEPTUAL STORMWATER MANAGEMENT REPORT

Proposed Residential Subdivision
819 County Road 23
Merrickville-Wolford, Ontario

Prepared For:
2873633 Ontario Inc.
210 Prescott Street, Unit 1
Kemptville, Ontario
K0G 1J0

PROJECT #: 210816

Distribution:
2873633 Ontario Inc.
Township of Merrickville-Wolford
United Counties of Leeds and Grenville
Kollaard Associates Inc.

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1.0 INTRODUCTION

Kollaard Associates was retained by 2873633 Ontario Inc. to complete a conceptual stormwater management design in support of an application for draft plan conditions for a proposed residential subdivision in the Township of Merrickville-Wolford. The proposed subdivision will occupy an irregularly shaped parcel of land located on the northwest side of county road 23, approximately 500m southwest of Davis Road. The site is currently occupied by a single-family dwelling and detached garage.

For the purpose of this report, project north lies in a direction perpendicular to County Road 23, at the southern limit of the site. The site is located along the north side of County Road 23 and along the east and south side of the Rideau River. For the remainder of this report County Road 23 will be considered to be on an east-west axis and the proposed Street 1 will be oriented on a north-south axis.

The site will be accessed by means of a proposed roadway extending from County Road 23. The proposed development areas will be divided into 29 single family residential estate lots, inclusive of the existing single family dwelling. The existing detached garage will also be converted to a single family dwelling as a part of the proposed development and will occupy one of the 29 lots. There will be two blocks reserved for stormwater management, as well as two blocks for parkland. Each lot will be serviced with a private on-site septic system and well.

This report is intended to present the results of a preliminary stormwater management design in support of the application for draft plan conditions. The report will summarize the stormwater management (SWM) design requirements and proposed works that will address stormwater flows arising from the site under post-development conditions on a conceptual level sufficient to ensure that stormwater management meeting the criteria for the site is feasible. The report is not intended to represent a final or completely comprehensive design for the proposed development.

The report is to be read in conjunction with the stormwater management system design presented in the Kollaard Civil Drawings: 210816-PRE, 210816-POST, 210816-GR1, 210816-GR2, 210816-ESC, 210816-PP1, and 210816-PP2.

1.1 Background

The proposed development has a total area of about 24.0 hectares and is occupied by a single family dwelling and detached garage. The area consists of a mixture of maintained grassland and landscaping adjacent the dwelling. The remainder of the site consists of brush mixed with young to mature forest growth.

The site was split into two portions divided by an undesignated wetland located about 240m north of County Road 23. The south portion of the site consists of the area between County Road 23 and the undesignated wetland. The south portion of the development is currently overgrown and has been characterized as unmaintained apple orchard, brush and woodlands and drains by overland flow to the north towards the undesignated wetland. The existing single-family dwelling and detached garage are located on the north portion of the development. The area immediately adjacent to the house and garage were characterized as maintained grassland and landscaping. The area immediately south of the dwelling and garage drain via overland flow to the undesignated wetland, and the remaining portions drain by overland flow to the west towards the Rideau River. The northern approximately 12.9 hectares of property is characterized as brush and woodland and drains by overland flow toward the Rideau River to the west. The Rideau River changes direction at the northernmost portion of the site from a north-south axis to an east-west axis, abutting the north property line of the site.

The unevaluated wetland, which intersects the site approximately 240m north of the south property line is labeled as a non-PSW per available GIS mapping from the Rideau Valley Conservation Authority (RVCA). This wetland meets the criteria which subjects it to regulation by RVCA under Ontario Regulation 148/06 as there is an intermitant watercourse flowing into and out of the wetland area. The aforementioned unnamed water course is a tributary to the Rideau River.

It is acknowledged that the RVCA has determined that the unevaluated wetland is subject to O.Reg 148/06 and that development and site alteration within 30m of the unevaluated wetland is regulated by the RVCA.

1.2 Summary

The proposed stormwater management plan for the subdivision will make use of best management practices in combination with a treatment train approach to meet the requirements for stormwater management on the site. Stormwater from the majority of the developed portion of each lot including the driveways, majority of the dwelling footprint and the adjacent roadway will be collect by means of the roadside ditches and or swales and will be directed towards one of the two stormwater management areas. The stormwater will be detained in these areas and released at a controlled rate to either the Rideau River or to the existing undesignated wetland.

It is intended to provide vegetative filtration and sedimentation for removal of total suspended solids within the roadside ditches and swales upstream of the respective stormwater management area. Further settlement will take place within the stormwater management area prior to discharge to either the wetland or the Rideau River. There is no intent to alter the permanent water level or function of the wetland.

1.3 Governing Authorities and Standards

This Stormwater Management Report has been prepared to present the design information to satisfy conditions set by the following authorities:

- Township of Merrickville-Wolford (Township)
- Rideau Valley Conservation Authority (RVCA)
- United Counties of Leeds and Grenville
- Ministry of Environment, Conservation and Parks (MECP) formerly Ministry of Environment (MOE)

2.0 STORMWATER MANAGEMENT DESIGN

The subject lands are within the Township of Merrickville-Wolford. Stormwater management guidelines are set out by the Ministry of the Environment, *Stormwater Planning and Design Manual* (2003). Stormwater management criteria were established based on the nature and location of the development and on the nature of the receiving watercourse for the stormwater discharge from the site.

2.1 Design Criteria

2.1.1 Quantity Control Criteria

- Post-development peak runoff rates will be restricted to less than or equal to the pre-development peak runoff rate for all storms up to and including the 100 year storm event.
- Minimize surface runoff volumes through infiltration techniques where possible;
- Accommodate offsite drainage where encountered;
- Control onsite stormwater storage and flow so as to not affect lands adjacent to or downstream from the development site.

2.1.2 Quality Control Criteria

- Mitigate sediment transportation and erosion through the use of best management practices.
- Provide an enhanced level of protection corresponding to 80% long-term suspended solids removal for the controlled runoff leaving the developed portion of the subdivision.
- Provide a level of treatment by means of vegetative filtration for the uncontrolled runoff leaving the developed portions of the subdivision

2.2 Guidelines, Manuals and Reports

The following guidelines and manuals were utilized in the creation of the stormwater management design and the preparation of this report.

Ottawa Sewer Design Guidelines
City of Ottawa, November 2012.
(CofO Guidelines)

Stormwater Management Planning and Design Manual
Ministry of the Environment (now known as MECP), March 2003
(SWMP Design Manual)

Visual OTTHYMO V2.0: Reference Manual
Greenland International Consulting Inc., July 2002

MTO Drainage Management Manual
Ontario Ministry of Transportation

Part 650 Hydrology National Engineering Handbook – Chapter 15 Time of Concentration
United States Department of Agriculture (USDA Chapter 15)

Urban Hydrology for Small Watersheds Technical Release 55
United States Department of Agriculture (USDA TR55)

2.3 Overview and Best Management Practices

As indicated in the SWMP Design Manual, the recommended strategy for stormwater management is to provide an integrated treatment train approach to water management.

Each element of the treatment train within the development when combined forms the stormwater management facility for the development.

In general, best management practices for stormwater management are divided into three categories: source control, conveyance control and end-of-pipe control. As indicated in the Ministry of Transportation Drainage Management Manual, the priority in applying these BMPs should follow the sequence presented with end of pipe measures applied as the last resort.

The proposed BMPs utilized in the proposed development will include both lot level and general BMPs. Lot level BMPs may include reduced lot grading, direction of roof drainage to the vegetated ground surfaces, reducing directly connected impervious areas, vegetative buffer strips and re-vegetating any surface areas of the lot disturbed during construction as soon as possible. The general BMPs will include reduced swale slopes and increased swale cross sections were possible to reduce flow rates and provide filtration and the removal of sediments.

3.0 PROPOSED HYDROLOGIC MODEL

3.1 Design Storms

Rainfall Intensity-Duration-Frequency curves were obtained using the Ministry of Transportation Ontario IDF Curve lookup tool. The IDF curve and rainfall summary data are included in Appendix A following the text of this report.

The data obtained from the IDF curves for the site was used to generate SCS Type II Design Storms with select durations and return periods up to and including the 100 year storm event. The 25 millimeter 4 hour Chicago storm is considered by the Ontario Ministry of Environment to be the design storm for quality control purposes and was also included. *Table 3-1* summarizes the selected design storms included for analysis

Table 3-1: Design storms considered

Simulation 01.	25mm4hrChicago
Simulation 02.	SCS II 12hr 2yr
Simulation 03.	SCS II 12hr 5yr
Simulation 04.	SCS II 12hr 10yr
Simulation 05.	SCS II 12hr 50yr
Simulation 06.	SCS II 12hr 100yr

3.2 Methodology

The hydrologic modeling software, Visual OTTHYMO (V6.2) was used to assess pre and post-development stormwater conditions at the site.

The pre-development conditions were calculated using the NASHYD watershed command. The post-development conditions were also modeled using the NASHYD watershed command as the impervious ratio for reach post-development catchment area was less than twenty percent.

The NASHYD hydrograph method uses the Nash instantaneous unit hydrograph which is made of a cascade of 'N' linear reservoirs and is used to model rural areas.

Both the pre and post-development conditions were modeled for quantity control purposes using the selected design storms summarized above in *Table 3-1*.

Detailed output data from the above Visual OTTHYMO model is provided in Appendix B, C, and is summarized in Section 4.0 and 5.0 of this report.

3.3 OTTHYMO Storm Analysis Variables

The **NASHYD** command uses the following inputs:

DT – Simulation time step increment (min) – must be shorter than TP
Area – Watershed or catchment area (hectares)
DWF – A constant Dry Weather Flow or Baseflow (m^3/s) assumed to be 0 (doesn't change from pre to post development)
CN – SCS Modified Curve Number
IA – Initial Abstraction (mm)
N – Number of Linear reservoir used for derivation of the Nash Unit Hydrograph
TP – Unit hydrograph time to peak (hr)

3.3.1 Curve Numbers

The NasHyd hydrograph method which uses the SCS loss method for pervious areas was used to model both the pre- and post development conditions of the proposed subdivision. Runoff Curve Numbers (CN) are utilized in the SCS hydrology method. The Curve Number is a function of soil type, ground cover, and antecedent moisture conditions. The soil type was chosen to be Group C, considering the glacial till, and silty clay underlying the topsoil at the site. A tabulation of CN values for both pre and post development conditions is presented in Appendix A. For the purpose of analysis presented in this report the surface cover for undeveloped areas was considered as brush/weeds in fair hydrologic condition and with a

CN values equal to 70. The CN Values for the existing developed area consisted of the gravel driveway, impervious areas (dwelling, roofs, patio areas etc.), and maintained grassed areas. The assigned CN values for the gravel, impervious areas are equal to 89 and 98 respectively. The maintained grassed area was considered as open space in good condition and with a CN value equal to 74. The wetland area was assigned a CN value of 89 for semi-permanent wetland. The CN values were taken from the CofO Guidelines Table 5.9 (2012) and from the United States Department of Agriculture Urban Hydrology for Small Watersheds Technical Release 55 (USDA TR55).

3.3.2 Initial Abstraction and Potential Storage

The initial abstraction includes all losses before runoff begins, and includes water retained in surface depressions, water taken up by vegetation, evaporation, and infiltration. This value is related to characteristics of the soil and the soil cover. Initial abstraction is a function of the potential storage and is generally assumed to be equal to 0.2 S where S is the potential storage.

It is considered that for lower CN values, the relationship IA = 0.2S tends to overestimate the initial abstraction resulting in underestimated peak runoff. As such, suggested guidelines are as follows:

CN ≤ 70 IA = 0.075S
CN > 70 ≤ 80 IA = 0.10S
CN > 80 ≤ 90 IA = 0.15S
CN > 90 IA = 0.2S

The potential storage S is related to the runoff coefficient as follows:

S = (25400/CN) - 254

A calculation of the initial abstraction IA and potential storage S values for both the pre- and post-development conditions are also presented in Appendix A.

3.3.3 Time of Concentration and Time to Peak

The time to peak is typically considered to be two thirds of the time of concentration of a catchment area. The time of concentration of each catchment was determined using the Velocity method. The velocity method assumes that the time of concentration is the sum of travel times for segments along the hydraulically most distant flow path. The segments used in the velocity method may be of three types: sheet flow T_s, shallow concentrated flow T_{sc}, and open channel flow T_c.

Example calculation of time of Concentration for the pre-development catchment PRE-1:

Travel time for Sheet Flow:

The Manning's roughness coefficient for sheet flow was taken as n = 0.40.

$$T_s = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

Where T_s = travel time, h
n = Manning's roughness coefficient sheet flow = 0.40
l = sheet flow length, 100.00 ft
P_2 = 2-year 24-hour rainfall, = 2.07 inches
S = Slope of land surface m/m = 0.058
T_s = 0.29 hours

Sheet flow length is given in feet by:

$$l = \frac{100\sqrt{S}}{n}$$

The sheet flow length given by this equation was given a lower bound of 50 feet and an upper bound of 100 feet.

Travel time for shallow concentrated flow:

The flow velocity used to calculate the time of travel for shallow concentrated flow was determined using Figure 15-4 of Chapter 15 of the USDA handbook. This chart can be used to estimate the velocity when the slope and ground cover are known. The ground cover used in reading Figure 15-4 for the pre-development catchment PRECA-1, subject to shallow concentrated flow, was considered to be roughly in between heavy forest and woodlands. The slope was calculated to be on average 4.2% based on the existing topography. Figure 15-4 indicates a velocity of V= 0.31 ft/s. The distance of shallow concentrated flow is the distance between the point at which sheet flow ended and open channel flow begins or the end of the catchment. Travel time is given by:

$$T_{sc} = \frac{l}{3600\,V}$$

Where T_{sc} = travel time, h
l = distance of shallow concentrated flow = 623 ft
V = average velocity = 0.30 ft/s
T_{sc} = 0.17 hrs

There was no open channel flow considered during pre-development conditions. The open channel flow during post-development conditions was modelled using the Route Channel in OTTHYMO.

The total time of concentration for the pre-development catchment PRE-1 is therefore equal to

Tt = 0.29 + 0.17 = 0.46 hrs.

The time to peak is therefore equal to 0.46 x 2/3 = 0.31 hrs = 18.3 mins.

3.3.4 Manning Coefficients

The Manning Roughness (n) Coefficients for overland flow selected for impervious site areas (MNI) was assumed to be 0.013 based on the CofO Guidelines: Appendix 6-C Manning Coefficient values for street and gutter flow assuming weathered asphalt.

The Manning's roughness coefficient for pervious surfaces (MNP) during post-development conditions was selected to be 0.25 based on sheet flow through good quality grass in the developed pervious areas and 0.4 for the undeveloped pervious areas.

4.0 PRE-DEVELOPMENT STORMWATER ANALYSIS

4.1 Off Site Runoff Conditions and Contributing Areas

As previously mentioned, the site is bound on the west and north sides by the Rideau River. The south property line of the site borders with County Road 23. As such, there is no contributing runoff from the south east or north property line.

Based on available satellite mapping, the property to the east of the subject site consists of a mixture of treed areas, as well as open field/meadow. Upon review of the available topographic survey, the site to the east does not appear to have any existing drainage patterns directing runoff towards the site. As such, it has been assumed that there is no contributing off-site runoff.

4.2 No-Development Areas

The proposed development will incorporate two on-site catchment areas, which are to be left unchanged between pre- and post-development conditions. The first no-development area consists of the above-mentioned wetland, which divides the site into the north portion and south portion. The wetland and associative setbacks will not be altered by the proposed development, and are to be left in a natural state, with exception of the small portion of the proposed road which will cross the wetland. To minimize the impacts on the wetland, the proposed roadway will follow the approximate location of the existing gravel driveway through the wetland. This catchment has been assigned the label ND-WETLAND.

The second no-development area consists of the area immediately adjacent the Rideau River along the west and north portions of the property. The no-development along the Rideau River has been proposed to ensure that the shoreline of the Rideau River is left in a natural state following the development of the subdivision and will remain consistent from pre- to post-development. This catchment has been assigned the label ND-RIVER

4.3 Site Runoff Conditions

The proposed development has a total area of about 24.0 hectares including no-development zones and presently consists of a mixture of a small developed area consisting of a single family dwelling, detached garage and associative grassed and landscaped areas, and the remainder of the site being undeveloped brush and weeds mixed with young to mature woodlands. Under pre-development conditions the site has been modeled with 2 catchments in addition to the two no-development catchments defined in section 4.2 above. PRE-SOUTH and PRE-NORTH catchments were modeled with the NASHYD hydrograph method. A detailed tabulation of parameters assigned to each pre-development catchment can be found in Appendix A

For an illustration of pre-development catchments see Kollaard drawing #210816-PRE.

It is noted that all of the catchment areas in both pre- and post-development conditions discharge either directly or indirectly into the Rideau River. During pre-development conditions, catchment PRE-SOUTH discharges by way of the wetland into the river.

4.4 Pre-Development Runoff

4.4.1 Total Site Pre-development Runoff

Table 4-1 summarizes the total pre-development peak runoff rate and runoff volumes for the above storm events. Appendix B contains detailed output data for the last link in the OttHymo model. The detailed output data for the last link provides a summary of the pre-development outflow from the proposed development

Table 4-1: Pre-Development Peak Runoff Rates and Volumes (Total)

Design Storm Event (Total Catchment Area = 24.03 ha)	**Pre-Development Peak Runoff Rate (m^3/s)**	**Time to Peak (hrs)**	**Runoff Volume (mm)**
01. 25mm4hrChicago	0.056	2.58	2.08
02. SCS II 12hr 2-yr	0.236	6.50	9.23
03. SCS II 12hr 5yr	0.455	6.42	14.96
04. SCS II 12hr 10yr	0.629	6.42	20.20
05. SCS II 12hr 50yr	1.069	6.33	33.05
06. SCS II 12hr 100yr	1.269	6.33	38.82

4.4.2 Pre-development Runoff from No-Development Areas

Pre-development model incorporates two no-development areas, which will be unaffected by the proposed subdivision. As such, runoff rate generated from these two catchments will remain unchanged between pre- and post-development conditions. *Table 4-2* summarizes the peak runoff rate and runoff volumes for the two no-development catchments.

Table 4-2: Pre-Development Peak Runoff Rates and Volumes (ND River and ND Wetland)

Design Storm Event (No-Development Areas = 4.88 ha)	**Pre-Development Peak Runoff Rate (m^3/s)**	**Time to Peak (hrs)**	**Runoff Volume (mm)**
01. 25mm4hrChicago	0.020	2.50	3.55
02. SCS II 12hr 2yr	0.066	6.50	11.34
03. SCS II 12hr 5yr	0.119	6.42	19.25
04. SCS II 12hr 10yr	0.159	6.42	25.22
05. SCS II 12hr 50yr	0.259	6.42	39.43
06. SCS II 12hr 100yr	0.303	6.33	45.70

5.0 POST-DEVELOPMENT STORMWATER ANALYSIS

Preliminary grading for the site establishes elevations for the proposed road and ditches. The road profile has been designed such that runoff generated in controlled catchments will be conveyed by the roadside ditches and will outlet to one of the two stormwater management areas. The runoff from the southern portion of the site will be directed to the stormwater management block located along the south side of the roadway immediately south of the wetland prior to discharging to the existing wetland. Runoff from the north portion of the site will discharge into the stormwater management block located in the northeast corner of the site. In general it is assumed that dwellings will be constructed approximately 40 meters from the road with a grade break that will result in runoff generated on the front roof, driveway, and front lawn travelling by overland flow to the proposed road side ditch. Runoff generated in the rear of the dwellings will travel by overland flow to the rear of the lot where it will either be intercepted by ditches or follow existing drainage patterns.

Total post-development peak runoff rates for design storm events up to and including the 100 year event will be limited to pre-development rates for each storm event.

For a detailed illustration of post-development catchments and flow paths see Kollaard drawing #210816-POST in conjunction with the OTTHYMO Post-Development model schematic in Appendix C.

5.1 Stormwater Conveyance

Under post-development conditions storm water runoff is conveyed by the following mechanisms:

- Sheet Flow;
- Property line swales;
- Easement Swales and;
- Roadside Ditches.

5.1.1 Roadside Ditches

The proposed road will be accompanied by road side ditches throughout the site. Proposed Street 1 will have a shallow trapezoidal or U-shaped ditch along its length. It will have a bottom width of 0.50m in width, feature side slopes of between 2.5H:1V and 3H:1V, and have a centreline of between about 0.50m and 2.0m below the centreline of road. There will be ditches located on both the east and west sides of Proposed Street 1.

Runoff captured in the roadside ditches along the north portion of the site will discharge to the stormwater management facility located in the block along the northeast corner of the site. Runoff captured within the roadside ditch along the south portion of the site will be directed towards the stormwater management facility located south of the western half of the wetland.

Proposed Street 2, which includes the cul-de-sac on the east side of the about mid-point of the site, will have a shallow trapezoidal or U-shaped ditch along its length. It will have a bottom width of 0.50 metres, feature side slopes of approximately 3H:1V, and have a centerline 1m below the centreline of road. Runoff generated in controlled catchments along Street 2 will be conveyed to the south where it will intersect with and discharge into the street side ditches along proposed street 1.

5.2 Quantity Control

5.2.1 Stormwater Storage Swale

As previously indicated, the post-development flow rate will be restricted such that the maximum release rate from the development area of the subdivision will be less than or equal to the pre-development flow rate for each storm event up to and including the 100 year design storm event. Due to the increased impervious area and decreased time of concentration resulting from the proposed development, the post-development unrestricted runoff rates from the site will be much greater than the pre-development runoff rates.

As previously discussed, the existing wetland effectively divides the proposed development into the north and south development areas. Runoff from the controlled areas of the development will be conveyed by the swales and roadside ditches described in section 5.1 above to the proposed stormwater storage swale located within the respect development area. In order to meet the stormwater management criteria for the site with respect to runoff rate, temporary flow detention will be provided by means of restricting the flow rate from each of the stormwater storage swales using an outlet control structure described in the following Section 5.2.2

5.2.1.1 North Swale

The proposed storage swale in the north portion of the site is a flat bottom rectangular swale at the north end of the development east of Street 1. The swale has the following physical characterists:

- An outlet complete with outlet structure at the north end.
- Bottom length of about 175 metres and slope of 0.1 percent.
- Bottom width which is 2 metres for the first about 125 metres the increases to 15 metres at the south end.
- Side slopes of 5 horizontal to 1 vertical.

5.2.1.2 South Swale

The propose storage swale in the south portion of the site consists of a widened section of the roadside ditch at the north end of the south portion of the development. The storage swale is located along the south side of the section of roadway that is oriented parallel to the unevaluated wetland. The south storage swale has the following physical characteristics.

- An outlet complete with outlet structure at the west end.
- Bottom length of about 85 metres and a longitudinal slope of 0.0 percent.
- Bottom width which varies from about 2 metres to 9 metres.
- A subdrain consisting of a 150 mm diameter perforated single wall HDPE tile pipe in a 300 x 300 mm surround of clear stone complete with geotextile fabric around the clearstone.
- Side slopes of 3 horizontal to 1 vertical.
- The bottom has a cross slope which extends from the bottom of the road side slope to the subdrain. The subdrain is at the bottom of the back side slope.
- The subdrain extends to through the outlet structure and outlets through the sand filter into the wetland.

5.2.2 Outlet Control

The outlet from both stormwater storage swales will be controlled by an outlet control structure consisting of an orifice, a V-Notch weir and an overflow channel. The control structures will be placed within a weir wall located at the north end of each proposed stormwater storage swale.

5.2.2.1 Outlet Orifice

The first stage in the outlet control structure in both stormwater storage areas is by an orifice positioned within the weir plate, which will be discussed in further detail in section 5.2.2.2.

The discharge rates through the orifices were calculated using the equation:

$$Q = \frac{0.72\sqrt{2gD^5}}{[(C_w\eta^{1.98})^{-2.14} + (C_d\eta^{0.52})^{-2.14}]^{0.4673}}$$

Which is a calculation that unifies flow through a partially full orifice as weir flow with flow through a fully submerged orifice as orifice flow.

Where Q = flow (cubic meters per second)
C_d = Coefficient of Discharge for a sharp orifice = 0.60
Cw = Coefficient of Discharge for a sharp crested weir = 0.62
D = Orifice diameter (m)
η = y/D
Y = water-head relative to orifice invert (m)
g = acceleration from gravity (9.81 m/s^2)

The orifice in the south stormwater storage area will consist of a 100mm diameter orifice and will have an invert of 90.10m. The orifice in the north stormwater storage area will consist of a 100mm diameter orifice and will have an invert of 93.10 metres.

5.2.2.2 V-Notch Weir

The second portion of the outlet control structure in the respective stormwater storage areas is by a V-notch weir. The cast in place weir wall will have a rectangular cutout above the invert of the swale that will allow an aluminum weir plate to be installed. The weir plate of the southern storage area will have a 60-degree V-notch weir with a notch invert of 90.35m. The weir plate of the north stormwater storage area will have a 40-degree V-notch weir with a notch invert of 93.55m. Both aluminium weir plates will also have the orifice cutouts as discussed above to ensure that both the orifice and the V-notch weir are processed on the same plate. The weir wall construction will be similar in appearance to the illustration in Figure 5-1 on the following page. Since the flow above the V-notch is overflow, the weir plate will not extend above the poured concrete. Overflow of the storage swale will be over top of the weir wall. The bottom and side slopes on both sides of the weir walls should be protected with Rip-Rap and geotextile. The Rip-Rap should conform to the grading requirements for OPSS 1004 G-10 Gabion Stone. The geotextile should consist of a minimum 6 ounce per square yard non-woven fabric. The Rip-Rap and Geotextile should be placed in keeping with OPSS 511 and OPSD 810.010.

Figure 5-1: V-notch Cast-In-place Concrete Weir Wall Illustration

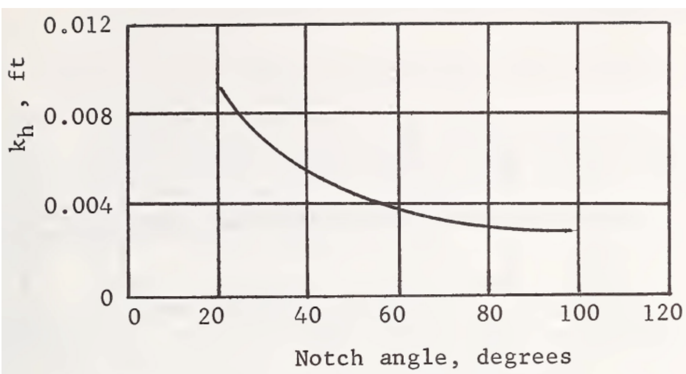
The discharge rate through the V notch weir was calculated using the Kindswater-Shen relationship (Kulin, G and Compton, P., A Guide to Methods and Standards for the Measurement of Water Flow, NBS Special Publication 421 May 1975, Institute of Basic

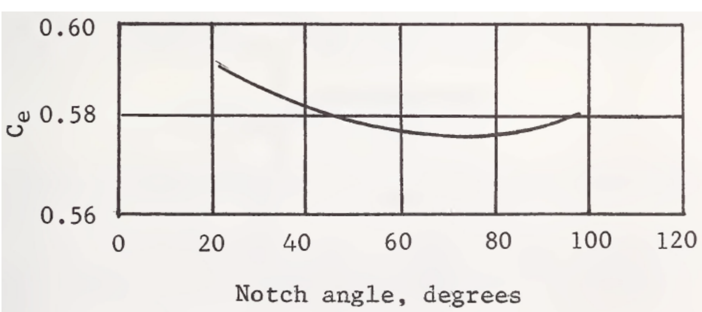
Standards, National Bureau of Standards, Washington, D.C. 20234) as the discharge will be passing over the weir in free flow conditions. The equation is as follows:

$$Q = 4.28\, C_e \tan\left(\frac{\theta}{2}\right) h_{1e}^{2.5}$$

Where:

Q = discharge in cfs
Ce = effective discharge coefficient (see figure copied on following page)
h1 = head on the weir (ft)
h1e = h1 + kh
kh = head correction factor (ft) (see figure copied on following page)
θ = angle of V- notch
It is noted that the head on the weir was converted from metres to feet to facilitate the use of the equation and then the results were converted from cfs to m3/s.





5.2.2.3 Overflow Channel

The third component of the outlet control structure consists of an overflow channel located above the V-notch weir. The rectangular overflow channel will take place above the aluminum weir plate, but lower than the top of the concrete weir wall. The south stormwater storage area will consist of a 0.80m wide overflow channel at 90.65 m. The north stormwater storage area will consist of a 1.00m wide overflow channel at 94.20 m.

5.2.2.4 Total Outlet

The orifice size, notch angle and overflow channel width were determined by iteration for each respective stormwater storage area. In a relatively large storm water storage facility, it is generally assumed that the hydraulic gradient is a function of the water surface elevation only. Since the discharge rate Q is a function of the depth of water-head relative to the weir invert, the discharge rate will be directly related to the water surface elevation relative to the inverts of each contributing component of the outlet control structure. The volume of the storage swale is also directly related to the depth of water or water surface elevation. Since the discharge rate and storage swale volume are both related to the ponding depth, a Discharge vs Storage curve can be developed for each outlet control structure.

5.2.3 Stormwater Storage Swale Storage-Discharge Relationship

As previously discussed, the existing wetland effectively divides the proposed development into the north and south development areas. An independent stormwater storage swale has been designed for each of these development areas. In conjunction with the outlet control structures discussed above to ensure that the quality control criteria is met without overflow

A storage discharge relationship for each of the proposed stormwater storage areas was determined. A detailed worksheet can be seen in Appendix-D. Figure 5-2 and Figure 5-3 below illustrates the storage-discharge relationship that was plotted into the RouteResevoir command (NHYD 40 and NHYD 41) within the OttHymo model to obtain the post-development results summarized in Table 5-1 below.

Figure 5-2: South Storage Swale Storage-Discharge Relationship

	DISCH [m³/s]	STORAGE [ha.m]
1	0.000000	0.000000
2	0.000000	0.002198
3	0.004400	0.006113
4	0.008300	0.011838
5	0.011000	0.019389
6	0.019300	0.028709
7	0.038700	0.039621
8	0.098800	0.051895
9	0.259700	0.065257
10		

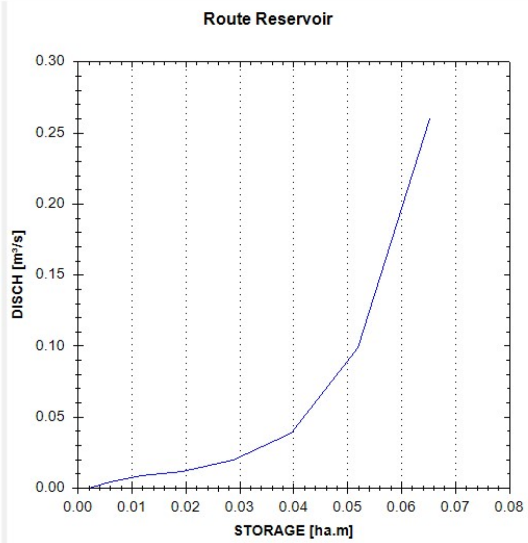
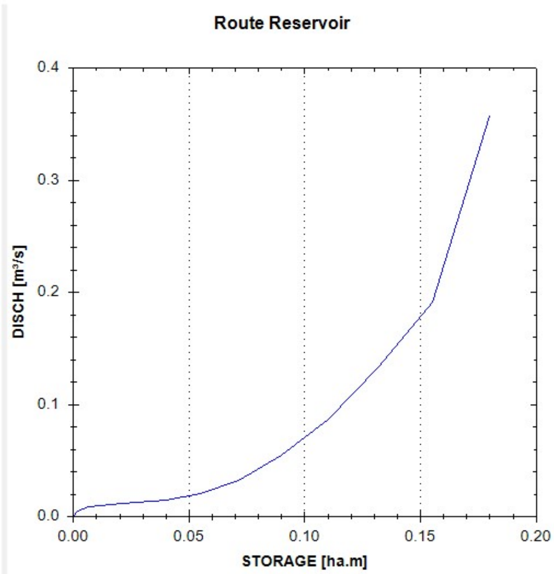


Figure 5-3: North Storage Swale Storage-Discharge Relationship

	DISCH [m³/s]	STORAGE [ha.m]
1	0.000000	0.000000
2	0.004400	0.001831
3	0.008300	0.006718
4	0.010600	0.016075
5	0.012400	0.027246
6	0.014200	0.040237
7	0.019700	0.055056
8	0.032200	0.071712
9	0.054000	0.090211
10	0.086700	0.110563
11	0.131900	0.131826
12	0.191000	0.155474
13	0.357300	0.180121
14		



Discharge from the south storage swale will be directed to the wetland area and will be conveyed by the wetland to the Rideau River. Discharge from the north swale will be conveyed to the Rideau River

5.3 Post-Development Runoff

The proposed development has been divided into controlled and uncontrolled catchments. Generally the uncontrolled catchment areas consist of those from which runoff is allowed to exit the site without restriction to the runoff rate. The controlled areas are those areas from which the runoff is collected and directed to the one of the two stormwater storage facilities on the site. The outlet from the stormwater management facility will be controlled to ensure that the post-development runoff rate does not exceed the pre-development runoff rate.

The controlled and uncontrolled areas are illustrated on drawing # 210816-POST. In general, the controlled areas consist of the proposed roadway, with the exception of the sections through the wetland and immediately north of the wetland, the proposed driveways, front yards and front half of each dwelling.

5.3.1 Total site Post-development Runoff

Table 5-1 summarizes the total post-development peak runoff rates for design storm events. The table also provides a comparison to the pre-development runoff rates.

Table 5-1: Post Development Peak Runoff Rates (Total)

Design Storm Event (Total Catchment Area = 24.03 ha)	**Pre-Development Peak Runoff Rates (m^3/s) [24.03 ha]**	**Post-Development Peak Runoff Rate (m^3/s) [24.03 ha]**	**Post<Pre**
01. 25mm4hrChicago	0.056	0.055	Yes
02. SCS II 12hr 2yr	0.236	0.197	Yes
03. SCS II 12hr 5yr	0.455	0.359	Yes
04. SCS II 12hr 10yr	0.629	0.497	Yes
05. SCS II 12hr 50yr	1.069	0.981	Yes
06. SCS II 12hr 100yr	1.269	1.267	Yes

A review of the above table indicates that the total runoff from the site, inclusive of the no-development zones in post-development conditions is less than the runoff rate of the respective pre-development storm event. The stormwater storage areas temporarily detain the runoff generated within the controlled areas and release at a controlled rate.

Appendix C contains the OTTHYMO detailed output file for the last link in the model. The detailed output file for the last link summarizes the post-development flows exiting the proposed development.

5.3.2 Post-Development Runoff to Adjacent Properties

Post-development Catchments direct all runoff towards stormwater management facilities, which discharge to either the wetland within the subject site, or to the Rideau River. Based on the available existing topography, no runoff is directed towards adjacent private property. Runoff directed towards the roadside ditch of County Road 23 is conveyed along the County Road ditch to the proposed ditches along Street 1.

5.3.3 Post-Development Runoff from uncontrolled areas

The subject site has four uncontrolled catchment areas in post-development conditions. U-F and U-WETL are located within the south portion of the site. U-WETL discharges directly into the wetland without restriction. U-F discharges towards the east and south following existing drainage patterns. The north portion of the site has two uncontrolled areas. U-RIVER is located along the rear yards of the lots on the west side of the road adjacent the river, but outside of the no-development zone. The second uncontrolled area in the north portion of the site is located within the park block, and drains via overland flow towards the Rideau River following existing drainage patterns.

5.3.4 Post-Development Runoff from No-Development Areas

The subject site has two no-development areas in post-development conditions as was in pre-development conditions. Section 4.4.2 discusses the no-development zones, which remain consistent between pre-development and post-development conditions and were included in the model to ensure that pre-development flows do not exceed post development flows for the entire site.

5.3.5 Storage Requirements

As previously mentioned the volume of the storage swale is directly related to the depth of water and the discharge rate through the outlet control structure. The south storage area will allow for a maximum ponding elevation of 90.80 metres and the north storage area will have a maximum ponding elevation of 94.30 metres. The bottom of the south storage area is at an elevation of 90.00m and north storage area is at 93.10m. The invert of the south storage swale’s outlet control structure is 0.10m above the bottom of the swale. This means that the stormwater captured below the invert of the outlet control structure will outlet by infiltration only. The invert of the outlet control structure within the north stormwater storage swale is at the invert of the storm swale. *Table 5-3 and Table 5-4* below summarizes the volume, depth, and drawdown time for each respective stormwater storage area for each storm event.

Table 5-2: South Storage Swale Storage Requirements

Design Storm Event	Volume (ha.m)	Depth (m)	Drawdown Time (hrs)*
01. 25mm4hrChicago	0.0091	0.25	6.7
02. SCS II 12hr 2yr	0.0246	0.46	7.0
03. SCS II 12hr 5yr	0.0400	0.60	8.4
04. SCS II 12hr 10yr	0.0477	0.67	8.6
05. SCS II 12hr 50yr	0.0603	0.76	8.7
06. SCS II 12hr 100yr	0.0648	0.80	8.8

*The drawdown time was only considered from the maximum ponding depth to the invert of the first outlet control, 0.10m above the invert of the storage swale. Discharge below the outlet control invert is by means of evapotranspiration and infiltration.

Given the presence of glacial till and silty clay soils at the site, an assumed percolation rate of 50 min/cm would equate to a hydraulic conductivity K of roughly 1.0×10^{-9} m/s or an infiltration rate of roughly 12 mm/hr. As such, the time for the 100mm depth of stored stormwater within the stormwater storage swale to infiltrate is about 8.3 hours. This would be considered as a conservative estimate as evaporation and water draw from vegetation is neglected. As such, the total drawdown time following the design storm events analyzed is between 15.0 hours and 17.1 hours respectively.

Table 5-3: North Storage Swale Storage Requirements

Design Storm Event	Volume (ha.m)	Depth (m)	Drawdown Time (hrs)
01. 25mm4hrChicago	0.0201	0.36	6.1
02. SCS II 12hr 2yr	0.0597	0.63	12.8
03. SCS II 12hr 5yr	0.0933	0.82	14.9
04. SCS II 12hr 10yr	0.1161	0.93	15.5
05. SCS II 12hr 50yr	0.1638	1.14	16.3
06. SCS II 12hr 100yr	0.1787	1.19	16.4

Since the invert of the first stage of the outlet control structure is level to the bottom elevation of the swale, the swale will drain in the above noted drawdown time following storm events without reliance on evapotranspiration and infiltration.

6.0 QUALITY CONTROL

As indicated in the Stormwater Management Planning and Design Manual published by the Ontario Ministry of the Environment (2003), the recommended strategy for stormwater management is to provide an integrated treatment train approach to water management. In general, best management practices for stormwater management quality control are divided into three categories: source control, conveyance control and end-of-pipe control. As indicated in the Ministry of Transportation Drainage Management Manual, the priority in applying these BMPs should follow the sequence presented with end of pipe measures applied as the last resort.

As previously stated, an enhanced level of treatment is required for the runoff from the site. An enhanced level of treatment corresponds to 80 percent total suspended solids removal.

6.1 Source Control

The primary source of total suspended solids and associated runoff pollution under post-development conditions is considered to be the areas of a site subject to vehicle traffic. At the proposed development, this consists of the driveways and roadways. The vegetated landscaped surfaces and dwelling roofs are typically not considered to be significant sources of suspended solids following the completion of the development and establishment of the vegetation in landscaped areas.

The application of de-icing chemicals including salts and sand can be reduced with a best management plan for the application of these products. BMPs with respect to de-icing chemicals include such measures as timing of application, targeted application, and clearing of snow cover before application.

6.2 Conveyance Control

In general runoff generated from the driveways will be directed across the grass surfaced front yards to the roadside ditch in front of the lots. The runoff from the roadways will also be conveyed along the roadside ditches to the stormwater storages swales located adjacent lot 28 and the wetland for the south portion of the site and between lot 12 and the park block on the north portion of the site. The longitudinal slope for the ditches on the south portion of the site upstream of the storage swale is proposed at between 2.3% and 2.6%. The roadside ditches along the north portion of the site immediately upstream of the stormwater storage swale have a longitudinal slope of between 0.5% and 0.9%. The low sloping of the swales in conjunction with the vegetation within the ditches will provide preliminary treatment by removing larger suspended solids.

Researchers at the University of Guelph (Rudra, Ramesh et al. “Sediment Removal Efficiency of Vegetative Filter Strips”, University of Guelph, 2001 have shown that vegetative filters can partially remove sediments and pollutants attached to sediment particles in runoff. Field experiments on vegetative filter strips showed average sediment removal varying from 50 to 98% as flow path length increases from 2.5 to 10 metres. The research indicates that almost all particles larger than 40 microns in diameter are captured within the first five meters of a filter strip provided the flow velocity is limited to less than 0.5 m/s during the quality control storm event. About 50% of the sediments are removed within the first 2.5 metre of travel over the vegetative filter flow path. An additional 25% to 45% of sediments are removed within the next 2.5 m of the flow path depending on the flow rate and velocity. The removal efficiency of the vegetative filtration does not significantly increase with a flow path length beyond 10 m.

The Ontario Ministry of Environment, Conservation and Parks considers the quality storm event to be the 4 hr 25 mm Chicago storm. When considering the peak runoff rate in the roadside ditches, the proposed development can be divided into the following areas based on the point of discharge of the ditch:

- Section 1 - Roadside Ditch along the west side of Street 1 south of the wetland;
- Section 2 - Roadside Ditch along the east side of Street 1 south of the wetland;
- Section 3 - Roadside Ditch along the west side of Street 1 north of the wetland conveying uncontrolled flow;

- Section 4 - Roadside Ditch along the east side of Street 1 north of the wetland conveying uncontrolled flow;
- Section 5 - Roadside Ditch along the west side of Street 1 north of the wetland conveying flow to the north storage swale;
- Section 6 - Roadside Ditch along the east side of Street 1 north of the wetland conveying flow to the north storage swale;

The peak flow rate in the above ditch sections during the quality control storm event and the resulting flow velocity in each section are summarized in the following Table 6-1

Table 6-1 – Peak Quality Control Flow Rate and Velocity in Roadside Ditches

Ditch Section	OTTHYMO Hydrograph #	Peak Flow Rate (m^3/s)	Ditch Slope (%)	Peak Flow Velocity (m/s)
Section 1	19	0.013	2.3	0.41
Section 2	18	0.011	1.9	0.36
Section 3	14(1)	0.007	4.8	0.45
Section 4	14(1)	0.007	3.7	0.39
Section 5	24	0.010	0.5	0.21
Section 6	33	0.030	0.5	0.28

1) The peak runoff rate in the sections of ditch north of the wetland convey uncontrolled flow was considered to be half of the uncontrolled flow into the wetland from the north portion of the development.

Since the peak flow velocity in each section of ditch is less than 0.5 m/s during the quality control storm event, the vegetation in the roadside ditch will be effective at removing total suspended solids prior to the runoff reaching the outlet of the ditch.

6.3 End-of-Pipe Control

The discharge from both of the north and south portions of sites are through an outlet control structure located at the north end of each respective stormwater storage area. Once discharged from the storage swale, the runoff from the south portion of the site will be directed towards the existing wetland via an outlet swale. Runoff discharging from the storage swale in the north portion of the site will be directed towards the Rideau River via an outlet swale located in adjacent the park block east of Lot 1. A sand filter will be installed within each of the respective outlet swales and will be discussed in sections 6.3.1 below.

The sand filter will be designed in accordance with the Ministry of Environment requirements for sand filters.

When considering the discharge from the stormwater storage areas during a quality storm event, the south storage area has a release rate of 0.006m3/sec and the north storage area has a release rate of 0.011m3/sec respectively. The controlled outlet of the stormwater storage swales allows for a sand filter to be appropriately sized ensuring that all of the outflow from the stormwater storage area during a quality storm event passes through the sand filter prior to discharging to the wetland and Rideau River respectively.

As mentioned above, the outflow from the south portion of the site through the outlet control structure is directed towards an outlet swale, which conveys the runoff towards the existing wetland. The outlet swale has a bottom width of 0.50m and minimum depth of about 0.90m immediately downstream of the outlet control structure in the area of the proposed sand filter. The side slopes are approximately 3 units horizontal to 1 unit vertical and has a longitudinal slope of 2.1%.

The swale downstream of the stormwater storage swale on the north portion of the site has a depth of approximately 0.80 metres and bottom width of 0.50m, and side slope of 3 units horizontal to 1 unit vertical. The longitudinal slope of the swale is 1.6%.

The flow rates and associated flow velocity and depth during various storm events are as shown in the following Table 6-2 and Table 6-3:

Table 6-2 – Flow Rate, Velocity and Depth along Outlet Swale for South Portion of Site

Storm Event	Flow Rate (m^3/s)	Flow Velocity (m/s)	Flow Depth (m)
Quality Control	0.006	0.28	0.04
2 year Storm	0.016	0.38	0.06
5 year Storm	0.040	0.5	0.1
10 year Storm	0.078	0.6	0.14
50 year Storm	0.198	0.77	0.22
100 year Storm	0.253	0.82	0.25

Table 6-3 – Flow Rate, Velocity and Depth along Outlet Swale for North Portion of Site

Storm Event	Flow Rate (m^3/s)	Flow Velocity (m/s)	Flow Depth (m)
Quality Control	0.011	0.31	0.06
2 year Storm	0.023	0.39	0.08
5 year Storm	0.059	0.5	0.13
10 year Storm	0.098	0.58	0.17
50 year Storm	0.246	0.73	0.26
100 year Storm	0.346	0.8	0.31

Section 4.5.9 of the MOE Manual provides the design guidance with respect to the use of Grassed Swales as summarized in the following Table 6-4. A column has been added to indicate how the proposed design conforms to the Criteria for each.

Table 6-4 – Summary of MOE Design Guidance for Grassed Swales and Conformance

Design Element	**Design Objective**	**Minimum Criteria**	**Design Conformance / Comment**
Physical Criteria	Intended to facilitate the Flow Criteria		
Drainage Area	Limit Peak Flow Rate and Velocity below Flow Criteria	Dependent on Impervious Area < 1.5 hectares	The controlled areas in the south portion of the site have a total area of 4.15 hectares, 0.68 hectares are impervious (including roofs) The controlled areas in the north portion of the site have a total area of 8.19 hectares, 1.88 hectares of impervious (including roofs) Exceeds criteria
Side Slopes	Increase vegetative contact	Max 2.5H:1V	3H:1V side slopes on both outlet swales
Channel Slope	reduce flow velocity	< 1 percent	North Swale: 1.6% South Swale: 2.1% slightly higher than recommended.
Bottom Width	Increase vegetative contact	min 0.75 m	0.50 m bottom width, vegetated side slopes allow for increased vegetated contact

Flow Criteria	Required to Achieve Quality control		
Peak Flow Velocity	Facilitate Sedimentation and vegetative filtration	< 0.5 m/s	Velocity is -0.5 m/s or less for all storm events up to the 5 year design storm for both north and south outlet swales
Flow Depth	Promote Vegetative Filtration	< 0.5 m	0.04 m and 0.06 m for the south and north swales respectively during the quality control storm 0.1 m and 0.17 m for the south and north swales respectively during the 5 year design storm
Flow Rate	Sedimentation and prevent re-suspension	≤ 0.15 m^3/s	0.006 m^3/s and 0.011 m^3/s for the south and north swales respectively during the quality control storm 0.004 m^3/s and 0.059 m^3/s for the south and north swales respectively during the 5 year design storm

The physical criteria with respect to stormwater treatment by vegetative filtration consisting of side slope, channel slope and bottom width are intended to ensure that the flow criteria are not exceeded. The physical criteria are exceeded for each overall catchment. However, the outlet swales are downstream of the outlet structure for each stormwater storage area ensuring that the flow rate into the outlet swale is controlled. Since the flow rate is controlled, the fow rate, flow velocity and flow depth are much lower than the flow criteria during a quality storm event and remain lower than the flow criteria during a 5 year storm event. Since the flow rate, flow depth and flow velocity are less than the design criteria during a quality control storm, the vegetated outlet swales will be effective at removing suspended solids during quality storm events.

Table 2.3 of the Ontario Ministry of Natural Resources Technical Guide – River and Stream Systems: Erosion Hazard Limit provides an allowable flow velocity for a bare channel in sand and silt of 0.61 m/s and in clay of 0.76 m/s. The allowable flow velocity increases to a maximum of 0.91m/s with fair vegetative cover.

The flow velocity of 0.5 m/s, which occurs in both outlet swales during the peak flow of the 5 year storm event, is less than the scour velocity in a bare channel in sand and silt. Since the flow velocity during a 5 year storm is less than the scour velocity of a bare channel there will be no scour or re-suspension of sediment. Velocities in both swales exceed 0.61 m/s in design storms with intensities exceeding the 50-year storm event, but remain below the maximum allowable velocity of 0.91 m/s assuming fair vegetative cover even during the 100 year design storm events.

It is assumed that the vegetation will have a chance to grow through the sediment prior to successive 100 year storm events resulting in at least a fair vegetative condition in the swale. As such, the vegetation in the outlet swales will provide effective pre-treatment and total suspended solids removal upstream of the sand filters which will be used for effluent polishing prior to outlet in the undesignated wetland or the Rideau River.

6.3.1 Sand Filter

Part 4 of the MOE Manual details the design requirements for several types of end of pipe stormwater management facilities. Design guidance for filtration is provided in Section 4.6.7 – Filters of the MOE Manual. Section 4.6.7 provides the design guidance with respect to the use of a filter as summarized in Table 6-5 below. A column has been added to indicate how the proposed design conforms to the criteria outlines in the MOE Manual.

Table 6-5 – Summary of MOE Design Guidance for Filters

Design Element	Design Objective	Minimum Criteria	Design Conformance
Drainage Area		< 5 hectares	South portion of the site ~4.15 ha North portion of the site ~8.19 ha
Pre-treatment	Longevity	Pre-treatment by means of sedimentation chamber, or forebay, vegetated filter strip, swale or oil/grit separator	Vegetative filtration on grass covered side slopes and bottom of roadside ditches, additional settlement within the stormwater storage swales, followed by additional vegetative filtration within the outlet swale prior to reaching sand filter.

Storage Depth	Avoid Filter Compaction	Subsurface sand and organic filters: 0.5 m Maximum 1.0 m	South Portion of the site: Maximum 0.15m ponding elevation before overtopping filter and berm. North portion of site: Flow depth of between 0.03m - 0.19m within swale over the filter. Maximum ponding depth of 0.15 m due to berm.
Filter Media Depth	Filtering	Sand: 0.5 m	Sand Filters each have a minimum thickness of 0.5 m (South storage area horizontally, north storage area vertically).
Under-drain	Discharge	Minimum 100 mm perforated pipes bedded in 150 – 300 mm of 50 mm gravel	South - No under-drain provided. Filtration is horizontal in South filter rather than vertical. North - 100mm perforated under-drain provided in the North filter Vegetation to consist of plants suitable for extended periods of submersion.
Land use		any land use, often employed for commercial and industrial	Residential
Volumetric Sizing		provided in Table 3.2 under infiltration. By-pass flows should not occur below a 4 hr 15 mm design event	Upstream attenuation on both north and south portions of the site ensure that no bypass of the filter occurs for storm events up to and including the 4 hr 25mm storm event.
Filter Size		Determined using the Darcy Equation	Sized to ensure that the peak flow of the filer during a quality storm event does by-pass the filter
Filter Lining	prevent clogging	liner to prevent native material from entering filter	Non-woven geotextile filter cloth used between sand filter and clearstone, rip-rap and native soils
Overflow / by-pass		required	Overflow is provided above the filters to ensure excessive flows are still contained within the swale without overtopping the berm or road.

Drawdown time	prevent standing water	from 24 to 48 hours 24 hours preferred	Drawdown 5.3.5 above shows drawdown within the storage swale is <24 hours. Sand filter has estimated "T" time of 2 min/cm. As such long term detention upstream of the filter and downstream of the outlet control structure is not anticipated.

6.3.2 Volumetric Sizing and Filter Size

The MOE Manual in section 4.6.7 under the heading Volumetric Sizing provides the following additional design guidance when using filtration for quality control:

"Water quality volumes to be used in the design are provided in Table 3.2 under the "infiltration" heading. Erosion and quantity control volumes are not applicable to this type of SWMP. The design should be such that at a minimum, the by-pass of flows should not occur below or at the peak runoff from a 4 hour 15 mm design event."

As previously discussed, the 24mm 4 hr Chicago storm was selected for the purposes of quality control for the site. As such, the entire quality control volume required by the MOE Manual will be detained and will pass through the sand filter without by-pass. The proposed filters have been sized based on the space available for the filter. The flow rate through the filter was calculated and drawdown time was determined based on the volume of the quality storage in the catchment.

The sand filter locations and details area illustrated on drawing 210816-GR and details for the filters are included on drawings 210816-DET.

6.3.3 Sand Filter – South Stormwater Storage Area

The sand filter in the stormwater management facility at the southern portion of the site will be constructed in place of the back slope of the roadside ditch downstream of the outlet control structure. The bottom of the filter will be at the same elevation as the bottom of the subdrain from the storage swale. Flow from the subdrain and roadside ditch will flow horizontally through the filter to the adjacent wetland. The sand filter will be placed for a length of 25 metres parallel to the road and then will be extended perpendicularly to the roadway to tie into the road sideslope. The filter will be overtopped at a depth of 0.15 metres.

The filter will have a top width of 0.50m and side slopes of 1 units horizontal to 1 vertical. The sand filter will have a height of 0.45m at the core. The filter will be covered with a 6 ounce per square yard non-woven geotextile followed by rip-rap conforming to the grading specifications of OPSS 1004 R-10 rip-rap. The rip-rap will be sloped at 2 units horizontal to 1 unit vertical and minimum thickness of 0.20m.

The filter will be constructed with imported filter media sand meeting the gradation requirements specified in Part 8.7.5.3(3) of the Ontario Building Code. This sand will have a percolation time of about 2 min/cm, which corresponds to a hydraulic conductivity of roughly 0.001 m/sec in accordance with the table below:

Table C1: Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

Details for the sand filter can be found on drawing 210816-DET.

6.3.4 Discharge Through the Sand Filter – South Stormwater Storage Area

The average flow rate through the sand filter was calculated using Darcy's Equation to be:

$$A = \frac{1000Vd}{k(h+d)t}$$

Where:

A = surface area of the filter
d = depth of the controlling filter medium
k = coefficient of permeability of the controlling filter media
h = operating head of the water on the filter
t = drawdown time

The formula can further be simplified to consider the upstream detention to the peak flow rate and the hydraulic gradient over the trapezoidal cross section of the filter to:

$$A = \frac{Q}{ki}$$

Where:

A = surface area of the filter
k = coefficient of permeability of the filter media sand = $1x10^{-3}$ m/s
Q = quality storm flow rate = 0.006 m^3/sec
i = hydraulic gradient across the filter = h/L where
h is head across the filter = 0.45 m
L = average flow length through the filter = 0.8 m

$$A = \frac{0.006\frac{m^3}{s}}{0.001\frac{m}{s} * 0.56\frac{m}{m}}$$

A= 10.7 m^2

As such, the minimum surface area of the filter is 10.7 m^2. When considering the maximum height of the filter is 0.45m, the length of the filter can be taken as:

$$L = \frac{A}{h}$$

Where:

L = length of the filter
A = minimum surface area of the filter = 10.7 m^2
h = height of filter = 0.45m

As such the minimum length of the filter is 23.8 m

The actual filter length will be 25 metres exceeding the minimum design length of the filter ensuring all of the flow from the quality storm event is filtered without by-pass of the filter.

6.3.5 Sand Filter – North Stormwater Storage Area

The sand filter in the north portion of the site will be constructed as a vertical filter within the conveyance swale in the park block between the cul-de-sac and the discharge point of the swale into the Rideau River. The proposed filter will be constructed with a width of 0.50m (The same as the bottom width of the swale) and a sand depth of 0.50m. The filter sand will be placed on top of a 0.15m deep layer of clear stone complete with 100mm diameter drain tile. The 100mm diameter subdrain will discharge to the surface at the outlet location of the swale. A non-woven geotextile fabric (such as Terrafix 270R or an approved alternative) will be placed between the sand and the clear stone and around the clear stone and sand to avoid contamination from the underlying native material and mixing of the filter sand with the clear stone.

The swale at downstream end of the filter will be fitted with a check dam consisting of a clay core covered with a non-woven geotextile and rip-rap conforming to the grading specifications of OPSS 1004 R-10 rip-rap. The rip-rap will be sloped at 3 units horizontal to 1 unit vertical and minimum thickness of 0.20m. The check dam will have a height of 0.15 m.

6.3.6 Discharge Through the Sand Filter – North Stormwater Storage Area

The average flow rate through the sand filter was calculated using Darcy's Equation as was done in section 6.3.1.3 above. As determined above, the flow depth within the swale during a quality storm event is 0.03m. Inserting the flow depth into the modified equation with the peak flow and maximum hydraulic gradient can be determined as follows:

$$A = \frac{Q}{ki}$$

Where:

A = surface area of the filter
k = coefficient of permeability of the filter media sand = $1x10^{-3}$ m/s
Q = quality storm flow rate = 0.011 m^3/sec
i = hydraulic gradient across the filter = (h+d)/d where:
 h = flow depth over filter during quality event = 0.04 m
 d = depth of filtering layer thickness = 0.5

$$A = \frac{0.011\frac{m^3}{s}}{0.001\frac{m}{s} * 1.08\frac{m}{m}}$$

$$= 10.2\ m^2$$

As such, the minimum surface area of 10.2 square metres is required for the filter located on the north portion of the site. This will be facilitated by a 0.50m wide filter (same as the bottom width of the filter) with a length of 22m resulting a slightly larger surface area than required. As previously indicated, a check dam will be located on the downstream side of the filter to ensure the entire quality flow is directed through the filter.

6.4 Quality Control Summary

Based on the above information, quality control to an enhanced level will be achieved as follows:

- Potential pollutants can be reduced at the source by proper best management practices with respect to snow and ice removal.
- Roadside ditches will provide preliminary treatment removing larger suspended solids
- Final polishing will occur by settling within the stormwater storage areas prior to passing through a sand filter at the outlet of each of the respective stormwater storage areas.

6.5 Best Management Practices

Best Management Practices shall be implemented during and following construction as follows to reduce transport of sediments.

- Construction works are to be timed in order to reduce the length of time between the beginning of construction and the establishment of vegetative cover.
- Keep sediment and erosion control measures in place and maintained during and following construction until vegetation is established.
- Do not disturb vegetated areas outside of the development foot print.
- Use appropriate equipment for the development to reduce the duration of the development.
- Work should be timed to avoid the wet seasons of the year.
- Roof runoff should be discharged onto the ground surface and directed to the grass surfaced swales.
- Winter snow removal, together with salting and sanding should be completed in accordance with an established plan and best management practices to reduce the amount of sand and salt required.

7.0 STORMWATER MANAGEMENT SUMMARY

In general, the stormwater management plan consists of collecting the stormwater runoff from the controlled area portions of the site and controlling the release rate such that the post-development release rates do not exceed the pre-development maximum flow rates for all quantity control storm events to and including the 100 year storm events. The proposed stormwater management design will provide an enhanced level of treatment for the runoff from the site.

Specifically, stormwater runoff from controlled catchments will be directed to roadside ditches and rear yard swales before being conveyed to their respective stormwater management facilities. Stormwater from the south portion of the site will be directed towards the stormwater storage swale located adjacent lot 28, and will discharge into the wetland on the site prior to ultimately outletting to the Rideau River. Runoff from the north portion of the site will be directed towards the stormwater storage swale located along the east property line adjacent the park block and will discharge directly into the Rideau River

The outlet of the each stormwater storage area will be controlled to ensure peak post-development runoff is less than peak pre-development runoff for the site for design storm events up to and including the 100 year event. Roadside ditches will provide preliminary quality treatment with final polishing occurring within the stormwater storage areas by settling and sand filter to achieve an enhanced level of protection corresponding to 80% long-term suspended solids removal.

8.0 EROSION AND SEDIMENT CONTROL

8.1 Sediment and Erosion Control

The owner (and/or contractor) agrees to prepare and implement an erosion and sediment control plan at least equal to the stated minimum requirements and to the satisfaction of the Township and the Conservation Authority, appropriate to the site conditions, prior to undertaking any site alterations (filling, grading, removal of vegetation, etc.) and during all phases of site preparation and construction in accordance with the current best management practices for erosion and sediment control. It is considered to be the owners and/or contractors responsibility to ensure that the erosion control measures are implemented and maintained.

In order to limit the amount of sediment carried in stormwater runoff from the site during construction, it is recommended to install a silt fence along the property line or limit of development line, as shown in Kollaard Associates Inc. drawing #210816-ESC. The silt fence may be polypropylene, nylon, and polyester or ethylene yarn.

If a standard filter fabric is used, it must be backed by a wire fence supported on posts not over 2.0 m apart. Extra strength filter fabric may be used without a wire fence backing if posts are not over 1.0 m apart. Fabric joints should be lapped at least 150 mm (6") and stapled. The bottom edge of the filter fabric should be anchored in a 300 mm (1 ft) deep trench, to prevent flow under the fence. Sections of fence should be cleaned, if blocked with sediment and replaced if torn.

A straw bale check damn should be installed at the outlet of each of the stormwater management facilities upstream of the wetland and Rideau River respectively to prevent sediment from migrating to the wetland and river.

The proposed landscaping works should be completed as soon as possible. The proposed asphaltic concrete surfaced areas should be surfaced as soon as possible.

These measures will reduce the amount of sediment carried from the site during storm events that may occur during construction.

As each lot is developed, proper sediment and erosion controls will be installed and maintained until the development of the lot is completed and the vegetative cover is established. Sediment controls shall consist of, at minimum, a silt fence barrier at the down gradient property line. Grass shall be established as soon as reasonably possible.

The attached drawing #210816-ESC for the subdivision includes the above noted measures. These measures are intended to ensure no sediment laden runoff leaves the site or impacts the water way either during construction or after development has been completed

9.0 CONCLUSIONS

- The proposed residential subdivision consists of an area of approximately 24.0 hectares which will be divided into 29 single family residential estate lots each with private septic and well inclusive of the existing single family dwelling on the site as well as the existing detached garage, which is to be renovated to a single family dwelling as part of the proposed development..
- The property is currently occupied by a single family dwelling and detached garage. The areas immediately adjacent the structures is maintained and landscaped. The remainder of the site is in a natural state and consists of a mixture of scrub vegetation and young to mature trees.
- Stormwater will be directed to stormwater management facilities where it will be detained and released at a controlled rate by outlet structures at each facility.
- The stormwater runoff will be treated in a treatment train configuration with pre-treatment occurring within roadside ditches and final polishing occurring in sand filters within the two stormwater management areas proposed on the site.

- Two no-development areas are proposed to not be altered as a result of the proposed development; the area adjacent the shoreline of the Rideau River, and the proposed wetland which crosses the site are to be left in a natural state as a result of the proposed development, but were still included in the stormwater management model.
- Runoff will be managed from the site to ensure that the post-development runoff does not exceed the pre-development runoff.
- Erosion measures will be placed prior to construction and during development and will remain in place until construction is complete. Disturbed areas will be top soiled and seeded as soon as reasonably possible.

We trust that this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we can be of any further assistance to you on this project, please do not hesitate to contact our office.

Sincerely,
Kollaard Associates Inc.

Steve deWit, P.Eng.

APPENDIX A: SITE PARAMETERS

IDF parameters for Kemptville, Ontario

Active coordinate

44° 37' 45" N, 75° 49' 44" W (44.629167,-75.829167)

Retrieved: Tue, 08 Nov 2022 15:57:27 GMT

Coefficient summary

IDF Curve: 44° 37' 45" N, 75° 49' 44" W (44.629167,-75.829167)

Retrieved: Tue, 08 Nov 2022 15:57:27 GMT

Data year: 2010
IDF curve year: 2010

Return period	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
A	20.5	27.3	31.8	37.4	41.5	45.6
B	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Statistics

Rainfall intensity (mm hr^{-1})

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	116.4	71.7	54.0	33.3	20.5	12.6	5.9	3.6	2.2
5-yr	155.1	95.5	71.9	44.3	27.3	16.8	7.8	4.8	3.0
10-yr	180.6	111.3	83.8	51.6	31.8	19.6	9.1	5.6	3.4
25-yr	212.4	130.9	98.6	60.7	37.4	23.0	10.7	6.6	4.1
50-yr	235.7	145.2	109.4	67.4	41.5	25.6	11.9	7.3	4.5
100-yr	259.0	159.5	120.2	74.0	45.6	28.1	13.0	8.0	4.9

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	9.7	12.0	13.5	16.6	20.5	25.3	35.2	43.3	53.4
5-yr	12.9	15.9	18.0	22.2	27.3	33.6	46.8	57.7	71.1
10-yr	15.1	18.5	21.0	25.8	31.8	39.2	54.5	67.2	82.8
25-yr	17.7	21.8	24.6	30.4	37.4	46.1	64.1	79.0	97.3
50-yr	19.6	24.2	27.3	33.7	41.5	51.1	71.2	87.7	108.0
100-yr	21.6	26.6	30.0	37.0	45.6	56.2	78.2	96.3	118.7

APPENDIX A: SITE PARAMETERS

Catchment Area Properties and OttHymo Parameters

Refer to Drawing # 220226-PRE and Drawing # 220226-POST for an illustration of the specified catchment areas.

See next page…

HYDROLOGIC SOIL GROUP	C									
	NASHYD CATCHMENT AREAS										

	TOTAL AREA m^2	2-2c [BRUSH WEED GRASS](FAIR)	2-2a [OPEN SPACE](GOOD)	2-2c [SEMI PERMENANT WETLAND)	2-2a [IMP](ROOFS ETC)	2-2a [STREET](GRAVEL)	2-2a [URBAN DISTRICT](COMMERCIAL)	2-2a [URBAN DISTRICT](INDUSTRIAL)	WEIGHTED AVERAGE CN	Impervious Ratio	POTENTIAL STORAGE [mm]
		CN= 70	CN= 74	CN= 89	CN= 98	CN= 89	CN= 94	CN= 91			
PRE-SOUTH	52716	51763	0	0	0	953	0	0	70.3	0.0%	107.3
PRE-NORTH	138760	128837	7780	0	740	1403	0	0	70.6	0.5%	105.8
ND-WETLAND	14428	0	0	14428	0	0	0	0	89.0	0.0%	31.4
ND-RIVER	34346	32736	1300	0	310	0	0	0	70.4	0.9%	106.8
C-1	12272	3068	5654	0	3250	300	0	0	79.7	26.5%	64.7
C-2	29269	7317	18092		3560	300	0	0	76.1	12.2%	79.8
C-3	6772	1693	3299		1560	220			79.0	23.0%	67.5
C-4	21381	5345	11906		3630	500			77.4	17.0%	74.2
C-5	12572	3143	6026		3254	150			79.4	25.9%	65.9
C-6	10272	2563	5819		1780	110			77.3	17.3%	74.6
C-7	10159	2540	5719		1750	150			77.4	17.2%	74.2
C-8	6654	1664	2451		2300	240			81.8	34.6%	56.5
C-9	2033	508	840		600	85			80.7	29.5%	60.7
C-10	4270	1068	1553		1570	80			82.1	36.8%	55.4
C-POND	7853	785	5498	680	700	190	0	0	77.4	8.9%	74.2
U-PARK	4362	1091	2702		500	70	0	0	76.0	11.5%	80.2
U-RIVER	21900	5475	14425		2000	0	0	0	75.2	9.1%	83.8
U-WETL	30530	7633	19268		3290	340	0	0	75.8	10.8%	81.1
U-F(FRONT)	11177	2794	8383		0	0	0	0	73.0	0.0%	93.9
ND-WETLAND	14428	0	0	14428	0	0	0	0	89.0	0.0%	31.4
ND-RIVER	34346	32736	1300	0	310	0	0	0	70.4	0.9%	106.8

OTTHYMO NASHYD PARAMETERS

NHYD	NAME	COMMENTS 1	DT [min]	AREA [ha]	DWF [m³/s]	CN	IA [mm]	N	TP [hr]	STORM INDEX	RAIN [mm/hr]
1	PRE-SOUTH	PRE	5	5.272	0	70.3	10.7	3	0.30	1	0
2	PRE-NORTH	PRE	5	13.876	0	70.6	10.6	3	0.57	1	0
3	ND-	PRE	5	1.443	0	83.0	4.7	3	0.76	1	0
4	ND-RIVER	PRE	5	3.435	0	70.4	10.7	3	0.38	1	0
1	C-1	POST -	5	1.227	0	70.7	6.5	3	0.17	1	0
2	C-2	POST -	5	2.927	0	76.1	8.0	3	0.31	1	0
3	C-3	POST -	5	0.677	0	73.0	6.8	3	0.17	1	0
4	C-4	POST -	5	2.138	0	77.4	7.4	3	0.17	1	0
5	C-5	POST -	5	1.257	0	73.4	6.6	3	0.17	1	0
6	C-6	POST -	5	1.027	0	77.3	7.5	3	0.17	1	0
7	C-7	POST -	5	1.016	0	77.4	7.4	3	0.17	1	0
8	C-8	POST -	5	0.665	0	81.8	5.7	3	0.17	1	0
9	C-9	POST -	5	0.203	0	80.7	6.1	3	0.17	1	0
10	C-10	POST -	5	0.427	0	82.1	5.5	3	0.17	1	0
11	C-POND	POST -	5	0.785	0	77.4	7.4	3	0.17	1	0
12	U-PARK	POST -	5	0.436	0	73.0	8.0	3	0.17	1	0
13	U-RIVER	POST -	5	2.190	0	75.2	8.4	3	0.28	1	0
14	U-WETL	POST -	5	3.053	0	75.8	8.1	3	0.31	1	0
15	U-F(FRONT)	POST -	5	1.118	0	73.0	9.4	3	0.24	1	0
16	ND-	PRE	5	1.443	0	83.0	4.7	3	0.76	1	0
17	ND-RIVER	PRE	5	3.435	0	70.4	10.7	3	0.38	1	0

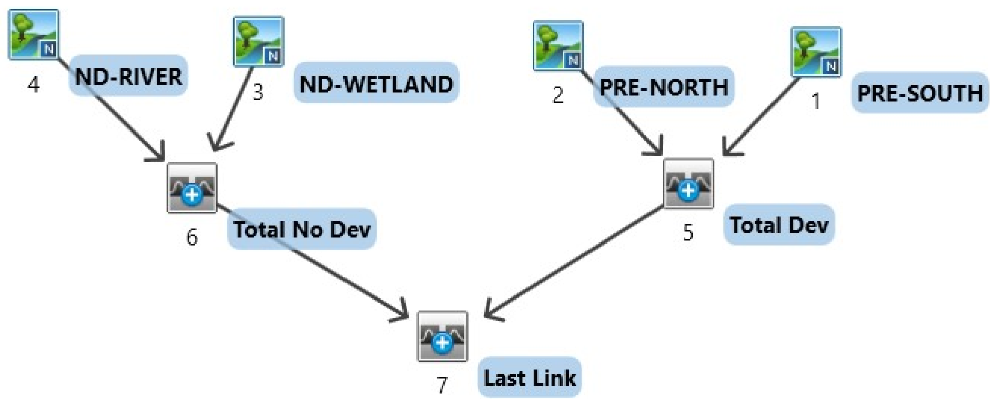
APPENDIX B: PRE-DEVELOPMENT DATA

Pre-development OTTHYMO Model Schematic

Pre-development Schematic Summary Table

Pre-development Detailed Output File from Last link in Model

OTTHYMO Pre-Development Model Schematic



Schematic Summary Table

Hydrograph No.	Command	Catchment Represented	Comment
For a detailed illustration of pre-development catchment areas see drawing 210816 – PRE			
1	NASHYD	PRE-SOUTH	South catchment. Represents pre-development catchment between County Road 23 and the existing wetland
2	NASHYD	PRE-NORTH	North Catchment. Represents pre-development catchment between the wetland and the no-development area adjacent the Rideau River.
3	NASHYD	ND-WETLAND	Wetland area. Represents the no-development area occupied by the existing wetland on the site
4	NASHYD	ND-RIVER	Represents the no-development area immediately adjacent the Rideau River.
5	ADDHYD	N/A	Used to sum the flows from the development areas.
6	ADDHYD	N/A	Used to sum the flows from the development areas.
7	ADDHYD	N/A	Last link in model. Represents total pre-development flow.

```
*********************************
** SIMULATION:25mm4hrChicago  **
*********************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):    19.15   0.036     2.67     1.71
      + ID2= 2 (  0006):     4.88   0.020     2.50     3.55
        =====================================================
        ID = 3 (  0007):    24.03   0.056     2.58     2.08

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------------
*************************************
** SIMULATION:SCS II 12hr 100yr M **
*************************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):    19.15   0.966     6.33    37.07
      + ID2= 2 (  0006):     4.88   0.303     6.33    45.70
        =====================================================
        ID = 3 (  0007):    24.03   1.269     6.33    38.82

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------------
************************************
** SIMULATION:SCS II 12hr 10yr M **
************************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):    19.15   0.470     6.42    18.92
      + ID2= 2 (  0006):     4.88   0.159     6.42    25.22
        =====================================================
        ID = 3 (  0007):    24.03   0.629     6.42    20.20

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------------
***********************************
** SIMULATION:SCS II 12hr 2yr M **
***********************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):    19.15   0.169     6.50     7.43
      + ID2= 2 (  0006):     4.88   0.066     6.50    11.34
        =====================================================
        ID = 3 (  0007):    24.03   0.236     6.50     8.23

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------------
```

```
**********************************
** SIMULATION:SCS II 12hr 50yr M **
**********************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):     19.15   0.810     6.33    31.42
      + ID2= 2 (  0006):      4.88   0.259     6.33    39.43
        =====================================================
        ID = 3 (  0007):     24.03   1.069     6.33    33.05

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
*********************************
** SIMULATION:SCS II 12hr 5yr M **
*********************************

--------------------
| ADD HYD  (  0007)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0005):     19.15   0.337     6.42    13.86
      + ID2= 2 (  0006):      4.88   0.119     6.42    19.25
        =====================================================
        ID = 3 (  0007):     24.03   0.455     6.42    14.96

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
```

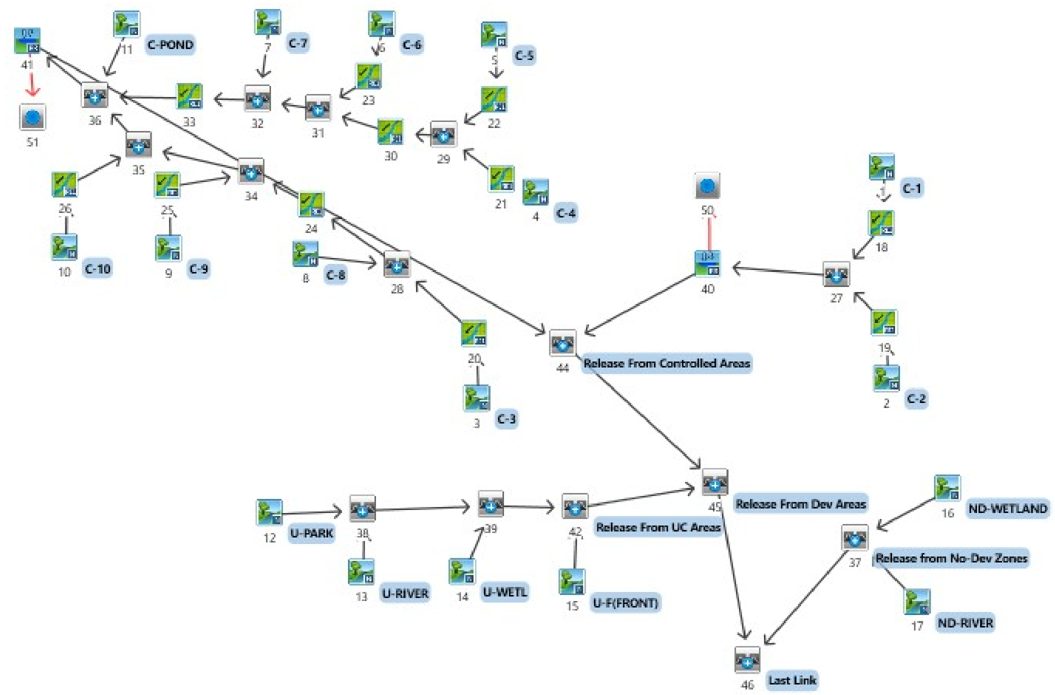
APPENDIX C: POST-DEVELOPMENT DATA

Post-Development OTTHYMO Model Schematic

Post-Development Schematic Summary Table

Post-Development Detailed Output File from Last Link in Model

OTTHYMO Post-Development Model Schematic



Schematic Summary Table

Hydrograph No.	Command	Catchment Represented	Comment
For a detailed illustration of pre-development catchment areas see drawing 210816 – POST			
1,2,3,4,5,6,7,8,9, 10	NASHYD	C-1,C-2,C-3,C-4, C-5, C-6,C-7,C-8, C-9,C-10	Controlled Catchment (impervious ratio <20%)
11	NASHYD	C-Pond	Controlled Catchment occupied by stormwater storage swale in the north portion of the site.
12,13	NASHYD	U-PARK, U-RIVER	Uncontrolled Catchment (Impervious Ratio <20%) on north portion of site, which discharges directly into the Rideau River without restriction.
14, 15	NASHYD	U-WETL, U-F(FRONT)	Uncontrolled Catchment (Impervious Ratio <20%) on south portion of site, which discharges directly into the Wetland without restriction.
16,17	NASHYD	ND-WETLAND, ND-RIVER	No-development areas, unchanged from pre-development conditions.
18,19,20,21,22, 23,24,25,26,30, 33	ROUTECHANNEL	N/A	Represents roadside ditches
37	ADDHYD	N/A	Sums release from no-development areas

38	ADDHYD	N/A	Sums total discharge from uncontrolled areas along the north portion of the site.
39	ADDHYD	N/A	Additional sum of discharge from uncontrolled areas from the south portion of the site.
42	ADDHYD	N/A	Total outflow from the uncontrolled areas of the site.
27	ADDHYD	N/A	Total flow from the controlled areas in the south portion of the site prior to entering the respective stormwater storage swale
28,29,31,32,34, 35	ADDHYD	N/A	Adds flows from catchments and ditches
36	ADDHYD	N/A	Sums all flows from controlled catchments in the north portion of the site prior to entering the stormwater storage swale.
40	ROUTERESERVOIR	N/A	Represents the storage swale in the south portion of the site
41	ROUTERESERVOIR	N/A	Represents the storage swale in the north portion of the site.
50, 51	JUNCTION	N/A	Overflow from the storage swales (used to ensure discharge is only through the respective outlet control structure).
44	ADDHYD	N/A	Total discharge from controlled areas
45	ADDHYD	N/A	Total discharge from developed areas
46	ADDHYD	N/A	Last link in model – represents total discharge from the site in post-development conditions.

```
********************************
** SIMULATION:25mm4hrChicago  **
********************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.020     2.50     3.55
      + ID2= 2 (  0045):    19.15   0.037     2.08     3.16
        ====================================================
        ID = 3 (  0046):    24.02   0.055     2.33     3.23

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
************************************
** SIMULATION:SCS II 12hr 100yr M **
************************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.303     6.33    45.70
      + ID2= 2 (  0045):    19.15   0.968     6.42    46.35
        ====================================================
        ID = 3 (  0046):    24.02   1.267     6.42    46.22

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
***********************************
** SIMULATION:SCS II 12hr 10yr M **
***********************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.159     6.42    25.22
      + ID2= 2 (  0045):    19.15   0.347     6.17    25.26
        ====================================================
        ID = 3 (  0046):    24.02   0.497     6.25    25.25

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 2yr M **
**********************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.066     6.50    11.34
      + ID2= 2 (  0045):    19.15   0.141     6.17    11.03
        ====================================================
        ID = 3 (  0046):    24.02   0.197     6.25    11.09

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
```

```
***********************************
** SIMULATION:SCS II 12hr 50yr M **
***********************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.259     6.33    39.43
      + ID2= 2 (  0045):    19.15   0.725     6.42    39.90
        =====================================================
        ID = 3 (  0046):    24.02   0.981     6.42    39.81

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 5yr M **
**********************************

--------------------
| ADD HYD  (  0046)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0037):     4.88   0.119     6.42    19.25
      + ID2= 2 (  0045):    19.15   0.252     6.17    19.13
        =====================================================
        ID = 3 (  0046):    24.02   0.359     6.25    19.15

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
-------------------------------------------------------------------------------
```

APPENDIX D: STORMWATER STORAGE SWALES

Outlet Control Design Sheet – South Storage Swale

Outlet Control Design Sheet – North Storage Swale

Kollaard Associates
Engineers
210 Prescott Street, Unit 1
P.O. Box 189
Kemptville, Ontario K0G 1J0

Civil ●
Geotechnical ●
Hydrogeological ●
Inspection Testing ●
Septic Systems Grading ●
Structural ● Environmental ●

APPENDIX D: STORMWATER MANAGEMENT MODEL

OUTLET CONTROL DESIGN SHEET - SOUTH STORAGE SWALE

Client: 2873633 Ontario Inc
Job No.: 210816
Location: 819 County Road 23, Village of Merrickville Wolford, Ontario
Date: December 20, 2024

Orifice	
Dia (m):	0.100
Area (m2.):	0.0079
Orifice Coeff, C_d:	0.60
Orifice Coeff , C_w:	0.62
Orifice Top (m):	90.20
Orifice Cen (m):	90.15
Orifice Inv (m):	90.10

Outlet Weir	
	V-Notch
Weir Invert (m):	90.35
Notch Angle	60

Overflow Channel	
Bottom Channel Width (m):	0.80
Channel Invert (m):	90.65

							Orifice Flow			Weir			Overflow Channel				
Stage, WSE Elev (m)	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Volume in Swale (m^3)	Quality Storage Volume (m3)	Quantity Storage (m3)	Head (m)	n	Flow (m^3/sec)	Head (ft)	Cw	Weir Flow (m^3/sec)	Head (m)	Channel Flow (m^3/sec)	Total Outflow (m^3/sec)	Total Outflow (L/sec)	Draw Down Time (hrs)
90.80	0.100	1383.7	1289.4	133.6	630.6	630.6	0.70	7.0	0.0166	1.48	0.58	0.1075	0.1500	0.1356	0.2597	259.7	8.9
90.70	0.100	1289.4	1166.3	122.7	497.0	497.0	0.60	6.0	0.0153	1.15	0.58	0.0573	0.0500	0.0261	0.0988	98.8	8.7
90.60	0.100	1166.3	1017.8	109.1	374.2	374.2	0.50	5.0	0.0139	0.82	0.58	0.0247	0.0000	0.0000	0.0387	38.7	8.4
90.50	0.100	1017.8	848.7	93.2	265.1	265.1	0.40	4.0	0.0124	0.49	0.58	0.0069	0.0000	0.0000	0.0193	19.3	7.6
90.40	0.100	848.7	665.4	75.5	171.9	171.9	0.30	3.0	0.0106	0.16	0.58	0.0004	0.0000	0.0000	0.0110	11.0	6.3
90.30	0.100	665.4	484.4	57.2	96.4	96.4	0.20	2.0	0.0083	0.00	0.58	0.0000	0.0000	0.0000	0.0083	8.3	4.4
90.20	0.100	484.4	305.4	39.1	39.1	39.1	0.10	1.0	0.0044	0.00	0.58	0.0000	0.0000	0.0000	0.0044	4.4	2.4
90.10	0.000	305.4	0.0	0.0	0.0	0.0	0.00	0.0	0.0000	0.00	0.58	0.0000	0.0000	0.0000	0.0000	0.0	0.0

```
********************************
** SIMULATION:25mm4hrChicago  **
********************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193      0.0287
                          0.0000     0.0022   |   0.0387      0.0396
                          0.0044     0.0061   |   0.0988      0.0519
                          0.0083     0.0118   |   0.2597      0.0653
                          0.0110     0.0194   |   0.0000      0.0000

                                AREA     QPEAK      TPEAK       R.V.
                                (ha)     (cms)      (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.021      1.92        3.31
   OUTFLOW: ID= 1 (  0040)      4.154      0.006      4.25        2.73
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00        0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 30.19
                   TIME SHIFT OF PEAK FLOW         (min)=140.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0091

--------------------------------------------------------------------------------
************************************
** SIMULATION:SCS II 12hr 100yr M **
************************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193      0.0287
                          0.0000     0.0022   |   0.0387      0.0396
                          0.0044     0.0061   |   0.0988      0.0519
                          0.0083     0.0118   |   0.2597      0.0653
                          0.0110     0.0194   |   0.0000      0.0000

                                AREA     QPEAK      TPEAK       R.V.
                                (ha)     (cms)      (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.399      6.08       46.70
   OUTFLOW: ID= 1 (  0040)      4.154      0.253      6.50       46.12
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00        0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 63.28
                   TIME SHIFT OF PEAK FLOW         (min)= 25.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0648

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```

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***********************************
** SIMULATION:SCS II 12hr 10yr M **
***********************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193      0.0287
                          0.0000     0.0022   |   0.0387      0.0396
                          0.0044     0.0061   |   0.0988      0.0519
                          0.0083     0.0118   |   0.2597      0.0653
                          0.0110     0.0194   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.209      6.08      25.53
   OUTFLOW: ID= 1 (  0040)      4.154      0.078      6.83      24.94
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 37.20
                   TIME SHIFT OF PEAK FLOW         (min)= 45.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0477

--------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 2yr M **
**********************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193      0.0287
                          0.0000     0.0022   |   0.0387      0.0396
                          0.0044     0.0061   |   0.0988      0.0519
                          0.0083     0.0118   |   0.2597      0.0653
                          0.0110     0.0194   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.086      6.17      11.23
   OUTFLOW: ID= 1 (  0040)      4.154      0.016      7.75      10.64
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 18.17
                   TIME SHIFT OF PEAK FLOW         (min)= 95.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0246

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```

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***********************************
** SIMULATION:SCS II 12hr 50yr M **
***********************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193     0.0287
                          0.0000     0.0022   |   0.0387     0.0396
                          0.0044     0.0061   |   0.0988     0.0519
                          0.0083     0.0118   |   0.2597     0.0653
                          0.0110     0.0194   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.344      6.08       40.22
   OUTFLOW: ID= 1 (  0040)      4.154      0.198      6.58       39.64
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00        0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 57.67
                   TIME SHIFT OF PEAK FLOW         (min)= 30.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0603

-------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 5yr M **
**********************************
--------------------
| RESERVOIR(  0040)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0193     0.0287
                          0.0000     0.0022   |   0.0387     0.0396
                          0.0044     0.0061   |   0.0988     0.0519
                          0.0083     0.0118   |   0.2597     0.0653
                          0.0110     0.0194   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0027)      4.154      0.156      6.17       19.36
   OUTFLOW: ID= 1 (  0040)      4.154      0.040      7.17       18.78
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00        0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 25.84
                   TIME SHIFT OF PEAK FLOW         (min)= 60.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0400

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Kollaard Associates
Engineers
210 Prescott Street, Unit 1
P.O. Box 189
Kemptville, Ontario K0G 1J0

APPENDIX D: STORMWATER MANAGEMENT MODEL

OUTLET CONTROL DESIGN SHEET - NORTH STORAGE SWALE

Client: 2873633 Ontario Inc
Job No.: 210816
Location: 819 County Road 23, Village of Merrickville Wolford, Ontario
Date: December 20, 2024

Orifice	
Dia (m):	0.100
Area (m2.):	0.0079
Orifice Coeff, C_d:	0.60
Orifice Coeff , C_w:	0.62
Orifice Top (m):	93.20
Orifice Cen (m):	93.15
Orifice Inv (m):	93.10

	Outlet Weir
	V-Notch
Weir Invert (m):	93.55
Notch Angle	40

	Overflow Channel
Bottom Channel Width (m):	1.00
Channel Invert (m):	94.20

							Orifice Flow			Weir			Overflow Channel				
Stage, WSE Elev (m)	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Volume in Swale (m^3)	Quality Storage Volume (m3)	Quantity Storage (m3)	Head (m)	n	Flow (m^3/sec)	Head (ft)	Cw	Weir Flow (m^3/sec)	Head (m)	Channel Flow (m^3/sec)	Total Outflow (m^3/sec)	Total Outflow (L/sec)	Draw Down Time (hrs)
94.30	0.100	2514.8	2414.9	246.5	1808.3	1808.3	1.20	12.0	0.0220	2.46	0.58	0.2430	0.1000	0.0923	0.3573	357.3	16.7
94.20	0.100	2414.9	2315.0	236.5	1561.8	1561.8	1.10	11.0	0.0210	2.13	0.58	0.1699	0.0000	0.0000	0.1910	191.0	16.5
94.10	0.100	2315.0	1943.1	212.6	1325.3	1325.3	1.00	10.0	0.0200	1.80	0.58	0.1119	0.0000	0.0000	0.1319	131.9	16.2
94.00	0.100	2128.7	1943.1	203.5	1112.7	1112.7	0.90	9.0	0.0190	1.48	0.58	0.0678	0.0000	0.0000	0.0867	86.7	15.7
93.90	0.100	1943.1	1758.4	185.0	909.2	909.2	0.80	8.0	0.0178	1.15	0.58	0.0362	0.0000	0.0000	0.0540	54.0	15.1
93.80	0.100	1758.4	1574.4	166.6	724.2	724.2	0.70	7.0	0.0166	0.82	0.58	0.0156	0.0000	0.0000	0.0322	32.2	14.1
93.70	0.100	1574.4	1391.3	148.2	557.6	557.6	0.60	6.0	0.0153	0.49	0.58	0.0043	0.0000	0.0000	0.0197	19.7	12.7
93.60	0.100	1391.3	1209.0	129.9	409.4	409.4	0.50	5.0	0.0139	0.16	0.58	0.0003	0.0000	0.0000	0.0142	14.2	10.6
93.50	0.100	1209.0	1027.5	111.7	279.5	279.5	0.40	4.0	0.0124	0.00	0.58	0.0000	0.0000	0.0000	0.0124	12.4	8.1
93.40	0.100	1027.5	846.9	93.6	167.8	167.8	0.30	3.0	0.0106	0.00	0.58	0.0000	0.0000	0.0000	0.0106	10.6	5.6
93.30	0.100	846.9	266.7	53.0	74.2	74.2	0.20	2.0	0.0083	0.00	0.58	0.0000	0.0000	0.0000	0.0083	8.3	3.1
93.20	0.100	266.7	163.2	21.3	21.3	21.3	0.10	1.0	0.0044	0.00	0.58	0.0000	0.0000	0.0000	0.0044	4.4	1.3
93.10	0.000	163.2	163.2		0.0	0.0	0.00	0.0	0.0000	0.00	0.58	0.0000	0.0000	0.0000	0.0000	0.0	0.0

```
********************************
** SIMULATION:25mm4hrChicago  **
********************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)     8.195     0.048      2.00       3.72
   OUTFLOW: ID= 1 (  0041)     8.195     0.011      4.33       3.71
   OVERFLOW:ID= 3 (  0003)     0.000     0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 23.25
                   TIME SHIFT OF PEAK FLOW         (min)=140.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0201
--------------------------------------------------------------------------------
************************************
** SIMULATION:SCS II 12hr 100yr M **
************************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)     8.195     0.959      6.08      48.73
   OUTFLOW: ID= 1 (  0041)     8.195     0.346      6.67      48.72
   OVERFLOW:ID= 3 (  0003)     0.000     0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 36.10
                   TIME SHIFT OF PEAK FLOW         (min)= 35.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.1787
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```
***********************************
** SIMULATION:SCS II 12hr 10yr M **
***********************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)      8.195      0.501      6.08      26.99
   OUTFLOW: ID= 1 (  0041)      8.195      0.098      7.17      26.98
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 19.64
                   TIME SHIFT OF PEAK FLOW         (min)= 65.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.1161
-------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 2yr M **
**********************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)      8.195      0.200      6.17      12.12
   OUTFLOW: ID= 1 (  0041)      8.195      0.023      9.08      12.11
   OVERFLOW:ID= 3 (  0003)      0.000      0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 11.55
                   TIME SHIFT OF PEAK FLOW         (min)=175.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0597
-------------------------------------------------------------------------------
```

```
***********************************
** SIMULATION:SCS II 12hr 50yr M **
***********************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)      8.195     0.820      6.08      42.10
   OUTFLOW: ID= 1 (  0041)      8.195     0.246      6.75      42.09
   OVERFLOW:ID= 3 (  0003)      0.000     0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 30.04
                   TIME SHIFT OF PEAK FLOW         (min)= 40.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.1638
-------------------------------------------------------------------------------
**********************************
** SIMULATION:SCS II 12hr 5yr M **
**********************************
--------------------
| RESERVOIR(  0041)|     OVERFLOW IS ON
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0322     0.0717
                          0.0044     0.0018   |   0.0540     0.0902
                          0.0083     0.0067   |   0.0867     0.1106
                          0.0106     0.0161   |   0.1319     0.1318
                          0.0124     0.0272   |   0.1910     0.1555
                          0.0142     0.0402   |   0.3573     0.1801
                          0.0197     0.0551   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0036)      8.195     0.369      6.17      20.61
   OUTFLOW: ID= 1 (  0041)      8.195     0.059      7.50      20.60
   OVERFLOW:ID= 3 (  0003)      0.000     0.000      0.00       0.00

                   TOTAL NUMBER OF SIMULATION OVERFLOW  =     0
                   CUMULATIVE TIME OF OVERFLOW  (HOURS) =  0.00
                   PERCENTAGE OF TIME OVERFLOWING   (%) =  0.00

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 15.95
                   TIME SHIFT OF PEAK FLOW         (min)= 80.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0933
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LIST OF DRAWINGS