PROPOSED ANIMAL HOSPITAL/ OTTAWA VALLEY WILD BIRD CARE CENTRE

SITE SERVICING AND STORM WATER MANAGEMENT REPORT
for
8520 MCARTON ROAD,
OTTAWA, ONTARIO

MAY 18, 2021

Advanced Design, Assessment & Development Inc
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ADVANCED DESIGN, ASSESSMENT & DEVELOPMENT Inc.

SITE SERVICING AND STORMWATER MANAGEMENT REPORT

for

8520 MCARTON ROAD,

OTTAWA, ONTARIO

Prepared BY

ADAD Inc.

SUBMITTED TO:

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1. INTRODUCTION

1.1 General:

The following report demonstrates, in brief, the site plan servicing and the stormwater management in support of a Site Plan Application for 8520 McArton Road, City of Ottawa, Ontario. Out of 7 hectares total area of the land, the 0.863 hectare development being proposed will potentially be the new location for the Ottawa Valley Wild Bird Centre. The development includes an animal hospital with rehabilitation centre, parking area, and all required services. Figure 1 shows summer and winter aerial views of the property.

The principal concept in the site plan servicing and the stormwater management is to integrate the site with the surrounding environment in a sustainable yet cost-effective approach. The report covers brief explanation of each service, detailed calculations in compliance with the City of Ottawa and provential requirements and by-laws for submission with the engineering drawings for City of Ottawa site plan approval.

1.2 Report Structure:

The City of Ottawa and the Ontario Ministry of the Environment Guidelines were used where applicable.

This report is divided into four sections:

i. Introduction.

ii. Site Service: includes current site condition, proposed site, fire protection, and sanitary servicing.

iii. Stormwater Management: includes design criterion, pre-development release rate, calculation of allowable release rate, calculation of post-development release rate, storage requirements, and proposed stormwater management plan.

iv. Erosion and Sediment Control.

v. Summary and Conclusions.

vi. Three Appendices:

Appendix A: Calculation of Fire Flow Requirements

Appendix B: Stormwater Management Design Sheets/ Tables.

Appendix C: Stormceptor Sizing and Maintenance Report.

1.3 References:

Various documents were referred to in preparing the current report, including:

- Ottawa Sewer Design Guidelines, City of Ottawa, SDG002, October 2012.

- Tech Bulletin ISTB-2018-02, Revision to Ottawa Design Guidelines – Water Distribution.
- Fire Underwriters Survey, Water Supply for Public Fire Protection, CGI Group Inc., 2007.
- Ontario Building Code, Ontario, https://www.ontario.ca/laws/regulation/120332
- Stormwater Management Planning and Design Manual, Ontario Ministry of the Environment (MOE), 2003.

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Summer view

Winter View

Figure 1-1 – Site Location

2. SITE SERVICES

2.1 General

2.1.1 Current Site Condition

The property has an approximate surface area of 7 ha (17 acres) located at 8520 Mcarton Road in Ottawa, Ontario. The site is currently vacant and is covered by tall grasses and shrubs, with trees being on the western and southern extent of the site. The land has an irregular rectangular shape with about 191m wide (east-west) by 365 deep (north-south). The site is mostly flat. The overall site drainage goes to the east towards the existing wetlands, with some low-lying areas lacking surficial drainage due to the topography of the site. The average site slope is approximately 3% from an elevation of +133.47 ± 0.15 meters above sea level (masl) to +131.82 ± 0.15 masl.

2.1.2 Proposed Site

It is proposed to construct an institutional building that will serve as an animal/bird hospital for local wildlife with offices and educational centre. Besides, the site will contain a parking area for employees and clients. The footprint of the building will be 623.25 m^2 with an additional 312.4 m^2 outdoor fly zone near the building on the south and west sides. There will be one entrance to access the site from McArton road. A total of 24 parking spots are required as well as a fire route running from the Mcarton Road beside the east side of the building. The site will be serviced by a private well and a conventional septic system. Refering to the approved hydrogeological report (dated September 18, 2019) regarding the (30 m) required distance between the well and the septic system; drawing SS-1 shows:

i. The distance between the well and the septic bed downgradient is 66.14 m;

ii. The distance between well and the sptic tank of the building is 33.6 m.

iii. The distance between well and the sptic tank of the Birds Aviaries is 56.2.

2.2 Water Supply and Well Water Quality

There is no municipal water available in this area of the city. The site will be serviced with water using a private well on site. The approximate location of the well was chosen on the north-west of the building (see drawing SS-1) based on the recommendations of the hydrogeological study prepared by Geofirma Engineering Ltd, dated September 18, 2018. Detailed information and hydraulic data and water quality are provided by Geofirma Engineering Ltd. final report titled “Hydrogeological Study/ 8520 McArton Road, Ottawa, Ontario”, on April 30, 2020. As related to the water quality, the report shows a omparison of the water quality results to aesthetic, analytical and indicator parameters outlined in Procedure D-5-5 and the ODWS indicate the following:

- Hardness, reported at 283 mg/L and 290 mg/L, is above the operation guideline (OG) range of 80-100 mg/L. The ODWS states that hardness values greater than 200 mg/L, but less than 500 mg/L, are considered poor but tolerable. Hardness is easily treated with standard water softener systems.

- Iron, reported at 0.4 mg/L for both samples, is just above the aesthetic objective (AO) of 0.3 mg/L. Iron concentrations up to 5 mg/L are easily treated with a water softener or manganese greensand filter.
- Laboratory reported color was 12 and 20 TCU for the 3-hour and 6-hour samples, respectively. However, color was measured at 0 TCU for both samples in the field and there was no visible color at the time of sampling. For these reasons, the elevated color reported in the lab samples is attributed to the elevated level of iron in the water, which decreased during the pump test, and is considered acceptable and treatable.
- The concentration of all other aesthetic, analytical and indicator parameters satisfy applicable criteria.

2.3 Fire Protection

A follow-up phone conversations and virtual meetings with the City of Ottawa fire protection engineer (Allan Evans) took place on February, 2021, to discuss fire fighting requirements for the site. The main driveway is widened to accommodate two fire trucks and the location of the fire hydrant is adjusted. It was confirmed that the fire department would require a volume of 10,000 US gallons to be available on-site at any given time as a reservoir to combat fires. This volume is provided by a precast concrete tank of the capacity of 40,000 liters (see Figure A-1of Appendix A for tank details). The top surface of the tank is to be at level 136.6 masl (as shown in drawing GR-1) to enable water flow by gravity from the tank to the hydrant. The container and the pipe connecting the tank to the hydrant will be heated using a geothermal system designed by the project's mechanical engineer to prevent water from freezing in the wintertime. Drawing SS-1 shows the location and elevation of the fire protection tank.

The required flow for the fire protection of the proposed site was estimated based on the Technical Bultin ISTB-2018-02, Revision to Ottawa Design Guidelines – Water Distribution. The following equation from Appendix H, Protocol to Clarify the Application of the Fire Flow Calculation Method Published by Fire Underwriters Survey (FUS), was used for calculation of the supply rates required to be supplied by the hydrant.

$F = 220 * C * \sqrt{A}$ (PP G92-G93)

Where:

F = the required fire flow in liters per minute (L/min)

C = coefficient related to the type of construction

A = the total floor area in square meters (m^2)

Table 2-1 – Summary of Required Fire Protection Flow

Item	Design Value
Floors Above Grade	1 Floor
Construction Coeficent (C)	0.8
Fire Protection Type	None
Building Height (m)	6.934 m
Building Area (m^2)	623.25 m^2
Required fire flow F (L/min)	5000
Reduction due to low Occupancy	- 25%
Increase due to Separation	0%
Fire Flow Requirement	4000 (L/min) or 66.7 (L/sec)

100 mm diameter pipe is proposed to connect the fire protection tank to the hydrant as shown in drawing SS-1. Table 6.1 of Appendix A shows the detailed calculation of the fire protection flow.

2.4 Sanitary Servicing

No sanitary sewers are servicing in this area of the city. The proposed sanitary system includes two septic tanks and two pumps, one septic tank and a pump for the building and one septic tank and a pump for the aviaries, which are connected into one leaching bed. The detailed design of the septic system will be conducted at a later stage. As per Ontario Building Code (OBC), Clause, 8.1.2.1, the septic system would consist of a conventional Class 4 leaching bed system, which its design will be reviewed and approved by the City of Ottawa at the time of construction. Refer to the servicing drawing SS-1 for the envelope of the septic system.

2.4.1 Sanitary Servicing for the Building

The building sewage demand estimated based on the maximum daily water demand. As follows:

i. The maximum number of employees is twenty (20), including six (6) practitioners; and assuming a maximum of10 cages.

ii. As per OBC, Table Table 8.2.1.3.B, Item 25, the maximum daily water demand is:

Q_{daily} = 6 X 275 + (20-6) X 75 + 10 X 75 = 3,450 leter/ day

iii. As per OBC, Clause 8.2.2.3. (1) (a), the minimum working capacity of a septic tank is:

V_{STank} = 3 X 3,450 = 10, 450 leter

iv. Use a septic tank with size of 12,500 leter for the building (see drawing SS-1 for the locations of the septic system parts).

2.4.2 Servicing for Outdoor Aviaries

In the proposed design, the Birds Aviaries have a wooden roof covering half of their area (50% of their total area). The aggregate wastewater demand results from the accumulated rainwater and the required water for washing the aviaries concrete floors and the walkway between the aviaries and the building. The rainwater from the wooden roofs is drained directly (by individual gutters) to the grass area on the west and south green zones. The design of the combined storm-and-sewage system of the Birds Aviaries based on two assumptions: (i) the events of cleaning the aviaries and the peak rain are not simultaneous; and (ii) the peak rain generates the dominant water discharge. Hence, the sewers and the septic tank designed to serve both stormwater and sewage of the cleaning water of aviaries.

Area of the Aviaries (A_{BC}) = 302 m^2

As the service life of the building in the Ontario Building Design Code is limited to 50 years, then only the rain intensity for 5 years storm is considered in this calculation.

Rain intensities for five years storm for T_c = 15 mins

Intensity, $I_5 = 998.071/(T_c+6.035)^{0.814}$ (5-year, City of Ottawa):

I_5 = 83.615 mm/hr

Total rainwater accumulated over the concentration-time of 15 mins is:

$Q = (A_{BC}/2) \times I_5 \times (T_c/60) = (302/2) \times (83.615/ 1000) \times (15/60) = 3.16\ m^3$

The peak discharge per second is = $3.16 \times 10^3 / (15 \times 60) = 3.5$ L/sec

125 mm diameter sanitary sewers are proposed with a minimum slope of 1.5 % having a Manning's full flow capacity of 3.5 L/sec. The pipes are progressively accumulating the wastewater from the aviaries floors by inlets (SWTC-A1 through STWC-A18 and STWC-B1). Three manholes are provided (SST-MH-1 through SST-MH-3).

As per the Ontario Building Code, the septic tank size for the building is 2.5 times of the wastewater demand, which is around 8000 liters. Since the majority of the wastewater is rainwater, and because of the nature of the birds' wastes, the septic tank size for the aviaries is reduced to 6,000 L (see drawing SS-1 for the locations of the septic system parts). As stated earliear, the events of cleaning the aviaries and the peak rain are not simultaneous; and the peak rain generates the dominant water discharge. The cleaning of the aviaries moves majority of the birds waste to the septic tank. The rainstorm water is then only moves some waste that occosianaly exist in-between two cleaning events. Since the water draied to the septic tank during a peak rain does not need remain the full time of 2.5 days. Based on this rational the volume of the septic tank is redused by 25%.

The background assumption here is that peaks of the two demands of the water pumped to the leaching bed (from the building septic tank and from the Birds Aviaries septic tank) are not simultaneous.

3. STORMWATER MANAGEMENT

3.1 Design Criterion

The storm flow is calculated in conformance with the latest version of the City of Ottawa Design Guidelines (October 2012). The allowable release rate for the site is limited to a 5-year storm event using a time of concentration of 10 minutes and a runoff coefficient as calculated in Table 7.1 of Appendix B. Flows in excess of the 5- year and up to the 100-year rate are detained on-site using on-site storage.

Minor Design Criteria

- The storm flow, Stormceptor inlet, and site open storage are designed and/or size based on the Modified Rational Method for the 5-year storm using a 10 minute inlet time.
- Inflow rates into the minor system are limited to an allowable release rate, as noted above.

Major Design Criteria

- The major storm flow, Stormceptor inlet, and site open storage are designed and/or size to accommodate on-site detention with sufficient capacity to attenuate the 100-year design storm. Excess runoff above the 100-year event will flow over a proposed riprap as per OPSD 810.010 (TYPE B) shown in drawing SS-1, GR-1, and WS-1.
- On-site surface storage is provided for up to the 100-year design storm by grading the area around the swale forming instantaneous pond (storage) that allows slow flow release avoiding possible local flood. Calculations of the required on-site storage volumes are supported by detailed calculations provided in Appendix B.
- Calculation of the required storage volumes has been prepared based on the Modified Rational Method, as identified in Section 8.3.10.3 of the City's Sewer Guidelines. The details of the surface storage are illustrated on the site servicing and grading plans (SS-1 and GR-1).

Water Quality: Enhanced, 80% total suspended solids (TSS) removal, quality control is required for institutional developments as per the Rideau Valley Conservation Authority requirements. Detailed information and hydraulic data and water quality are provided by Geofirma Engineering Ltd. final report titled "Hydrogeological Study/ 8520 McArton Road, Ottawa, Ontario", on April 30, 2020.

Method of Analysis: The Modified Rational Method has been used to calculate the runoff rate from the drainage catchment to quantify the detention storage for all controlled measures. Refer to Appendix B for all stormwater calculations.

The stormwater management criteria for this development is based on the City of Ottawa Sewer Design Guidelines (2012), and the Ministry of the Environment (MOE) Stormwater Management Planning and Design Manual (2003).

3.2 Pre-Development Release Rate

Although there is no requirement to control runoff to pre-development conditions, calculations of pre-development peak flows were estimated to ensure that the allowable release rate was less than pre-development conditions. Under pre-development conditions, the site consisted entirely of grass. From the existing ground elevations shown on the grading plan, storm runoff flowed westerly to the gravel lane behind the site. The pre-development runoff coefficient for the site was determined to be 0.21, with calculations shown below and in Appendix B, Table 7.1.

Using time of concentration (T_C) of 15 minutes and an average runoff coefficient of 0.21, the pre-development release rates from the site is determined for the 5-year and 100-year storms using the Rational Method as follows:

$Q_{PRE} = 2.78\ C\ I\ A$

Where:

	Q_{PRE}	=	Pre-development Peak Discharge (L/sec)	
	C_{AVG}	=	Average Runoff Coefficient	
	I	=	Average Rainfall Intensity for a return period (mm/hr)	
		=	$998.071/\ (T_C+6.053)^{0.814}$ (5-year)	
		=	$1735.688/\ (T_C+6.014)^{0.820}$ (100-year)	
	T_c	=	Time of concentration (mins)	
	A	=	Drainage Area (hectares)	
Therefore:	I_{5PRE}	=	$998.071/\ (15 + 6.053)^{0.814}$	= 83.62 mm/hr
	Q_{5PRE}	=	2.78 (0.21) (83.62 mm/hr) (0.9045 ha)	= 44.153 L/sec
	I_{100PRE}	=	$1735.688/\ (15+6.014)^{0.820}$	= 142.894 mm/hr
	Q_{100PRE}	=	2.78 (0.2625) (142.894 mm/hr) (0.9045 ha)	= 94.32 L/sec

The 5-year and 100-year pre-development flows were estimated at 43.543 L/sec and 93.015 L/sec, respectively.

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3.3 Calculation of Allowable Release Rate

The total site area is about 7ha. The only area around the proposed development is considered in this calculation as the remainder of the property towards the south will remain untouched. The total area of the site development portion is accounted for in this calculation is 0.9045 ha.

The existing site is divided into two watersheds, EWS-01 and EWS-02. EWS-01 has a total area of 0.0075 ha, and it drains towards the roadside ditch. EWS-02 accounts for the majority of the development, with an area of 0.8967 ha. This watershed drains towards the exiting wetlands located towards the southeast portion of the property.

The allowable release rate from the site development area is based on the 5-year and 100-year storm event with a pre-development runoff coefficient of 0.21 and 0.2625, respectively. The City of Ottawa Sewer Design Guidelines (2012) specifies a time of concentration of 10 mins in greenfield developments with low grades and a lack of conveyance for the major design criteria.

$Q_{\#\#\#ALLOW}$ = 2.78 C I A

Where:

	$Q_{\#\#\#ALLOW}$	=	Peak Discharge (L/sec) Runoff	
	C_{AVG}	=	Average Runoff Coefficient	
	$I_{\#\#\#ALLOW}$	=	Average Rainfall Intensity for a return period	(mm/hr)
		=	$998.071/(T_C+6.053)^{0.814}$ (5-year)	
		=	$1735.688/(T_C+6.014)^{0.820}$ (100-year)	
	T_c	=	Time of concentration (mins)	
	A	=	Drainage Area (hectares)	
Therefore:	I_{5ALLOW}	=	$998.071/(10+6.053)^{0.814}$	= 104.29 mm/hr
	Q_{5ALLOW}	=	2.78 (0.21) (83.62 mm/hr) (0.9045 ha ha)	= 55.070 L/sec
	$I_{100ALLOW}$	=	$1735.688/(10+6.014)^{0.820}$	= 178.56 mm/hr
	$Q_{100ALLOW}$	=	2.78 (0.2625) (142.894 mm/hr) (0.892 ha)	= 117.86 L/sec

Hence, the allowable release rate from the site is 55.070 L/s for the 5-year storm event and 117.86 L/s for the 100-years storm event.

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3.4 Calculation of Post-Development Release Rate

The site development area is divided into three (3) watersheds. WS-01 is a small uncontrolled area and will drain towards the roadside ditch. WS-03 will also be uncontrolled and will drain towards the wetlands by matching the existing drainage path of the property. Roof drainage will be directed towards the wide uncontrolled WS-03 using eavestroughs and downspouts as shown in drawing SS-1. WS-02 is to be controlled and will drain towards a proposed swale that will collect and convey the drainage towards an oil-grit separator (STC 750) using an inlet control device (ICD). This ICD will control the site runoff to the allowable release rate for all storm events up to and including 100- year storm event. The controlled flow will outlet to the STC and ultimately to the existing roadside ditch located north of the property. The required on-site storage will be provided through on-site surface ponding within the proposed swale shown in drawing SS-1 and GR-1. The controlled site drainage has been diverted towards the side-road ditch and away from the wetland to protect and preserve the wetland area. Refer to the Site Servicing Plan SS-1 for the proposed stormwater management layout and Watershed plan WS-1 for proposed drainage catchment areas.

Using time of concentration (T_C) of 15 minutes and an average runoff coefficients as calculated in Table 7.4, the post-development release rates from the site are determined for the three watersheds for the 5-year and 100-year storms using the Rational Method as follows:

Q_{PRE} = 2.78 C I A

Where:

	Q_{POST}	=	Post-development Peak Discharge (L/sec)	
	C_{AVG}	=	Average Runoff Coefficient	
	I	=	Average Rainfall Intensity for a return period (mm/hr)	
		=	$998.071/\ (T_C+6.053)^{0.814}$ (5-year)	
		=	$1735.688/\ (T_C+6.014)^{0.820}$ (100-year)	
	T_c	=	Time of concentration (mins)	
	A	=	Drainage Area (hectares)	
Therefore:	I_{5POST}	=	$998.071/\ (15 + 6.053)^{0.814}$	= 83.62 mm/hr
	I_{100PRE}	=	$1735.688/\ (15+6.014)^{0.820}$	= 142.894 mm/hr

Table 7.5 detail the calculations for the estimations of the 5-year and the 100-year post-development flows of the three watershed areas, while Table 3.1 summarize these flows.

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Table 3-1 – Summary of Post-Development Flows

No.	Area Name	Area (ha)	Storm: 5 Y Q_{5PRE} (L/Sec)	Storm: 100 Y Q_{100PRE} (L/Sec)
1	**EWS-01**	0.0078	1.6318	3.4858
2	**EWS-02**	**0.2594**	**28.7029**	**61.3146**
3	**EWS-03**	0.6373	42.359	90.487
Overall Site		0.9045	72.694	155.287

3.5 Storage Requirements

Comparing the post-development with the allowable release rates for the three watersheds; EWS-01, EWS-02, and EWS-02, for time concentration of 10 mins and five years and 100 years design storms, the increase in the release rate due to the development is quantified. The watersheds WS-01 and WS-03 are of uncontrolled flow release toward the wide undeveloped area, then to the wetland, where no surface storage is required. The watershed WS-02 involves a large paved zone, and hence its flow is controlled. Table 3.2 shows a summary of this comparison between the post-development and the allowable release rates. The increase of the runoff due to the development of watershed WS-02 are 19.269 L/Sec for five years design storm, and 41.24 L/Sec for 100 years design storm. Table 7.6 of Appendix B shows the detailed calculations of the excess runoff due to the proposed development.

Table 3-2 – Summary of Flows Runoff increase due to Post-Development

No.	Area Name	Area (ha)	Allowable Runoff Q_{ALL5} (L/Sec)	Allowable Runoff Q_{ALL100} (L/Sec)	P-D Increase of Runoff Q_{INC5} (L/Sec)	P-D Increase of Runoff Q_{INC100} (L/Sec)
1	**EWS-01**	0.0078	0.475	1.016	1.560	3.340
2	**EWS-02**	**0.2594**	**15.793**	**33.801**	**20.006**	**42.817**
3	**EWS-03**	0.6373	38.801	83.042	14.781	31.635
Overall Site		0.9045	55.069	117.86	34.090	72.961

The required surface storage is calculated based on maintaining the same allowable release rate for watershed WS-02 for both the five years of design storm and the 100 years design storm. Table 7.7 shows the detailed calculations for the required surface storage volume using the Modified Rational Method. The table shows that 192.69 m^3 storage volume is required for 5Y design storm and 412.4 m^3 storage volume is required for 100Y design storm.

3.6 Proposed Stormwater Management Plan

3.6.1 Proposed Runoff Flow Quantity Controls

The runoff flow of the proposed development site will be managed by two means; grading toward the wetland for around 71% of the site area; and using proposed swale and control structures for the rest 29% of the site area. WS-01 and WS-03 having a total area of 0.6332 ha, will have an open (uncontrolled) drain into the wetland following the grading, as shown in drawing GR-1. However, the drainage of watershed EWS-02 is to be managed using a storm system.

The 5-year and 100-year storm events have been analyzed. It was found that the 5-year storm governs the storm design. Hence the storm system was designed accordingly. The storm system is formed from a proposed swale, surface storage, and a control structure (see drawings SS-1, GR-1, and WS-1). The runoff will be controlled by an ICD located at the headwall at the north end of the swale. During the 5-year event, the controlled portion of the site development area will release a peak runoff rate of 15.783 L/Sec. However, the controlled area will release a peak runoff rate of 33.778 L/Sec at a maximum head of 0.65 m.

The ICD is to be installed and centered in the inlet leading to the stormceptor. The 100-year high water level (HWL) is expected to be at 134.90 masl. With the use of this ICD, and the 15.783 L/Sec release rate, 192.69 m^3 of storage volume needs to be provided on-site. The required storage will be provided as surface ponding storage in the proposed swale. The total storage volume provided is 198 m^3. The total release rate from the controlled area of the site for the 100-year storm event will be 33.778 L/s, which requires 412.4 m^3 of storage volume. The needed storage will also be provided as surface ponding storage in the proposed swale. The total storage volume provided is 416 m^3.

3.6.2 Proposed Quality Controls

Enhanced quality control providing 80% TSS removal will be accomplished with the use of a stormceptor (STC-750). The STC-750 will be located east of the development area, which will then discharge to the existing roadside ditch. Rip-rap will be used to prevent erosion at the stormceptor outlet. The STC750 will provide 84% TSS removal based on the anticipated flow rates. Therefore, on-site quality control is achievable and has been designed accordingly. Refer to Site Servicing Plan drawing SS-1 for the stormceptor location and Appendix C for the stormceptor sizing, maintenance, and technical manual-Canada.

It is noted that it will be the responsibility of the owner to ensure the adequate operation & maintenance of the stormceptor. If inspection indicates the potential need for maintenance, access is provided via the manhole lid of the Stormceptor. Maintenance is accomplished with the use of a sump-vac. Refer to Appendix C for manufacturer maintenance schedule recommendations.

4. EROSION AND SEDIMENT CONTROL

During all construction activities, erosion and sedimentation shall be controlled by the following techniques:

- Install a light-duty silt fence barrier along the perimeter of the property to capture any sediments from leading into the ditch.
- Strawbales are to be placed at the downstream end of any existing swales to act as a filtering agent.
- A visual inspection shall be completed daily on sediment control barriers and any damage will be repaired immediately. Care is to be taken to prevent damage during construction operations.
- In some cases, barriers may be removed temporarily to accommodate the construction operations. The affected barriers are to be reinstated at night when construction is completed.
- The sediment control devices are to be cleaned of accumulated silt as required. The deposits will be disposed of as per the requirements of the contract,
- During the course of construction, if ADAD Inc. performs an inspection and believes that additional prevention methods are required to control erosion and sedimentation, the contractor shall install additional silt fences or other methods as required to the satisfaction of ADAD Inc. civil team.
- Sediment control measures are to remain in place throughout the entire construction phase and monitored / maintained on a regular basis until all disturbed areas have been fully restored or vegetated.

Refer to the erosion and sediment control plan (ES-1) for more details.

5. SUMMARY & CONCLUSIONS

Based on the information presented in this report, the proposed civil engineering design ensures that stormwater management requirements for this site are achievable. The following is a summary of the stormwater management plan for this site:

- The project consists of constructing a 623.25 m^2 building along with 25 parking spots.
- The building will be serviced with a conventional septic system that will be designed and detailed at a later stage.
- The property will have water service using a drilled well.

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- 40,000 liters (>10,000 US gallons) will be provided through a precast reinforced concrete tank elevated above ground level to enable gravity water flow for fire protection.
- The allowable release rate for the site development area is 55.07 L/Sec & 117.86 L/Sec for the 5 year and 100 year storms, respectively.
- The 100-year post development flow of controlled watershed EWS-02 is to be in the same quantitative level of the 100-year pre-development rate, and the 5-year post development flow is to be in the same quantitative level of the 5- year pre-development rate.
- The runoff flow of the proposed uncontrolled watersheds EWS-01 and EWS-02 of the development site will be managed by grading mainly toward the wetland.
- Using the Modified Rational Method, the uncontrolled watersheds EWS-01 and EWS-02 will release a peak runoff rate of 4.36 L/Sec and 76.62 L/Sec, respectively; while the controlled watershed EWS-02 will be organized by an ICD to a peak rate of 33.778 L/Sec during the 100-year storm event, thereby meeting the allowable release rate.
- Using the Modified Rational Method, the uncontrolled watersheds EWS-01 and EWS-02 will release a peak runoff rate of 2.04 L/Sec and 35.80 L/Sec, respectively; while the controlled watershed EWS-02 will be organized by an ICD to a peak rate of 15.783 L/Sec during the 5-year storm event, thereby meeting the allowable release rate.
- The ICD will be located within the stormceptor inlet with an invert of 134.00 masl, producing a HWL of 134.7 masl during the 100-year storm event.
- With a controlled release rate of 15.78 L/Sec, and 33.78 L/Sec, the required storage volume capacities are 193.20 m^3 and 420.09 m^3 for 5 Y Storm and 100 Y storm, respectively. Accordingly, a total storage volume of 425 m^3 will be stored above ground as ponding within the proposed swale.
- Enhanced quality control of 80% TSS removal is required for this site. A stormceptor model STC-750, has been sized to provide 84% TSS removal thereby meeting quality control requirements.

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6. APPENDIX A: CALCULATION OF FIRE FLOW REQUIREMENTS

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Table 6-1 – Calculation of Fire Flow Requirements

The Calculations are Based on Tech Bulletin ISTB-2018-02.

1) An estimate of the Fire Flow required for a given fire area may be estimated by:

$$F = 220 * C * \sqrt{A}$$

Where:
F = the required fire flow in litres per minute (L/min)
A = the total floor area in square metres (m^2)
C = coefficient related to the type of construction (page G86, of Appendix H)

1.5	for wood construction (structure essentially combustible)
1.0	for ordinary construction (brick or other masonry walls, combustible floor and interior)
0.8	for noncombustible construction (unprotected metal structural components, masonary or metal walls)
0.6	for fire-resistive construction (fully protected frame, floors, roof)

No. of Floors =	1		
Area / Floor =	623.25	m^2	
A =	623.25	m^2	
C =	0.8		
F =	4,394	L/min	Rounded to the nearest 1000 = 5,000 L/min

2) The value obtained in (1) may be reduced by as much as 25% for occupancies having a low contents fire hazard.

Non-combustible =	-25%
Limited Combustible =	-15%
Combustible =	0%
Free Burning =	15%
Rapid Burning =	25%

Reduction due to low occupancy hazard = -25% X 5,000 = 3,750 L/min

3) The value above my be reduced by up to 50% for automatic sprinlker system (not decided).

Reduction due to automatic sprinker system = 0% X 3,750 = 3,750 L/min

4) The value obtained in (2) may be increased for structures exposed within 45 metres by the fire area under consideration..

Separation (metres)	Condtion	Charge
0m to 3.0m	1	25%
3.1m to 10.0m	2	20%
10.1m to 20.0m	3	15%
20.1m to 30.0m	4	10%
30.1m to 45.0m	5	5%
45.1m and more	6	0%

Exposure	Distance (m)	Condition	Charge
Front	6	0%	
Back	6	0%	
Side 1	6	0%	
Side 2	6	0%	

Increase due to separation = 0% X 3,750 = 0 L/min
Fire Flow Requierment is = 3,750 L/min
rounded to the nearest 1000 = 4,000 L/min Or 66.7 L/Sec

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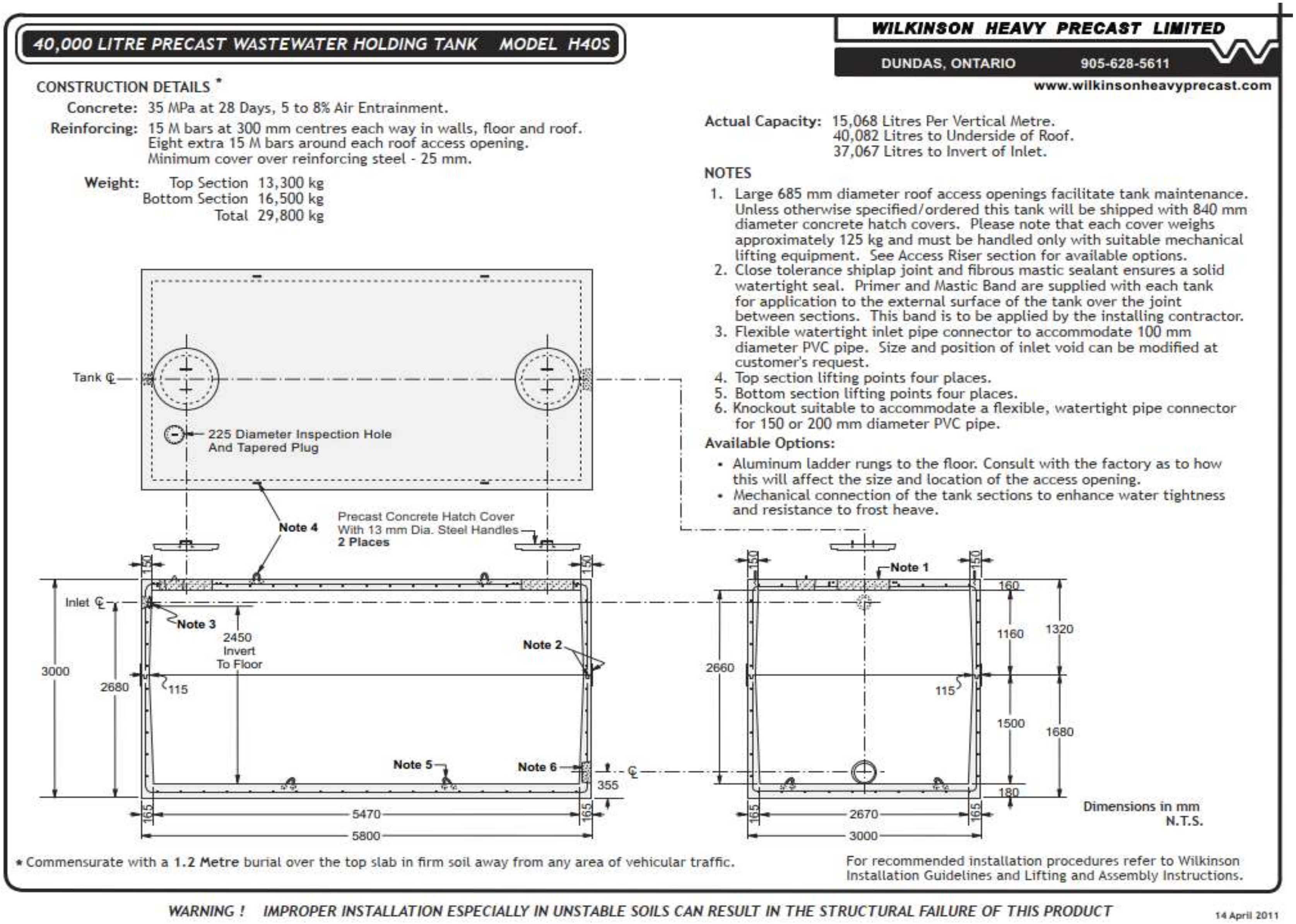

Figure 6-1 – Details of a Precast Concrete Fire Protection Tank with a 40,000 L Capacity

7. APPENDIX B: STORMWATER MANAGEMENT DESIGN SHEETS/ TABLES

Table 7-1 – Calculations of Average Runoff Coefficients (Pre-Development)

No.	Area Name	Area Type	Areas (m^2)	C_x	C_x Value	Sum of (A x Cx)
1	**EWS-01**	Asphalt/ Pavers	78	C_{ASPH}	0.9	70.2
2	**EWS-02**	Grassed	8967	C_{GRASS}	0.2	1793.4
Overall Area						**9045**
Average Runoff Coefficients (Pre- Development) C_{AVG}						**0.21**

Table 7-2 – Calculations of Peak Runoff (Pre-Development)

Area Discribtion	Area (ha)	Time of Concentration, T_c (min)	Storm: 5 Y			Storm: 100 Y		
			I_5 (mm/hr)	C_{AVG5}	Q_{5PRE} (L/Sec)	I_{100} (mm/hr)	C_{AVG100}	Q_{100PRE} (L/Sec)
Overall Site	0.9045	15	83.615	0.21	44.153	142.894	0.2625	94.32

Notes:
1) Intensity, $I = 998.071/(T_c+6.035)^{0.814}$ (5-year, City of Ottawa)
2) Intensity, $I = 1735.688/(T_c+6.014)^{0.820}$ (100-year, City of Ottawa)
3) Cavg for 100-year is increased by 25%

Table 7-3 – Calculations of Allowable Runoff (Pre-Development)

Area Description	Area (ha)	Time of Concentration, Tc (min)	Storm: 5 Y			Storm: 100 Y		
			I_5 (mm/hr)	C_{AVG5}	Q_{5ALLOW} (L/Sec)	I_{100} (mm/hr)	C_{AVG100}	$Q_{100ALLOW}$ (L/Sec)
Overall Site	0.9045	10	104.29	**0.21**	**55.070**	178.56	**0.2625**	**117.86**

Notes:
1) Allowable Capture Rate is based on 5-year storm with T_C = 10 mins
2) Intensity, I = 998.071/(Tc+6.035)0.814 (5-year, City of Ottawa)
3) Intensity, $I = 1735.688/(Tc+6.014)^{0.820}$ (100-year, City of Ottawa)
4) Cavg for 100-year is increased by 25%

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Table 7-4 – Calculations of Average Runoff Coefficients (Post-Development)

No.	Area Name	Area Type	Areas (m^2)	C_x	C_x Value	Sum of (A x Cx)
1	**EWS-01**	**Asphalt/ Pavers**	**78**	**C_{ASPH}**	**0.9**	**70.2**
2	**EWS-02**	Asphalt/ Pavers	1022.85	C_{ASPH}	0.9	920.565
	EWS-02	Grassed	1571.15	C_{GRASS}	0.2	314.23
2	**EWS-02**	**Total EWS-02**	**2594**	**$C_{AVDEWS-02}$**	**0.48**	**1234.8**
2	**EWS-03**	Roofs	623.25	C_{ROOFS}	0.9	560.925
	EWS-03	Birds Cages	159.15	C_{BCAGS}	0.9	143.235
	EWS-03	Grassed	5590.6	C_{GRASS}	0.2	1118.12
2	**EWS-03**	**Total EWS-02**	**6373**	**$C_{AVDEWS-03}$**	**0.29**	**1822.28**
					Oveall Area	**9045**
					Average Runoff Coefficients (Post-Development) $C_{AVGPost}$	**0.35**

Table 7-5 – Calculations of Peak Runoff (Post-Development)

No.	Area Name	Area (ha)	Time of Concentration Tc (min)	Storm: 5 Y			Storm: 100 Y		
				I_5 (mm/hr)	C_{AVG5}	Q_{5PRE} (L/Sec)	I_{100} (mm/hr)	C_{AVG100}	Q_{100PRE} (L/Sec)
1	**EWS-01**	0.0078	15	83.6154	0.9	1.6318	142.894	1.13	3.4858
2	**EWS-02**	**0.2594**	**15**	**83.6154**	**0.48**	**28.7029**	**142.894**	**0.595**	**61.315**
3	**EWS-03**	0.6373	15	83.6154	0.29	42.359	142.894	0.36	90.487
Overall Site		0.9045	15	83.6154	0.35	72.6938	142.894	0.431	155.287

Notes:
1) Intensity, $I = 998.071/(Tc+6.035)^{0.814}$ (5-year, City of Ottawa)
2) Intensity, $I = 1735.688/(Tc+6.014)^{0.820}$ (100-year, City of Ottawa)
3) Cavg for 100-year is increased by 25%

Table 7-6 – Calculations of Post-Development increase in Runoff Rate Based on Allowable Runoff

No.	Area Name	Area (ha)	Time of Concentration Tc (min)	Storm: 5 Y	Storm: 100 Y	Allowable Runoff		P-D Increase of Runoff	
				Q_{5POST} (L/Sec)	$Q_{100POST}$ (L/Sec)	Q_{ALL5} (L/Sec)	Q_{ALL100} (L/Sec)	Q_{INC5} (L/Sec)	Q_{INC100} (L/Sec)
1	**EWS-01**	0.0078	10	2.035	4.356	0.475	1.016	1.561	3.339
2	**EWS-02**	**0.2594**	**10**	**35.799**	**76.618**	**15.793**	**33.801**	**20.006**	**42.817**
3	**EWS-03**	0.6373	10	53.582	114.678	38.801	83.042	14.781	31.635
Overall Site		0.9045	10	89.159	190.820	55.069	117.859	34.090	72.961

Detailed Calculations of Post Development Runoff for EWS-02:

$Q_{5POST} = 2.78\ C_{AVG5POS}\ x I_5\ x\ A_{EWS-02} = 2.78 \times 0.48 \times 104.288 \times 0.2594 = 35.799$

$Q_{100POST} = 2.78\ C_{AVG100POS}\ x I_{100}\ x\ A_{EWS-02} = 2.78 \times 0.595 \times 178.559 \times 0.2594 = 76.618$

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Table 7-7 – Storage Volumes for 5-year and 100-year Return Period Storms (Modified Rational Method)

Duration, T_D (min)	Release Rate = 30 L/Sec Return Period = 5 Years Intensity, I = $998.071/(T_D+6.035)^{0.814}$ (5-year, City of Ottawa)					Release Rate = 65 L/Sec Return Period = 5 Years Intensity: I = $1735.688/(T_D+6.014)^{0.820}$ (100-year, City of Ottawa)				
	Rainfall Intensity I (mm/hr)	Peak Flow (L/sec)	Release Rate (L/sec)	Storage Rate (L/sec)	Storage (m^3)	Rainfall Intensity I (mm/hr)	Peak Flow (L/sec)	Release Rate (L/sec)	Storage Rate (L/sec)	Storage (m^3)
0	231.04	77.65	15.79	61.86	0.00	398.62	167.47	33.80	133.67	0.00
10	**104.29**	**35.05**	15.79	**19.26**	**192.59**	**178.56**	**75.02**	33.80	**41.22**	**412.16**
20	70.29	23.63	15.79	7.83	156.64	119.95	50.39	33.80	16.59	331.86
30	53.95	18.13	15.79	2.34	70.19	91.87	38.60	33.80	4.80	143.85
40	44.20	14.86	15.79	-0.94	-37.51	75.15	31.57	33.80	-2.23	-89.23
50	37.66	12.66	15.79	-3.13	-156.71	63.95	26.87	33.80	-6.93	-346.62
60	32.95	11.07	15.79	-4.72	-283.08	55.89	23.48	33.80	-10.32	-619.10
70	29.38	9.87	15.79	-5.92	-414.33	49.79	20.92	33.80	-12.88	-901.82
80	26.57	8.93	15.79	-6.86	-549.11	44.99	18.90	33.80	-14.90	-1191.9
90	24.29	8.16	15.79	-7.63	-686.55	41.11	17.27	33.80	-16.53	-1487.6
100	22.41	7.53	15.79	-8.26	-826.08	37.90	15.92	33.80	-17.88	-1787.7

Notes:
1) Peak flow is equal to the product of 2.78 C I A
2) I = $998.071/(T_D+6.035)^{0.814}$ [5-year] I = $1735.688/(T_D+6.014)^{0.820}$ [100-year] City of Ottawa. From Ottawa Sewer Design Guidelines, Section 5.4.2, where TD = storm duration (mins)
3) Release Rate = Desired Capture (Release) Rate
4) Storage Rate = Peak Flow - Release Rate
5) Storage = Duration x Storage Rate
6) Maximium storage = Max Storage Over Duration

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8. APPENDIX C: STORMCEPTOR SIZING , MAINTENANCE, AND TECHNICAL MANUAL- CANADA