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SERVICING DESIGN AND STORMWATER MANAGEMENT BRIEF

CAMM HEAVY MACHINERY MOVERS
WAREHOUSING FACILITY
6622 BANK STREET
OTTAWA, Ontario

Prepared For:

CAMM Warehousing and Rentals Inc.
3460 Rideau Road
Gloucester, ON K1G 3N4

PROJECT #: 170035

DISTRIBUTION
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1 INTRODUCTION

Kollaard Associates was retained by CAMM Warehousing and Rentals Inc. to complete a Site Servicing and Stormwater Management Report for a proposed industrial development to be located at 6622 Bank Street, Ottawa, Ontario.

This report will address the stormwater management requirements for the proposed Industrial Development, specifically relating to the requirements of the Shields Creek Subwatershed Study June 2004. This 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 and will identify any stormwater servicing concerns and also describe any measures to be taken during construction to minimize erosion and sedimentation. The report shall also summarize the sanitary and water requirements and proposed works that will address these requirements.

1.1 Light Industrial Development

For the purposes of this report, Bank Street is considered to be oriented along a north south axis. The proposed development site is located along the west side of Bank Street. Access to the site is by means of 2 existing access points. The main access point is located near the north side of the site, while a secondary access is located at the south side of the site. The proposed development site is approximately rectangular in shape and extends about 250 metres from Bank Street. The proposed industrial use of the site is as a storage warehouse with an auxiliary office space.

The proposed development will contain a warehouse building with a footprint of some 2310 square metres and an attached office with footprint of some 191 square metres.

The site has a total area of 6.019 hectares and was formerly cleared for agricultural purposes. Based on a review of aerial photography, the land has been fallow for several years. The land has recently been filled with a mixture of granular material and concrete rubble. The fill material extends from the front of the site to about 237 metres from Bank Street. Stormwater runoff from the site historically and currently consists of sheet flow to the west or to the roadside ditch along Bank Street.

The proposed development will be provided with private onsite services to meet the demands for water and sanitary services. Stormwater runoff will be directed by means of sheet flow to shallow swales along the north, east and south sides of the site. These swales and will be used to provide quality and quantity control and will discharge to the road side ditch along Bank Street.

2 WATER SERVICE

2.1 Domestic

The facility is to be serviced by a drilled well constructed on May 30, 2017. Information regarding the quality and quantity capabilities of this well can be found in the Hydrogeology Report prepared by Kollaard Associates, *Hydrogeological Study, 6622 Bank Street, File Number 170035*. This report also contains a copy of the Ministry of Environment Well Record.

The water usage calculations are based on domestic water requirements for the site for the proposed office space and for the loading bays associated with a warehouse. This is based on human occupants in the office space and washroom use by warehouse employees and truck drivers, etc., not on industrial water use. There is no industrial process water proposed for use as part of the development, therefore, the water use is domestic only.

The water system shall be pressurized with a submersible well pump, capable of supplying water at a flow rate of no greater than 37.8 litres/minute (10 gpm) as recommended on the Ministry of Environment Well Record. The well shall be fitted with a pitless adapter and protrude from the ground at least 400mm. A seamless 1.25" polyethylene pipe rated at 150psi shall be installed between the well and the building at a depth of at least 2.4m.

Based on Part 8 of the Ontario Building Code, the anticipated design water consumption for the proposed occupancy is up to 3,250 litres/day. The sewage design flow calculation was completed by Kollaard Associates Inc as shown in Table 2.1 below.

Table 2.1 Sanitary Design Flow Calculation

Establishment/Use	Volume	Quantity	Flow
Accessory Office Building (the greater of a or b below)			
a) per employee per 8 hr shift or	75 L/pers./day	8	600 L/day
b) per each 9.3 m^2 of flour space	75 L/9.3 m^2/day	192.1	1550 L/day
Warehouse (sum of a and b below)			
a) per water closet and	950 L/wc/day	1	950 L/day
b) per loading bay	150 L/bay/day	5	750 L/day
Total Daily Sewage Design Flow			3250 L/day

A working day can be defined as an 8 hour shift assuming that the industrial property will be used for 8 hours a day. The above sewage design flow of 3250 L/day results in a demand of 3250 L/shift or (3250 L/shift / 8 hrs/shift) = 406 L/hr or 6.8 L/minute during working hours. This flow rate is much less than suggested maximum flow rate provided on the Ministry of Environment Well record indicating that the suggested maximum flow rate will be sufficient to meet the needs of the industrial development.

2.2 Fire Water Storage

Fire water storage is required on this site as the proposed building is over 600 square metres in area. Total fire storage and requires fire flow was calculated using the Ontario Building Code (2012). Total required fire storage was calculated to be 312,112 L, as shown below.

Formulae:
$$Q = KVS_{Tot}$$
$$S_{Tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4} + ...]$$

OBC Classification of Building Use	Group, Division	F2 (OBC T-3.1.2.1)		
Assumed Type of Construction	(Most Protective Type)	Building is of limited-combustible construction. Floor assemblies are fire separations but with no fire-resistance rating. Roof assemblies, mezzanines. Load bearing walls, columns and arches do not have a fire-resistance rating. (OBC, Appendix A Table 1)		
Water Supply Coefficient (Table 1, OBC)	K	17		
Exposure Distance 1		>10	m	
Exposure Distance 2		>10	m	
Exposure Distance 3		>10	m	
Exposure Distance 4		>10	m	
Spatial Coefficient 1	Sside	0		
Spatial Coefficient 2	Sside	0		
Spatial Coefficient 3	Sside	0		
Spatial Coefficient 4	Sside	0		
Total Spatial Coefficient	Stot	1		
Average Building Height	H	7.3	m	
Building Footprint	A	2,515	sq.m	
Total Building Volume	V	18,360	cu.m	
Minimum Supply of Water	Q	312,112	L	
Required Fire Flow	Qf	9000	L/min	*per Table 2 on A-3.2.5.7 of the OBC*
		150	L/s	

Fire water storage is being provided on site in the form of three (5) storage tanks with a capacity of 65,000 L providing a total storage volume of 325,000 L.

3 SANITARY SERVICE

No municipal sanitary services are available at this site.

As per Ontario Building Code (OBC) table 8.2.1.3.B, the daily design sanitary sewage flow for the proposed occupancy is 3250 litres/day. Sanitary sewage will be disposed of in an onsite Class 4 sewage system with conventional fully raised disposal field. The onsite system will include a pressurized disposal field preceded by a pump chamber and a double chamber septic tank. The sewage effluent from the warehouse portion of the building will be directed through an oil, grease and grit separator prior to entering the septic tank. A sewage system application has been prepared for approval through the Ottawa Septic System Office. Details can be found on the septic design plan prepared by Kollaard Associates.

The septic system design has been submitted to the Ottawa Septic Office for Permit.

4 STORMWATER MANAGEMENT CONSIDERATION

4.1 Stormwater Management Design Criteria

Design of the proposed stormwater management works was completed in conformance with the City of Ottawa Sewer Design Guidelines (October 2012), the Ministry of Environment Stormwater Management Planning and Design Manual and the Shields Creek Subwatershed Study June 2004.

Stormwater management for the proposed industrial development was considered with respect to the following SWM design criteria:

- Post-development peak runoff rates will be restricted for all design storm events (2 year to 100 year inclusive) to less than or equal to the pre-development peak runoff rate for the respective storm event
- Pre-development conditions for the site are to be considered as undeveloped with a runoff coefficient of C = 0.25
- Provide an enhanced level of protection with 80 percent removal of total suspended solids for the downstream water body receiving runoff from the site in accordance with South Nation Conservation Authority requirements
- Achieve a minimum level of groundwater recharge as required by the Shield's Creek Subwatershed study
- Implement Erosion Control Measures as required to ensure development and runoff does not cause additional downstream erosion and to mitigate the potential for offsite transport of sediment during construction.

4.2 Storm Analysis Variables

4.2.1 Runoff Coefficients

The Runoff coefficient for the development was calculated as a weighted average by area. Runoff coefficients for impervious surfaces (roofs, asphalt, and concrete) were taken as 0.90, whereas pervious surfaces (grass) were taken as 0.25. The City Of Ottawa Sewer Design Guidelines allow provide a range in the allowable Runoff Coefficient of between 0.25 and 0.7 for gravel surfaced areas.

The runoff coefficient for the gravel surfaced yard area surface was taken as 0.40 based on two site visits completed during the beginning of May 2017 following significant rainfall events for the following reasons:

- At the time of the site visits, no significant runoff or evidence of significant runoff was observed;
- There was no significant ponding on the unevenly graded existing granular fill surface;
- There was no significant ponding in the roadside ditch east of the site;
- The granular and concrete rubble fill material appeared coarse graded which results in high permeability;
- The proposed granular surfaced yard area will be finished with a thin layer of Granular A or Granular B Type II material with an overall slope of less than 1 percent;

A 25% increase for the post development 100-year runoff coefficients was used as per City of Ottawa guidelines.

4.2.2 Impervious Ratio

The impervious ratio for the developed portion of the site is equal the total impervious area divided by the total developed area.

4.2.2.1 Catchment Area CA1

The total area of catchment area CA1 is 1.953 hectares. Of this area, 0.124 hectares will be occupied by pavement and roof, 0.480 hectares by gravel surfaced areas and the remainder by a granular yard and landscaped grass surface.

Impervious Ratio = 0.124/1.953 = 0.06

4.2.2.2 Catchment Area CA2

The total area of catchment area CA1 is 3.578 hectares. Of this area, 0.437 hectares will be occupied by pavement and roof, 0.238 hectares by gravel surfaced areas and the remainder by a granular yard and landscaped grass surface.

Impervious Ratio = 0.437/3.578 = 0.12

4.2.3 Time of Concentration

4.2.3.1 Pre-Development

The time of concentration for the site during pre-development conditions was calculated using a combination of the Airport formula and the Upland Method. The airport formula, developed by the U.S. Department of Transportation's Federal Aviation Administration (FAA), is more commonly used for rural development where the runoff coefficient is less than 0.40. The Uplands Method is commonly used when calculating flow velocity for shallow concentrated overland flow. It is considered that after the first 50 metres of sheet flow, the runoff will become more concentrated and flow along preferred flow channels.

Airport Formula:

$$t_{ca} = \frac{3.26\ x\ (1.1 - C)\ x\ {l_c}^{0.5}}{S^{0.33}}$$

Where C = Runoff Coefficient = 0.25
l_c = length of flow path = 50 m
S = Slope of flow path percent = 0.5 for this site.

Upland Method

$$V = K\ x\ \sqrt{S}$$
$$t_{cu} = \frac{l_c}{60\ x\ V}$$

Where K is the intercept coefficient for shallow concentrated flow and is 0.213 m/s for grass covered surfaces. For this site, the longest distance of travel for rainfall landing on the site is 200 metres. Since the first 50 metres is considered sheet flow, the remaining 150 metres will be along preferred channels. As such l_c = 150 m.

$$t_c = t_{ca} + t_{cu}$$

For this site, t*c* = 24.6 + 16.6 = 41.2 rounded to 41 minutes

Calculations are presented in Appendix A.

4.2.3.2 Post-Development

During post-development conditions, runoff will be directed by means of sheet flow to swales located along the north, south and east sides of the site. As with the pre-development time of concentration calculations, the post-development time of concentration was determined using a combination of the Airport formula and the Upland Method.

The longest distance of travel for rainfall during post development conditions is 119 metres. Since the first 50 metres is considered sheet flow, the remaining 69 metres will be along preferred channels.

$$t_c = t_{ca} + t_{cu}$$

For this site, t_c = 18.5 + 2.3 = 20.8 rounded to 20 minutes

Calculations are presented in Appendix A.

4.3 Stormwater Quantity Control

Peak Flow for runoff quantities for the Pre-Development and Post-Development stages of the project were calculated using the rational method. The rational method is a common and straightforward calculation, which assumes that the entire drainage area is subject to uniformly distributed rainfall. The formula is:

$$Q = \frac{CiA}{360}$$

Where

Q is the Peak runoff measured in ***m³/s***

C is the Runoff Coefficient, **Dimensionless**

A is the runoff area in ***hectares***

i is the storm intensity measure in ***mm/hr***

All values for intensity, i, for this project were derived from IDF curves provided by the City of Ottawa for data collected at the Ottawa International airport. For this project three return periods were considered: 2, 5 and 100-year events. The formula for each are:

2-Year Event

$$i = \frac{732.951}{(t_c + 6.199)^{0.810}}$$

5-Year Event

$$i = \frac{998.071}{(t_c + 6.053)^{0.814}}$$

100-Year Event

$$i = \frac{1735.071}{(t_c + 6.014)^{0.82}}$$

where t_c is time of concentration.

Using a pre-development time of concentration of 41 minutes, the 2-year, 5-year and 100 year storm intensities are 32.30, 43.42 mm/hr and 73.83 mm/hr respectively.

4.3.1 Pre-Development Site Conditions

As previously stated, the site is located along the west side of bank street within the Shields Creek Subwatershed. The site has a total area of about 6.019 hectares which has currently been covered with a granular and concrete rubble pad. Historically the site was fallow agricultural land. For the purposes of the stormwater management design predevelopment conditions were assumed to consist of fallow, grass and brush covered land.

Existing stormwater runoff in general consists of uncontrolled sheet flow to both the roadside ditch along the east side of the site and uncontrolled flow towards an undesignated wetland area west of the site. Grey's Creek, located south and west of the site provides outlet for the undesignated wetland area, and discharges to the Osgoode Westland Complex some 850 metres south of the site.

Pre-development site condition are summarised for the site in the following Table 2.1

Table 4.1 – Summary of Pre-Development Site Conditions

Entire Site – 6.019 hectares					
	Event Frequency	2 & 5 Year Return Period		100 year Return Period	
Surface Covering	Area of surface ha	Runoff Coef. C	C avg.	Runoff Coef. C	C avg.
Asphalt/Roof	0.000	0.9	0.25	1.0	0.31
Grass/Field	6.019	0.25		0.31	
Percent Impervious	0.0	Time of Concentration		41 min	

4.3.2 Pre-Development Runoff Rate

Using the Rational Method with the above calculated time of concentration, runoff coefficients and storm intensities, The pre-development runoff rates for the 2,5-year and 100-year storms are:

2 year = 0.25 x 32.30 x 6.019 / 360 = 0.135 m^3/s
5 year = 0.25 x 43.42 x 6.019 / 360 = 0.182 m^3/s
100 year = 0.31 x 73.83 x 6.019 / 360 = 0.383 m^3/s

4.3.3 Post-Development Site Conditions

As previously stated, the site will be developed as a light industrial warehouse site. The proposed site will contain a warehouse building with an accessory office having a combined area of 2,515 square metres. The current proposed development indicates that the parking area between the proposed building and Bank Street as well as the roadway along the south side of the building will be granular surfaced. The stormwater management design has allowed for these areas to be surfaced with asphaltic concrete as the client has indicated that these areas may be paved in the future. The majority of the remainder of the site will be surfaced with granular material. Shallow, wide, flat bottomed swales will be constructed along the north, south and east sides of the site. These swales will be the receivers for the majority of the runoff from the site.

The swales will discharge at a controlled rate to the roadside ditch along Bank Street.

4.3.4 Controlled and Uncontrolled Areas

For the purposes of this storm water management design, the site has been divided into uncontrolled and controlled areas as outlined on Kollaard drawing 170035-CA. Controlled areas are those areas from which runoff is collected and conveyed to the proposed swales for detention and quality control. Uncontrolled areas are those areas from which runoff flows directly off site along existing drainage patterns to the west of the site. The controlled areas consist of the proposed building and all of the proposed yard and parking areas, roadway areas as well as the septic system area. The uncontrolled area consists of the undeveloped area west of the proposed granular surfaced yard area.

Post-development site condition are summarised for the proposed development in the following Table 2.2

Table 4.2 – Summary of Post Development Site Conditions
Controlled and Uncontrolled Areas

Total Site Area 6.019 hectares					
	Event Frequency	5 Year Return Period		100 year Return Period	
Surface Covering	Area of surface ha	Runoff Coef. C	C avg.	Runoff Coef. C	C avg.
Controlled Area CA1 – 1.953 hectares					
Asphalt/Roof	0.124	0.9	0.45	1.0	0.55
Landscape/Pond	0.732	0.25		0.31	
Gravel Area	0.480	0.7		0.88	
Granular Yard Area	0.618	0.4		0.5	
Time of Concentration				20 min	
Controlled Area CA2 – 3.578 hectares					
Asphalt/Roof	0.437	0.9	0.46	1.0	0.56
Landscape/Pond	0.514	0.25		0.31	
Gravel Area	0.238	0.7		0.88	
Granular Yard Area	2.389	0.4		0.5	
Time of Concentration				20 min	
Uncontrolled Areas – 0.488 hectares					
Asphalt/Roof	0.000	0.9	0.25	1.0	0.31
Gravel/Granular	0.000	0.4		0.50	
Grass/Field	0.488	0.25		0.31	
Time of Concentration				20 min	
Percent Impervious				9.3 percent	

4.3.5 Allowable Release Rate

Based on the stormwater management criteria, the total allowable runoff rate from the site is equal to the pre-development runoff rate from the site for that storm event.

The allowable release rate from the proposed swales is equal to the total allowable runoff rate from the site less the runoff rate from the uncontrolled areas of the site.

As previously stated, the post-development flow rates were calculated assuming a time of concentration of 20 minutes. A time of concentration of 20 minutes yields an intensity of 52.03 mm/hr, 70.25 mm/hr and 119.95 mm/hr for the 2 year, 5 year and 100 year return periods, respectively.

Using the Rational Method the post-development flow rates from the uncontrolled areas (uncontrolled area runoff rate) are as follows:

2 year = 0.25 x 52.03 x 0.488 / 360 = 0.018 m^3/s

5 year = 0.25 x 70.25 x 0.488 / 360 = 0.024 m^3/s

100 year = 0.31 x 119.95 x 0.488 / 360 =0.051 m^3/s

The allowable release rates are calculated as follows:

Pre-development runoff rate – Uncontrolled area runoff rate = allowable release rate

2 year = 0.135 m^3/s – 0.018 m^3/s = 0.118 m^3/s

5 year = 0.182 m^3/s – 0.024 m^3/s = 0.158 m^3/s

100 year = 0.383 m^3/s – 0.051 m^3/s = 0.332 m^3/s

4.3.6 Post Development Flow Rate From Controlled Areas

The post-development flow rates from the controlled areas of the site were calculated assuming a time of concentration of 20 minutes.

Run-off from the roof areas will be directed via roof drains onto the ground surface adjacent the buildings where it will be directed by sheet flow to the swales along the north, south and east sides of the site.

Using the Rational Method the post-development flow rates from the controlled areas are as follows:

For CA1

2 year = 0.52 x 52.03 x 1.953 / 360 = 0.127 m^3/s

5 year = 0.52 x 70.25 x 1.953 / 360 = 0.172 m^3/s

100 year = 0.62 x 119.95 x 1.953 / 360 = 0.358 m^3/s

For CA2.

2 year = 0.50 x 52.03 x 3.578 / 360 = 0.238 m^3/s

5 year = 0.50 x 70.25 x 3.578 / 360 = 0.321 m^3/s

100 year = 0.60 x 119.95 x 3.578 / 360 = 0.688 m^3/s

Combined Controlled Areas

2 year = 0.365 m^3/s

5 year = 0.493 m^3/s

100 year = 1.023 m^3/s

The above calculated post-development flow rates from the controlled areas are greater than the allowable release rates. In order to meet the stormwater quantity control restriction, the controlled area runoff rate from the site cannot exceed the allowable release rate for each of the storm events. Runoff in excess of the allowable release rate will be detained and temporarily stored within the swales located on the sides of the site.

4.3.7 Proposed Storage Requirement and Outlet Control

In order to achieve the allowable controlled area storm water release rates, storm water runoff from catchment area CA1 will be directed to the swale along the north side of the site. Stormwater runoff from catchment area CA2 will be directed to the swales along the south and east sides of the site. Runoff from these swales will be discharged at a controlled rate to the road side ditch along the west side of Bank Street. The proposed swales are shown on Kollaard Associates drawing number 170035-GR *Grading and Drainage Plan*. Each swale will designed with a wide flat bottom to maximize the flow width and available storage at shallow depths. The swales are designed with a sand filter berm located 2 metres from the outlet structure. The filter berm is designed to retain and provide treatment for the first flush and for the quality control volume. During a 2 year storm event, the storage volume in front of the sand filter berm will fill. Once this available storage is occupied, the sand filter berm will be overtopped and the release rate from the storage areas will be controlled by the outlet structures. Since there is a 2 metre space between the outlet structures and the sand filter berm, the sand filter berm will not significantly affect the head on the outlet controls.

4.3.7.1 Catchment Area CA1

The proposed stormwater storage for catchment area CA1 consists of an about 150 metre long 6 metre wide flat bottomed swale which discharges by means of a 1 metre wide flat bottomed swale with a length of 52 metres to the road side ditch along bank street. The bottom of the swales have a longitudinal slope of 0.3 percent in order to maximize sedimentation and infiltration. The lowest swale elevation is 93.40 metres. The north side of the storage swale has an elevation of 94.30 metres to provide free board to the adjacent property during a 100 year storm event. The inside edge or south side of the swale adjacent the granular surfaced yard and gravel parking/roadway area ranges in elevation from 94.05 to 94.55 metres. The finished floor of the proposed building adjacent the swale ranges in elevation from 95.30 to 95.35 metres.

There is an unloading bay for transport trucks located within the building at the northwest corner of the building. The unloading bay is accessed with a reverse grade ramp sloped towards the building and has an elevation of 1.2 metres below the finished floor. The lowest elevation at the unloading bay is the exterior grade immediately adjacent the building and is 94.10 metres. Since the main finished floor of the building is everywhere above the exterior ground surface and the unloading bay is above the exterior grade at the bottom of the ramp, there is no requirement for foundation drainage. The grade at the exterior of the loading bay is positively sloped into the storage swale. It is considered that a storm event with a magnitude of greater than 100 years or a ponding level greater than that of the 100 year storm event could result in some flooding of the bottom of the unloading ramp. The unloading ramp construction has been designed to accommodate this eventuality. From Appendix A, the approximate volume of storage provided below the north edge of the swale is 752 cubic metres. The required storage volume during a 100 year storm event is 350 cubic metres which will be achieved at an elevation of 94.0 m.

4.3.7.2 Catchment Area CA2

The proposed stormwater storage for catchment area CA2 consists of an about 165 metre long 5 metre wide flat bottomed swale along the south side of the site and a 100metre long 3 metre wide flat bottomed swale along the east side of the site. The swale along the south side of the site outlets to the east swale which discharges to the road side ditch along bank street. The bottom of the swales have a longitudinal slope of 0.3 percent in order to maximize sedimentation and infiltration. The lowest swale elevation is 93.40 metres. The south side of the south storage swale has an elevation of 94.40 metres to provide free board to the adjacent property during a 100 year storm event. The east side of the east storage swale has an elevation of 94.30 metres. Overflow from this swale to the east would be to the road side ditch. The inside edges or north and west sides of the swales adjacent the granular surfaced yard range in elevation from 94.3 to 94.5 metres. These swales are not adjacent the proposed building.

From Appendix A, the approximate volume of storage provided below the east edge of the swale is 887 cubic metres. The required storage volume during a 100 year storm event is 680 cubic metres which will be achieved at an elevation of 94.15 m.

4.3.7.3 Summary

The storage volumes required in order to control the runoff rates from both catchments are summarized in the following Table

Table 4.3 – Summary of Stormwater Storage Requirements and Runoff Rates

Storm Event	2-Year	5-Year	100-Year
Total Allowable Runoff Rate (L/s)	135	182	383
Uncontrolled Area Release Rate (L/s)	18	24	51
Catchment CA1 Release Rate (L/s)	52	61	88
Catchment CA2 Release Rate (L/s)	51	82	169
Total Actual Runoff Rate (L/s)	121	167	308
Catchment CA1 Required Storage (m^3)	90	132	333
Catchment CA1 Available Storage (m^3)	753		
Catchment CA1 Ponding level (m)	93.79	93.83	94.01
Catchment CA2 Required Storage (m^3)	230	290	618
Catchment CA2 Available Storage (m^3)	868		
Catchment CA2 Ponding level (m)	93.91	93.95	94.15

These volumes are in addition to the quality storage volume calculated in the next section. Calculations of the ponding volumes and their respective elevations are provided in Appendix A.

4.3.8 Outlet Control

The restriction in the flow rate will be accomplished by an inlet control structure placed at the inlets of the outlet storm sewer pipe from each swale.

4.3.8.1 Catchment Area CA1

The inlet control will consist of one horizontal inlet pipe and two vertical risers of different diameters and heights in order to control the discharge to the ditch. Details are provided on Kollaard Associates Inc drawing: The structure consists of the following:

- 525 mm Diameter HDPE storm sewer with:
  - One horizontal 150 mm diameter inlet pipe with an invert elevation of 93.4 metres followed by,
  - One vertical 200 mm diameter riser with an inlet elevation of 93.7 metres followed by,
  - One vertical 350 mm diameter riser with an inlet elevation of 94.0 metres.

The structure will be placed on a cast-in-place concrete slab to ensure the elevation of the structure does not change. The 525 mm diameter HDPE pipe will be fitted with one 350 mm diameter vertical riser followed by one 200 mm diameter vertical riser shop welded to the top of the pipe followed by a custom end cap. The end cap will be fitted with a 150 mm diameter stub in line with the invert of the sewer.

4.3.8.2 Catchment Area CA2

The inlet control will consist of one horizontal inlet pipe and two vertical risers of different diameters and heights in order to control the discharge to the ditch. Details are provided on Kollaard Associates Inc drawing: The structure consists of the following:

- 525 mm Diameter HDPE storm sewer with:
  - One horizontal 100 mm diameter inlet pipe with an invert elevation of 93.4 metres followed by,
  - One vertical 200 mm diameter riser with an inlet elevation of 93.7 metres followed by,
  - One vertical 300 mm diameter riser with an inlet elevation of 93.9 metres.

The structure will be placed on a cast-in-place concrete slab to ensure the elevation of the structure does not change. The 525 mm diameter HDPE pipe will be fitted with one 300 mm diameter vertical riser followed by one 200 mm diameter vertical riser shop welded to the top of the pipe followed by a custom end cap. The end cap will be fitted with a 100 mm diameter stub in line with the invert of the sewer.

4.4 Stormwater Quality Design

Stormwater treatment of 80% TSS removal will be provided for by the use of a sand filter in each swale in combination with pre-treatment utilizing Best Management Practices (BMP's) including the use of low sloped swales.

Quality Control

Quality Control for each catchment will be provided by providing temporary detention of the entire volume of runoff specified in the MOE Stormwater Manual for quality control in front of a sand filter. Discharge of this quality control volume will be through the sand filter only. The runoff entering the storage swale in front of the sand filter will be pre-treated by means of sedimentation to prolong the life of the sand filter.

The MOE Stormwater Manual in section 4.6.7 under the heading Volumetric Sizing provides the following design guidance in order to calculate the quality control volume:

"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."

The water quality storage volume requirement to achieve an enhanced level of treatment using filtration was determined from the MOE Stormwater Manual Table 3.2.

4.4.1.1 Catchment Area CA1

The impervious ratio for CA1 = 0.06 From Table 3.2 by extrapolation, the storage requirement is 17.75 m^3/ha. 1.953 ha x 17.75 m^3/ha gives a total storage requirement of 34.7 m^3.

A 4 hour 15 mm design storm was entered into a Visual OTTHYMO 2.3.2 model using the catchment area of 1.953 hectares, an impervious ratio of 0.36, Manning's n of 0.25 and 0.013 for pervious and impervious areas. The model produced a total runoff volume of 0.25 mm/m^2 or 4.9 m^3.

As shown in Appendix A, there is a total storage volume for quality control purposes of 40.7 m^3 below the top of the sand filter. As such the entire quality control volume will be stored below the top of the sand filter and no by-pass or overtopping of the filter will occur below or at the peak runoff from a 4 hour 15 mm design event.

4.4.1.2 Catchment Area CA2

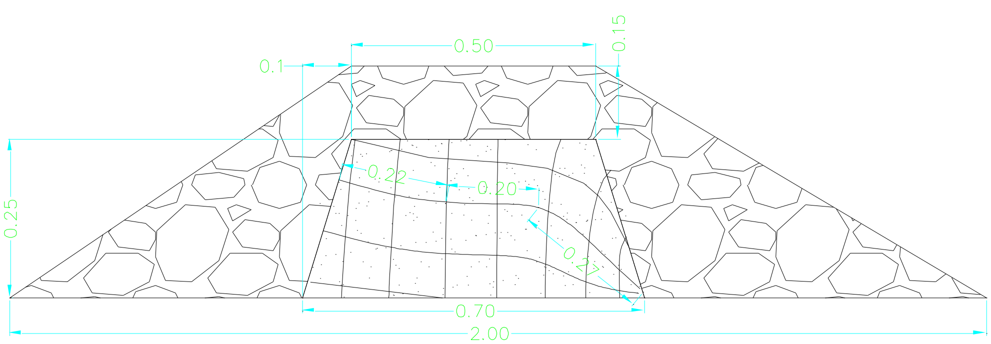
The impervious ratio for CA1 = 0.12 From Table 3.2 by extrapolation, the storage requirement is 19.25 m^3/ha. 3.578 ha x 19.25 m^3/ha gives a total storage requirement of 68.9 m^3.

A 4 hour 15 mm design storm was entered into a Visual OTTHYMO 2.3.2 model using the catchment area of 1.953 hectares, an impervious ratio of 0.36, Manning's n of 0.25 and 0.013 for pervious and impervious areas. The model produced a total runoff volume of 0.35 mm/m^2 or 12.5 m^3.

As shown in Appendix A, there is a total storage volume for quality control purposes of 70.5 m^3 below the top of the sand filter. As such the entire quality control volume will be stored below the top of the sand filter and no by-pass or overtopping of the filter will occur below or at the peak runoff from a 4 hour 15 mm design event.

Release rate through sand filter and Infiltration through bottom of storage swale

The sand filter will be placed in front of the outlet culverts and will have a depth of 0.25 metres, and length and width of 0.5m x 6.0m. The sand filter will be constructed of a medium grained sand having a percolation rate of T = 2 min/cm. According to the MOE Stormwater Manual, the seepage rate through a sand filter is to be calculated by using Darcy's Law and is equal to the projected surface area of the weir x coefficient of permeability x (hydraulic gradient across the filter). Where the hydraulic gradient was calculated as the head across the filter divided by the average length of the flow path through the filter. The average flow path length was determined by means of a flow net to be 0.69 metres as follows:



A coefficient of Permeability of 3600 mm/h was used in the Darcy Equation to represent the actual coefficient of permeability for the sand in the filter. This permeability was derived from the values given in Table 2: Approximate Relationship of Coarse grained Soil Types to Permeability and Percolation Time in the 2012 Building Code “Supplementary Standards -6: Percolation Time and Soil Descriptions”. The percolation rate “T” time of the soil to be used in the filter is 2 mins/cm. This corresponds to a coefficient of permeability of 0.1 cm/sec (or 3600 mm/h). This is based on the specified sand material to be used in the sand filter as indicated on Kollaard Associates Inc. drawing #170035 - GR.

From the geotechnical report prepared by Kollaard Associates Inc, the underlying soils consist of compact to dense glacial till with about 40 percent silt/clay and 45 percent sand sized particles. From SB-6 Table 2, the coefficient of permeability for this glacial till would be 1.0 x 10^{--7} m/sec.

The table quoted above shows the following; the fourth column has been added and is different from the quoted table.

Soil Type	*Coefficient of Permeability K – cm/sec*	*Percolation Time T – mins/cm*	*Coefficient of Permeability K – m/sec*
S.M.	*$10^{-3} – 10^{-5}$*	*8 - 20*	*$10^{-5} – 10^{-7}$*

The value provided in the table for a percolation rate (T) of 2 mins/cm is 0.1 cm/sec or 3600 mm/hr.

The flow rate through the sand filter would be:
Q = A k i
Where A = cross-sectional area of filter = 0.25*6.0 = 1.5 m^2.
k =coefficient of permeability = 1 x 10^{-3} m/s
i = hydraulic gradient = 0.25/0.69 = 0.36
Q = 5.4 x 10^{-4} m^3/s = 0.54 L/s

The flow rate through the bottom of the swales would be:
Q = A k i
Where A = surface area of the pond = 350 m^2 and 417 m^2 for CA1 and CA2 respectively.
k =coefficient of permeability = 1 x 10^{-7} m/s
Q = 8.8 x 10^{-6} m^3/s = 0.009 L/s for CA1
Q = 1.04 x 10^{-5} m^3/s = 0.01 L/s for CA2

With a combined flow rate of 0.55 L/s, the draw down time for a storage volume of 40.7 m^3 would be approximately 20.6 hours. The draw down time for a storage volume of 68.9 m^3 would be approximately 34.8 hours

The flow rate through the Rip-Rap protecting the sand filter can be calculated using the following Equation:
$Q = 0.327\, e^{1.5\,S} (g\, D_{50} / T)^{0.5}\, p\, W\, H^{1.5}$
Where: Q = Flow Rate through Rip-Rap (m^3/sec)
g = 9.806 m/sec^2
D_{50} = Mean diameter of the rock (m)
W = Width of the rock (m)
P = Porosity of the rock

T = total thickness of the rock (m)
H = Hydraulic head (m)
S = Slope of Channel (%)

Using a total thickness of rock of 2.0 – 0.7 = 1.3 and a mean rock diameter of 0.05 mm, the flow rate through the Rip-Rap at a depth of 0.2 m = 69.4 L/s. Since this is much greater than the flow rate through the sand filter, the Rip-Rap will not affect the flow rate through the sand filter.

This flow rate through the sand filter is not significant compared to the post development release rates indicated above for the 2 year, 5 year and 100 year storm events. Using this design permeability, the flow rate through the sand would be insignificant compared to the flow rate through the outlet structures.

Best Management Practices

Section 4.5.9 of the MOE Stormwater Management Planning and Design Manual (dated March 2003), discusses the use of grassed swales as a form of lot level and conveyance controls for stormwater management. This section promotes the use of shallow low gradient swales as opposed to deep narrow swales. Swales are also more effective for water quality purposes if the slope is less than 1% and the velocity less than 0.5m/s. These design aspects are incorporated into the detailed design of the development.

City of Ottawa Sewer Design Guidelines indicate that all swales with slopes of less than 1.5% must have a perforated sub-drain as per City of Ottawa Standard Detail S29. This standard detail is titled *Perforated Pipe Installation For Rear Yard and Landscaping Applications.* This detail specifies a surficial layer with a thickness of 100 mm followed by 300 mm of approved native backfill then by a clear stone drainage layer with a perforated pipe. The clear stone drainage layer has a minimum thickness of 600 mm. The perforated pipe has a diameter of 250 mm and is located a minimum of 75 mm from the bottom of the trench. This sub-drain or perforated pipe extends along the bottom of the swale to an outlet. In the case where the perforate pipe is used for rear yard drainage and landscaping purposes in an urban setting, the outlet for the perforated pipe is typically a storm sewer.

The purpose of the minimum swale slope requirement and mitigating detail where the minimum slope cannot be met due to physical limitations of a site is to ensure that there is no long term ponding within the swale. Long term ponding negatively affects vegetation and results in stagnant water leading to mosquito habitat and odor.

It is considered, however, that there is no outlet for a sub-drain at this site due to the limited elevation difference between the bottom of the storage swale and the immediate receiving body which is the roadside ditch along bank street. The bottom of the storage swale elevation is set at 93.4 metres and the existing roadside ditch elevation at the outlet location varies from

93.2 to 93.3 metres. The physical limitations of the site make the installation of a subdrain below the swales unfeasible.

In order to reduce the potential for improper drainage of the swales and the potential for surface ponding, the bottom of the swales will be lined with crushed concrete rubble. The crushed concrete will have a minimum thickness of 0.3 metres. As a result of the crushed concrete, any potential ponding within the swales will be below the ground surface.

Best Management Practices shall be implemented as follows to reduce transport of sediments and promote on site ground water recharge.

a) The storage swales will have widths of 5 metres and 6 metres and a bottom slope of 0.3 percent. The peak flow rate during a 5 year storm event into the swale for CA1 is 172 L/s and for CA2 is 321 L/s. These peak flow rates would result in a flow velocity of 0.32 m/s and a flow depth of 0.1 m for CA1 and a flow velocity of 0.41 m/s and a flow depth of 0.14 m for CA2. Since the first 0.25 metres depth of the storage swale are occupied by the quality storage, the actual flow depths will be 0.35 m and 0.39 m respectively. Since Q=VA, the actual velocity would be 0.08 m/s and 0.13 m/s. These velocities are well below the velocity at which re-suspension of settled particles will occur.

b) Minimum slope on the granular yard area and coarsely graded granular material to promote infiltration of rainwater through the yard area.

c) Discharge roof leaders to yards for natural infiltration / evaporation. Roof leaders or roof drainage will not be connected to a storm sewer system. They will discharge onto the ground adjacent to the buildings and travel through low gradient swales which will promote infiltration into the ground.

d) Servicing via swales and culverts instead of storm sewers. The drainage system for the development consists of swales and culverts (where needed) without the use of storm sewers. This will promote surface water infiltration.

The contractor shall implement BMP's to provide for protection of the area drainage system as further detailed in Section 5 of this report.

4.5 Groundwater Recharge

The Shields Creek Subwatershed Study provides guidance on Infiltration / Groundwater Protection in section 6.3.4.7. This section states that existing infiltration levels are to be maintained as part of a stormwater management plan for future development to protect the groundwater resources. Preliminary Infiltration targets have been developed and summarized in Table 6.3.2. Actual targets would be expected to have ranges on the order of 10% on either side of the specified rate in the table.

4.5.1 Infiltration Target

From the geotechnical report prepared by Kollaard Associates Inc, the underlying soils consist of compact to dense glacial till with about 40 percent silt/clay and 45 percent sand sized particles. From Table 6.3.2, a glacial till soil type would have a target infiltration rate of 50 – 100 mm/yr. Allowing 10% on either side of this results in a maximum expected infiltration rate of 110 mm/yr for the site.

4.5.2 Site Infiltration

Based on historical climate data for the Ottawa Area, there are, on average, approximately 46 days per year with a rainfall of between 5 and 10 mm, approximately 20 days per year with a rainfall of between 10 and 25 mm and 5 days per year with a rainfall of greater than 25 mm. The total average annual rainfall for the City of Ottawa Area is 733 mm.

As previously indicated, the site has been divided into three catchment areas. These consist of the uncontrolled area and catchments CA1 and CA2. The runoff coefficients for each of these areas are 0.25, 0.45 and 0.46 respectively.

The runoff coefficient (C) is a dimensionless coefficient relating the amount of runoff to the amount of precipitation received. It is a larger value for areas with low infiltration and high runoff (pavement, steep gradient), and lower for permeable, low gradient areas (landscape, coarse graded granular). As such a runoff coefficient of 0.46 indicates that 46 percent of the precipitation received will runoff the site and 54 percent of the precipitation received will infiltrate.

Based on the runoff coefficients for the site and the average annual precipitation, the expected infiltration for each area is as follows:

Catchment Area	Uncontrolled	Catchment CA1	Catchment CA2
Expected infiltration (mm)	549	403	396

The above calculated infiltration rates are all greater than the maximum expected target infiltration.

4.6 Operation and Maintenance

4.6.1 Outlet Structures and Culverts

The swales should be visually inspected on a semi-annual basis and following significant storm events. Any debris should be removed from the swales, culverts and outlet structures if present.

Removal of accumulated sediment from the swales should be conducted when the accumulation of the sediment begins to cause long term surface ponding within the swales and/or the drainage patterns along the swales are altered. The sand filter should be replaced when the drawdown time increases beyond 20% of the design value.

4.6.1.1 Catchment Area CA1

The draw down time for the proposed storage swale is about 21 hours. An increase of 20 percent would equate to a draw down time of about 25 hours. During a 5 year storm event the pond is expected to fill to about 0.43 meters above the bottom. During a 100 year storm event the pond is expected to fill to 0.61 meters above the bottom.

4.6.1.2 Catchment Area CA2

The draw down time for the proposed storage swale is about 35 hours. An increase of 20 percent would equate to a draw down time of about 42 hours. During a 5 year storm event the pond is expected to fill to about 0.55 meters above the bottom. During a 100 year storm event the pond is expected to fill to 0.75 meters above the bottom.

It is expected that observations should be made of the stormwater pond during and after significant rainfall events. If the pond appears to be significantly deeper than expected or it appears that it takes longer than expected for the water to completely leave the pond the engineer should be notified of the observations. At this point the engineer could make an assessment of the material in the upper portion of the filter. If the assessment indicates that the filter has become compromised with sediment, the filter will require maintenance.

The outer layer of the filter material (e.g., 0.1 to 0.15 m) should be removed and replaced with clear material when accumulated sediment is removed from the filter. The protective riprap may be reused if free of silt/sediment.

5 EROSION AND SEDIMENT 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 City of Ottawa and the South Nation 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 that the responsibility of implementing the sediment and erosion control measures is that of the Landowner. The Landowner is to make the contractor retained to complete the works aware of the sediment and erosion control measures and to ensure the contractor installs and maintains them.

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 sides of the property as shown in Kollaard Associates Inc. drawing #170035 – SWM/SEC – Combined Stormwater Management Plan and Sediment and Erosion Control Plan.

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 north side of the development area immediately adjacent the existing vegetated surface of the road side ditch. 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 greater than 2.0 m apart. Extra strength filter fabric may be used without a wire fence backing if posts are not greater than 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.

The exposed landscaped areas of the site should be mulched and seeded with a rapid growing grass mixture or sodded as soon as possible. The proposed asphaltic concrete surfaced areas should be surfaced as soon as possible.

The silt fences should only be removed once the site is stabilized and vegetation is established.

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

6 CONCLUSIONS

This report addresses the site servicing and stormwater management requirements for the proposed development on Bank Street. Based on the analysis provided in this report, the conclusions are as follows:

Stormwater management from a quantity control perspective for the proposed development will be achieved by restricting the runoff rate from the site by means of two outlet structures within the proposed storage swales to less than or equal to the allowable release rates.

Stormwater management from a quality control perspective will be provided by means of sedimentation within the storage swale followed by filtration of by means of a horizontal sand filter.

The infiltration of precipitation will be maintained the requirements of the Shields Creek Sub-watershed Study by using minimum gradients across the granular yard area. The granular yard area will be constructed with coarse graded granular to promote infiltration.

Water demand for the proposed development will be domestic in nature and will be provided by an onsite drilled cased well.

A fire storage tank will be provided to meet the fire fighting storage requirements for the site.

Sanitary sewage requirements will also be domestic in nature and will be addressed by means of an onsite septic system designed in accordance with Part 8 of the Ontario Building Code.

During all construction activities, erosion and sedimentation shall be controlled.

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.

Steven deWit, P.Eng.

Appendix A - Storm Design Information

- Pre-Development Runoff Rates
- Uncontrolled Area Runoff Rate
- Post-Development Flows CA1
- CA1 – Outlet Control Structure Design Sheet
- CA1 – Required Storage Vs. Release Rate
- CA1 – Discharge - Storage Curve
- CA1 – Ponding Elevation – Storage Curve

- Post-Development Flows CA2
- CA2 – Outlet Control Structure Design Sheet
- CA2 – Required Storage Vs. Release Rate
- CA2 – Discharge - Storage Curve
- CA2 – Ponding Elevation – Storage Curve

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APPENDIX A: STORMWATER MANAGEMENT MODEL

PRE DEVELOPMENT RUNOFF RATES

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

PRE DEVELOPMENT FLOW

Runoff Coefficient Equation

$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

Pre Dev run-off Coefficient "C"

			2 & 5 Year Event		100 Year Event	
Area (Ha)	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Asphalt/Roof	0.000	0.90	**0.25**	1.00	**0.31**
6.019	Gravel	0.000	0.40		0.50	
	Grass/Field	6.019	0.25		0.31	

*C value multiplied by 1.25 to a max. Of 1.00 for 100 year event

2 Year Event			
Pre Dev.	**C**	**Intensity**	**Area**
5 Year	0.25	32.30	6.019
2.78CIA= 135.11			
	135 L/s		

**Use a 41 minute time of concentration

Total Release: **135.1 L/s**

5 Year Event			
Pre Dev.	**C**	**Intensity**	**Area**
5 Year	0.25	43.42	6.019
2.78CIA= 181.63			
	182 L/s		

**Use a 41 minute time of concentration

Total Release: **181.6 L/s**

100 Year Event			
Pre Dev.	**C**	**Intensity**	**Area**
100 Year	0.31	73.83	6.019
2.78CIA= 382.98			
	383 L/s		

**Use a 41 minute time of concentration

Total Release: **383.0 L/s**

ALLOWABLE RELEASE RATE

2 Year Event	67.6
5 Year Event	181.6
100 Year Event	383.0

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APPENDIX A: STORMWATER MANAGEMENT MODEL

UNCONTROLLED AREA RUNOFF RATE

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

UA1 - UNCONTROLLED AREA

Runoff Coefficient Equation

$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area (Ha)	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Asphalt	0.000	0.90	**0.25**	1.00	**0.31**
0.488	Roof	0.000	0.90		1.00	
	Grass/Field	0.488	0.25		0.31	

Post Dev Free Flow

2 Year Event

Pre Dev.	C	Intensity	Area
2 Year	0.25	52.03	0.488
2.78CIA=	17.65		
18	L/S		

**Use a 20 minute time of concentration for 5 year

5 Year Event

Pre Dev.	C	Intensity	Area
5 Year	0.25	70.25	0.488
2.78CIA=	23.83		
24	L/S		

**Use a 20 minute time of concentration for 5 year

100 Year Event

Pre Dev.	C*	Intensity	Area
100 Year	0.31	119.95	0.488
2.78CIA=	50.45		
51	L/S		

**Use a 20 minute time of concentration for 100 year

*C value multiplied by 1.25 for 100 year event

Equations:

Flow Equation

$Q = 2.78 \times C \times I \times A$

Where:

C is the runoff coefficient

I is the intensity of rainfall, City of Ottawa IDF

A is the total drainage area

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APPENDIX A: STORMWATER MANAGEMENT MODEL
POST DEVELOPMENT FLOWS

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

CA1

Runoff Coefficient Equation
$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area (Ha)	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Roof/Asphalt	0.124	0.90	**0.45**	1.00	**0.55**
1.953	Gravel	0.480	0.70		0.88	
	Yard	0.618	0.40		0.50	
	Landscape/Swa	0.732	0.25		0.31	

Post Dev Free Flow
2 Year Event

Post Dev.	**C**	**Intensity**	**Area**
2 Year	0.45	52.03	1.953
2.78CIA= 127.13			
127 L/S			

**Use a 20 minute time of concentration for 5 year

5 Year Event

Post Dev.	**C**	**Intensity**	**Area**
5 Year	0.45	70.25	1.953
2.78CIA= 171.65			
172 L/S			

**Use a 20 minute time of concentration for 5 year

100 Year Event

Post Dev.	**C***	**Intensity**	**Area**
100 Year	0.55	119.95	1.953
2.78CIA= 358.21			
358 L/S			

**Use a 20 minute time of concentration for 100 year
*C value multiplied by 1.25 for 100 year event

Equations:
Flow Equation
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the intensity of rainfall, City of Ottawa IDF
A is the total drainage area

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APPENDIX A: STORMWATER MANAGEMENT MODEL

CA1 - OUTLET CONTROL STRUCTURE DESIGN SHEET

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

Storm Swale Characteristics

Front Section		Back Section	
Lowest pond elevation m	93.4	Lowest pond elevation m	93.6
bottom slope m/m	0.003	bottom slope m/m	0.003
bottom width m	2.0	bottom width m	6.0
bottom length m	54.0	bottom length m	151.0
Max side slope [H : V]	3	Max side slope [H : V]	3
pond depth m	1.0	pond depth m	1.0

Orifice		
Orifice 1	Dia (m):	0.150
	Area (mm.):	0.01767
	Orifice Invert:	93.400
Orifice 2	Dia (m):	0.200
	Area (mm.):	0.03142
	Orifice Invert:	93.700
Orifice 3	Dia (m):	0.350
	Area (mm.):	0.09621
	Orifice Invert:	94.000

Stage, WSE Elev (m)	Comments	Layer Thickness (m)	Front Section: Top Layer Area (m²)	Front Section: Bottom Layer Area (m²)	Front Section: Layer Volume (m³)	Back Section: Top Layer Area (m²)	Back Section: Bottom Layer Area (m²)	Back Section: Layer Volume (m³)	Total Quality Storage (m3)	Total Quantity Storage (m3)	Total Quantity Storage (ha*m)	Orifice 1 Flow: Head (m)	Orifice 1 Flow: Orifice Flow (m³/sec)	Orifice 2 Flow: Head (m)	Orifice 2 Flow: Orifice Flow (m³/sec)	Orifice 3 Flow: Head (m)	Orifice 3 Flow: Orifice Flow (m³/sec)	Total Outflow (m³/sec)
94.30		0.100	253.8	237.6	24.6	1268.4	1223.1	124.6		752.5	0.0752	0.900	0.045	0.600	0.0647	0.300	0.1401	0.249
94.20		0.100	237.6	221.4	22.9	1223.1	1177.8	120.0		603.3	0.0603	0.800	0.042	0.500	0.0590	0.200	0.1144	0.215
94.10		0.100	221.4	205.2	21.3	1177.8	1132.5	115.5		460.4	0.0460	0.700	0.039	0.400	0.0528	0.100	0.0809	0.173
94.00		0.100	205.2	189.0	19.7	1132.5	960.0	104.5		323.5	0.0324	0.600	0.036	0.300	0.0457	0.000	0.0000	0.082
93.90		0.100	189.0	172.8	18.1	960.0	690.0	82.1		199.3	0.0199	0.500	0.033	0.200	0.0373	0.000	0.0000	0.071
93.80		0.100	172.8	156.6	16.5	690.0	440.0	56.0		99.1	0.0099	0.400	0.030	0.100	0.0264	0.000	0.0000	0.056
93.70		0.050	156.6	148.5	7.6	440.0	322.5	19.0		26.6	0.0027	0.300	0.026	0.000	0.0000	0.000	0.0000	0.026
93.65		0.050	148.5	140.4	7.2	322.5	210.0	13.2	40.7	0.0	0.0000	0.250	0.023	0.000	0.0000	0.000	0.0000	0.023
93.60		0.100	140.4	76.7	10.7	210.0	0.0	7.0	20.2	0.0	0.0000	0.200	0.021	0.000	0.0000	0.000	0.0000	0.021
93.50		0.100	76.7	0.0	2.6	0.0	0.0	0.0	2.6	0.0	0.0000	0.100	0.015	0.000	0.0000	0.000	0.0000	0.015
93.40	Bottom of Swale	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		0.000	0.000	0.000	0.0000	0.000	0.0000	0.000

Orifice FLOW

$Q_{ORIFICE} = C\ A\ (2\ g\ H)^{0.5}$

where:

C = Discharge Coefficient
$Q_{ORIFICE}$ = Orifice Flow (m^3/s)
A = Orifice Area (m^2)
g = Accel due to Gravity (9.81 m/s^2)
H = Head above centre of orifice (m)

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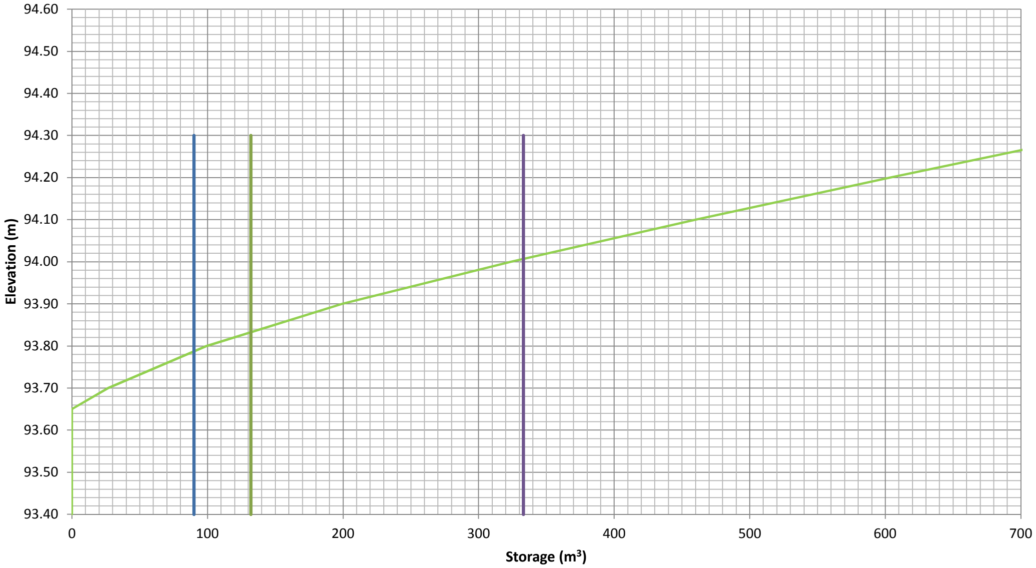
APPENDIX A: STORMWATER MANAGEMENT MODEL

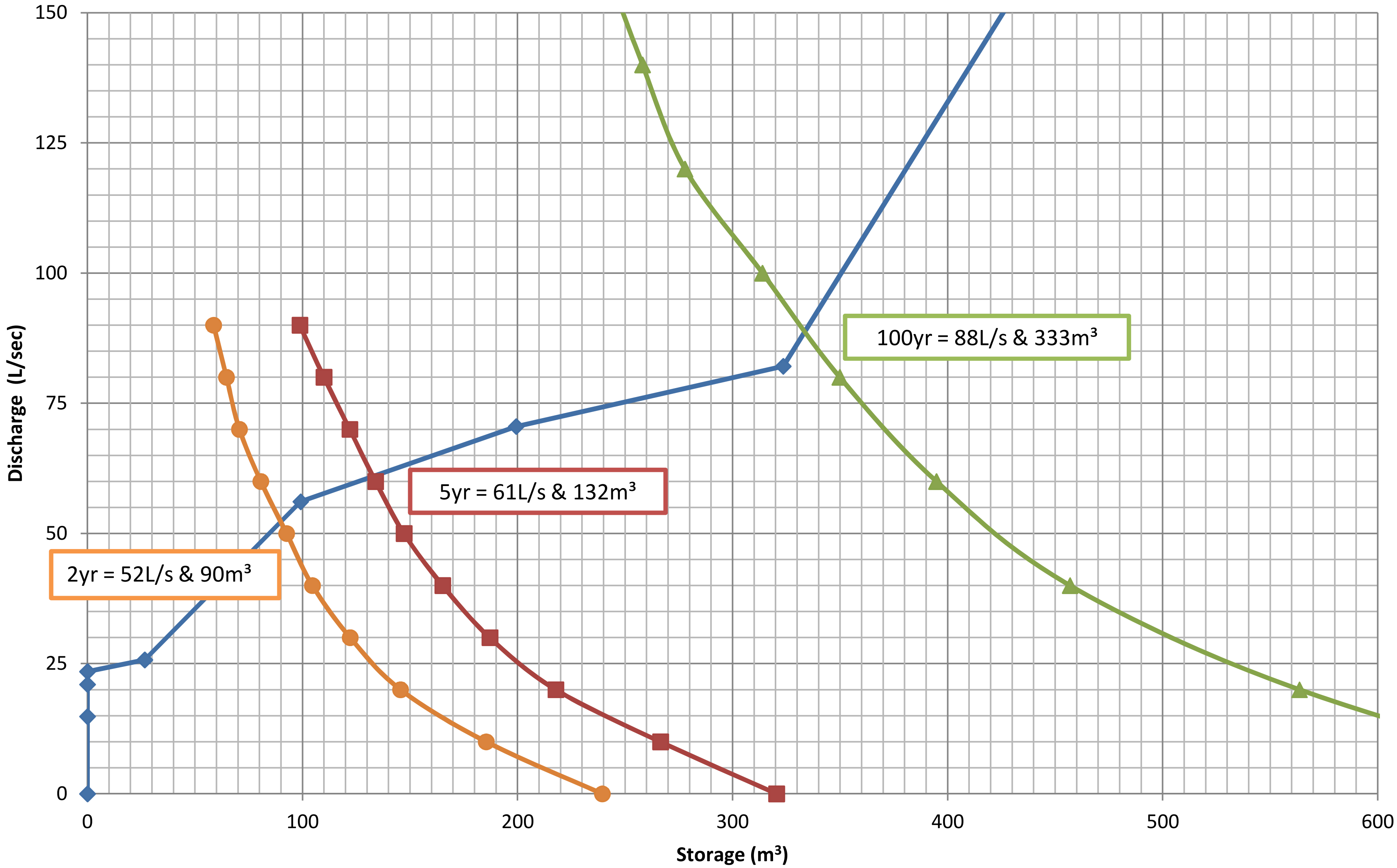
CA1 - Ponding Elevation - Storage Curve

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa
Date: July 6, 2017



APPENDIX A: STORMWATER MANAGEMENT MODEL
CA1 - Discharge-Storage Curve

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa
Date: July 6, 2017



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APPENDIX A: STORMWATER MANAGEMENT MODEL

CA1 - REQUIRED STORAGE VS. RELEASE RATE

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

CA1 Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area (Ha)	Surface	Ha	"C"	C_{avg}	"C" x 1.25	$C_{100\ avg}$
Total	Roof/Asphalt	0.124	0.90	0.45	1.00	0.55
1.953	Gravel	0.480	0.70		0.88	
	Yard	0.618	0.40		0.50	
	Land./Swales	0.732	0.25		0.31	

REQUIRED STORAGE VERSUS RELEASE RATE FOR 2 YEAR STORM

Runoff Coeffcient, C = **0.45** Duration Interval (min) = **10**
Drainage Area (ha) = **1.953** Release Rate Start (L/s) = **0**
Return Period (yrs) = **2** Release Rate Interval (L/s) = **10**

		Release Rate -->	0	10	20	30	40	50	60	70	80	90
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	Storage Required (m³)									
0	167.2	408.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	76.8	187.7	112.6	106.6	100.6	94.6	88.6	82.6	76.6	70.6	64.6	58.6
20	52.0	127.1	152.6	140.6	128.6	116.6	104.6	92.6	80.6	68.6	56.6	44.6
30	40.0	97.8	176.1	158.1	140.1	122.1	104.1	86.1	68.1	50.1	32.1	14.1
40	32.9	80.3	192.7	168.7	144.7	120.7	96.7	72.7	48.7	24.7	0.7	-23.3
50	28.0	68.5	205.5	175.5	145.5	115.5	85.5	55.5	25.5	-4.5	-34.5	-64.5
60	24.6	60.0	216.0	180.0	144.0	108.0	72.0	36.0	0.0	-36.0	-72.0	-108.0
70	21.9	53.5	224.9	182.9	140.9	98.9	56.9	14.9	-27.1	-69.1	-111.1	-153.1
80	19.8	48.5	232.6	184.6	136.6	88.6	40.6	-7.4	-55.4	-103.4	-151.4	-199.4
90	18.1	44.3	239.4	185.4	131.4	77.4	23.4	-30.6	-84.6	-138.6	-192.6	-246.6
Maximum Storage Rate =			239.4	185.4	145.5	122.1	104.6	92.6	80.6	70.6	64.6	58.6

REQUIRED STORAGE VERSUS RELEASE RATE FOR 5 YEAR STORM

Runoff Coeffcient, C = **0.45** Duration Interval (min) = **10**
Drainage Area (ha) = **1.953** Release Rate Start (L/s) = **0**
Return Period (yrs) = **5** Release Rate Interval (L/s) = **10**

		Release Rate -->	0	10	20	30	40	50	60	70	80	90
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	Storage Required (m³)									
0	230.5	563.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	104.2	254.6	152.7	146.7	140.7	134.7	128.7	122.7	116.7	110.7	104.7	98.7
20	70.3	171.6	206.0	194.0	182.0	170.0	158.0	146.0	134.0	122.0	110.0	98.0
30	53.9	131.8	237.2	219.2	201.2	183.2	165.2	147.2	129.2	111.2	93.2	75.2
40	44.2	108.0	259.1	235.1	211.1	187.1	163.1	139.1	115.1	91.1	67.1	43.1
50	37.7	92.0	276.0	246.0	216.0	186.0	156.0	126.0	96.0	66.0	36.0	6.0
60	32.9	80.5	289.8	253.8	217.8	181.8	145.8	109.8	73.8	37.8	1.8	-34.2
70	29.4	71.8	301.4	259.4	217.4	175.4	133.4	91.4	49.4	7.4	-34.6	-76.6
80	26.6	64.9	311.5	263.5	215.5	167.5	119.5	71.5	23.5	-24.5	-72.5	-120.5
90	24.3	59.3	320.5	266.5	212.5	158.5	104.5	50.5	-3.5	-57.5	-111.5	-165.5
Maximum Storage Rate =			320.5	266.5	217.8	187.1	165.2	147.2	134.0	122.0	110.0	98.7

REQUIRED STORAGE VERSUS RELEASE RATE FOR 100 YEAR STORM

Runoff Coeffcient, C = **0.55** Duration Interval (min) = **15**
Drainage Area (ha) = **1.953** Release Rate Start (L/s) = **0**
Return Period (yrs) = **100** Release Rate Interval (L/s) = **20**

		Release Rate -->	0	20	40	60	80	100	120	140	160	180
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	Storage Required (m³)									
0	398.6	1190.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	142.9	426.7	384.1	366.1	348.1	330.1	312.1	294.1	276.1	258.1	240.1	222.1
30	91.9	274.3	493.8	457.8	421.8	385.8	349.8	313.8	277.8	241.8	205.8	169.8
45	69.1	206.2	556.8	502.8	448.8	394.8	340.8	286.8	232.8	178.8	124.8	70.8
60	55.9	166.9	600.9	528.9	456.9	384.9	312.9	240.9	168.9	96.9	24.9	-47.1
75	47.3	141.1	635.0	545.0	455.0	365.0	275.0	185.0	95.0	5.0	-85.0	-175.0
90	41.1	122.8	663.0	555.0	447.0	339.0	231.0	123.0	15.0	-93.0	-201.0	-309.0
105	36.5	109.0	686.6	560.6	434.6	308.6	182.6	56.6	-69.4	-195.4	-321.4	-447.4
120	32.9	98.2	707.3	563.3	419.3	275.3	131.3	-12.7	-156.7	-300.7	-444.7	-588.7
135	30.0	89.6	725.6	563.6	401.6	239.6	77.6	-84.4	-246.4	-408.4	-570.4	-732.4
Maximum Storage Rate =			725.6	563.6	456.9	394.8	349.8	313.8	277.8	258.1	240.1	222.1

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APPENDIX A: STORMWATER MANAGEMENT MODEL
POST DEVELOPMENT FLOWS

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

CA2

Runoff Coefficient Equation
$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area (Ha)	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Roof/Asphalt	0.437	0.90	**0.46**	1.00	**0.56**
3.578	Gravel	0.238	0.70		0.88	
	Yard	2.389	0.40		0.50	
	Landscape/Sw	0.514	0.25		0.31	

Post Dev Free Flow
2 Year Event

Post Dev.	**C**	**Intensity**	**Area**
2 Year	0.46	52.03	3.578
2.78CIA=	238.06		
238	L/S		

**Use a 20 minute time of concentration for 5 year

5 Year Event

Post Dev.	**C**	**Intensity**	**Area**
5 Year	0.46	70.25	3.578
2.78CIA=	321.42		
321	L/S		

**Use a 20 minute time of concentration for 5 year

100 Year Event

Post Dev.	**C***	**Intensity**	**Area**
100 Year	0.56	119.95	3.578
2.78CIA=	668.12		
668	L/S		

**Use a 20 minute time of concentration for 100 year
*C value multiplied by 1.25 for 100 year event

Equations:
Flow Equation
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the intensity of rainfall, City of Ottawa IDF
A is the total drainage area

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

APPENDIX A: STORMWATER MANAGEMENT MODEL

CA2 - OUTLET CONTROL STRUCTURE DESIGN SHEET

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa
Date: July 6, 2017

Storm Swale Characteristics

East Section		South Section	
Lowest pond elevation m	93.4	Lowest pond elevation m	93.5
bottom slope m/m	0.003	bottom slope m/m	0.003
bottom width m	3.0	bottom width m	5.0
bottom length m	103.0	bottom length m	165.0
Max side slope [H : V]	3	Max side slope [H : V]	3
pond depth m	1.0	pond depth m	1.0

Orifice		
Orifice 1	Dia (m):	0.100
	Area (mm.):	0.00785
	Orifice Invert:	93.400
Orifice 2	Dia (m):	0.200
	Area (mm.):	0.03142
	Orifice Invert:	93.700
Orifice 3	Dia (m):	0.300
	Area (mm.):	0.07069
	Orifice Invert:	93.900

			East Section			South Section						Orifice 1 Flow		Orifice 2 Flow		Orifice 3 Flow		
Stage, WSE Elev (m)	Comments	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Layer Volume (m^3)	Top Layer Area (m²)	Bottom Layer Area (m²)	Layer Volume (m^3)	Total Quality Storage (m3)	Total Quantity Storage (m3)	Total Quantity Storage (ha*m)	Head (m)	Orifice Flow (m^3/sec)	Head (m)	Orifice Flow (m^3/sec)	Head (m)	Orifice Flow (m^3/sec)	Total Outflow (m^3/sec)
94.30		0.100	587.1	556.2	57.2	1221.0	1171.5	119.6		867.5	0.0867	0.900	0.020	0.600	0.0647	0.400	0.1188	0.203
94.20		0.100	556.2	525.3	54.1	1171.5	1122.0	114.7		690.7	0.0691	0.800	0.019	0.500	0.0590	0.300	0.1029	0.181
94.10		0.100	525.3	494.4	51.0	1122.0	1072.5	109.7		522.0	0.0522	0.700	0.017	0.400	0.0528	0.200	0.0840	0.154
94.00		0.100	494.4	463.5	47.9	1072.5	826.7	94.7		361.3	0.0361	0.600	0.016	0.300	0.0457	0.100	0.0594	0.121
93.90		0.100	463.5	432.6	44.8	826.7	590.0	70.5		218.7	0.0219	0.500	0.015	0.200	0.0373	0.000	0.0000	0.052
93.80		0.100	432.6	390.0	41.1	590.0	373.3	47.8		103.4	0.0103	0.400	0.013	0.100	0.0264	0.000	0.0000	0.040
93.70		0.020	390.0	358.4	7.5	373.3	332.4	7.1		14.5	0.0015	0.300	0.011	0.000	0.0000	0.000	0.0000	0.011
93.68		0.080	358.4	240.0	23.8	332.4	176.7	20.0	70.5	0.0	0.0000	0.280	0.011	0.000	0.0000	0.000	0.0000	0.011
93.60		0.100	240.0	110.0	17.1	176.7	0.0	5.9	26.6	0.0	0.0000	0.200	0.009	0.000	0.0000	0.000	0.0000	0.009
93.50		0.100	110.0	0.0	3.7	0.0	0.0	0.0	3.7	0.0	0.0000	0.100	0.007	0.000	0.0000	0.000	0.0000	0.007
93.40	Bottom of Swale	0.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		0.000	0.000	0.000	0.0000	0.000	0.0000	0.000

Orifice FLOW

$Q_{ORIFICE} = C A (2 g H)^{0.5}$

where:

C = Discharge Coefficient
$Q_{ORIFICE}$ = Orifice Flow (m^3/s)
A = Orifice Area (m^2)
g = Accel due to Gravity (9.81 m/s^2)
H = Head above centre of orifice (m)

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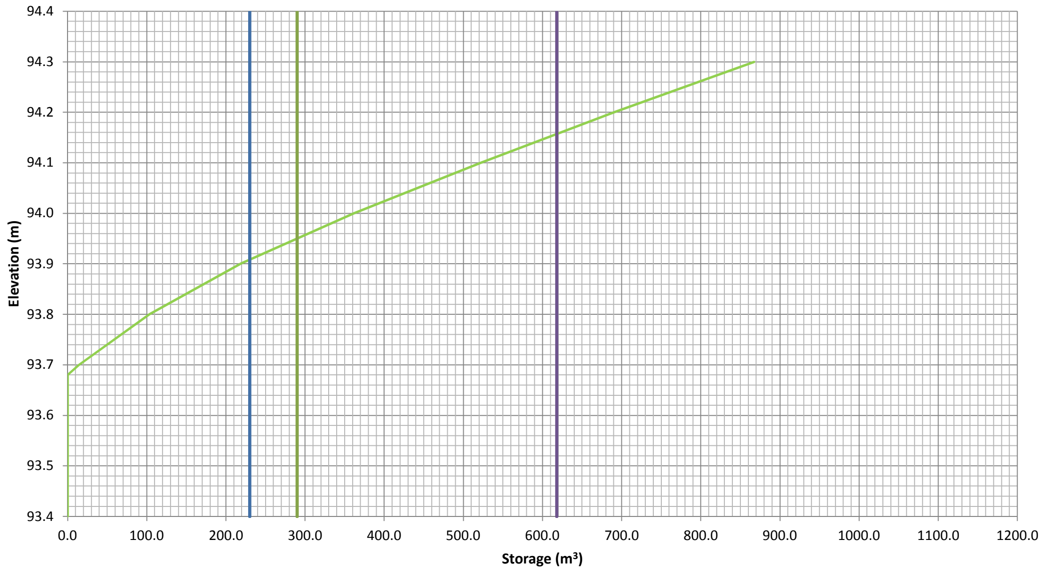
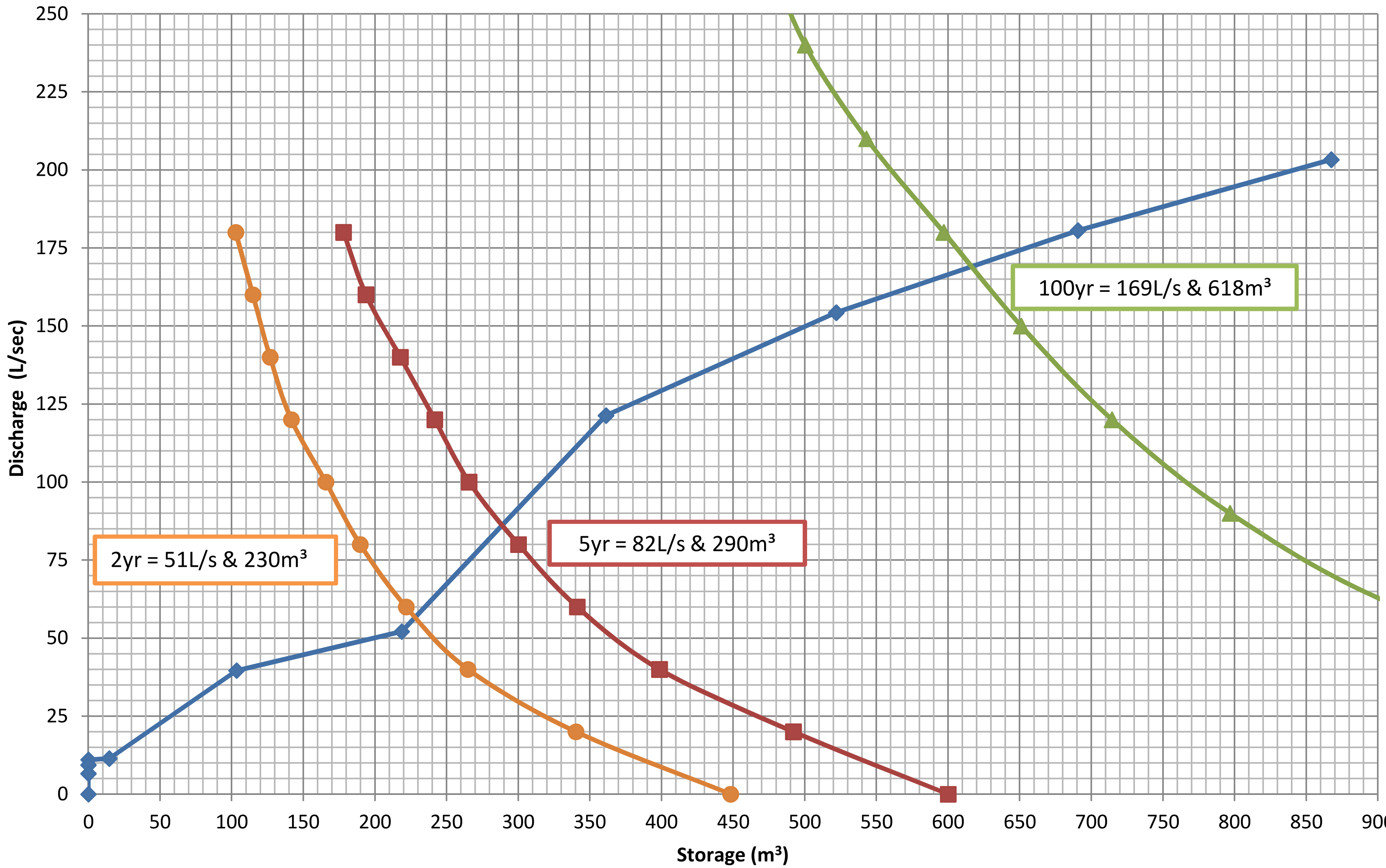
APPENDIX A: STORMWATER MANAGEMENT MODEL
CA2 - Ponding Elevation - Storage Curve

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa
Date: July 6, 2017



APPENDIX A: STORMWATER MANAGEMENT MODEL
CA2 - Discharge-Storage Curve

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa
Date: July 6, 2017



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

APPENDIX A: STORMWATER MANAGEMENT MODEL

CA2 - REQUIRED STORAGE VS. RELEASE RATE

Client: CAMM Warehousing and Rentals Inc
Job No.: 170035
Location: 6622 Bank Street, Ottawa, Ontario
Date: July 6, 2017

CA2 Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area (Ha)	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Roof/Asphalt	0.437	0.90	0.46	1.00	0.56
3.578	Gravel	0.238	0.70		0.88	
	Yard	2.389	0.40		0.50	
	Land./Swales	0.514	0.25		0.31	

REQUIRED STORAGE VERSUS RELEASE RATE FOR 2 YEAR STORM

Runoff Coeffcient, C = **0.46** Duration Interval (min) = **10**
Drainage Area (ha) = **3.578** Release Rate Start (L/s) = **0**
Return Period (yrs) = **2** Release Rate Interval (L/s) = **20**

Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	0	20	40	60	80	100	120	140	160	180
		Release Rate -->	**Storage Required (m^3)**									
0	167.2	765.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	76.8	351.4	210.8	198.8	186.8	174.8	162.8	150.8	138.8	126.8	114.8	102.8
20	52.0	238.1	285.7	261.7	237.7	213.7	189.7	165.7	141.7	117.7	93.7	69.7
30	40.0	183.2	329.8	293.8	257.8	221.8	185.8	149.8	113.8	77.8	41.8	5.8
40	32.9	150.4	360.9	312.9	264.9	216.9	168.9	120.9	72.9	24.9	-23.1	-71.1
50	28.0	128.3	384.9	324.9	264.9	204.9	144.9	84.9	24.9	-35.1	-95.1	-155.1
60	24.6	112.4	404.5	332.5	260.5	188.5	116.5	44.5	-27.5	-99.5	-171.5	-243.5
70	21.9	100.3	421.1	337.1	253.1	169.1	85.1	1.1	-82.9	-166.9	-250.9	-334.9
80	19.8	90.7	435.5	339.5	243.5	147.5	51.5	-44.5	-140.5	-236.5	-332.5	-428.5
90	18.1	83.0	448.3	340.3	232.3	124.3	16.3	-91.7	-199.7	-307.7	-415.7	-523.7
Maximum Storage Rate =			448.3	340.3	264.9	221.8	189.7	165.7	141.7	126.8	114.8	102.8

REQUIRED STORAGE VERSUS RELEASE RATE FOR 5 YEAR STORM

Runoff Coeffcient, C = **0.46** Duration Interval (min) = **10**
Drainage Area (ha) = **3.578** Release Rate Start (L/s) = **0**
Return Period (yrs) = **5** Release Rate Interval (L/s) = **20**

Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	0	20	40	60	80	100	120	140	160	180
		Release Rate -->	**Storage Required (m^3)**									
0	230.5	1054.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	104.2	476.7	286.0	274.0	262.0	250.0	238.0	226.0	214.0	202.0	190.0	178.0
20	70.3	321.4	385.7	361.7	337.7	313.7	289.7	265.7	241.7	217.7	193.7	169.7
30	53.9	246.7	444.1	408.1	372.1	336.1	300.1	264.1	228.1	192.1	156.1	120.1
40	44.2	202.2	485.2	437.2	389.2	341.2	293.2	245.2	197.2	149.2	101.2	53.2
50	37.7	172.3	516.8	456.8	396.8	336.8	276.8	216.8	156.8	96.8	36.8	-23.2
60	32.9	150.7	542.6	470.6	398.6	326.6	254.6	182.6	110.6	38.6	-33.4	-105.4
70	29.4	134.4	564.4	480.4	396.4	312.4	228.4	144.4	60.4	-23.6	-107.6	-191.6
80	26.6	121.5	583.3	487.3	391.3	295.3	199.3	103.3	7.3	-88.7	-184.7	-280.7
90	24.3	111.1	600.1	492.1	384.1	276.1	168.1	60.1	-47.9	-155.9	-263.9	-371.9
Maximum Storage Rate =			600.1	492.1	398.6	341.2	300.1	265.7	241.7	217.7	193.7	178.0

REQUIRED STORAGE VERSUS RELEASE RATE FOR 100 YEAR STORM

Runoff Coeffcient, C = **0.56** Duration Interval (min) = **15**
Drainage Area (ha) = **3.578** Release Rate Start (L/s) = **0**
Return Period (yrs) = **100** Release Rate Interval (L/s) = **30**

Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	0	30	60	90	120	150	180	210	240	270
		Release Rate -->	**Storage Required (m^3)**									
0	398.6	2220.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	142.9	795.9	716.3	689.3	662.3	635.3	608.3	581.3	554.3	527.3	500.3	473.3
30	91.9	511.7	921.1	867.1	813.1	759.1	705.1	651.1	597.1	543.1	489.1	435.1
45	69.1	384.6	1038.4	957.4	876.4	795.4	714.4	633.4	552.4	471.4	390.4	309.4
60	55.9	311.3	1120.8	1012.8	904.8	796.8	688.8	580.8	472.8	364.8	256.8	148.8
75	47.3	263.2	1184.5	1049.5	914.5	779.5	644.5	509.5	374.5	239.5	104.5	-30.5
90	41.1	229.0	1236.5	1074.5	912.5	750.5	588.5	426.5	264.5	102.5	-59.5	-221.5
105	36.5	203.3	1280.7	1091.7	902.7	713.7	524.7	335.7	146.7	-42.3	-231.3	-420.3
120	32.9	183.2	1319.2	1103.2	887.2	671.2	455.2	239.2	23.2	-192.8	-408.8	-624.8
135	30.0	167.1	1353.4	1110.4	867.4	624.4	381.4	138.4	-104.6	-347.6	-590.6	-833.6
Maximum Storage Rate =			1353.4	1110.4	914.5	796.8	714.4	651.1	597.1	543.1	500.3	473.3