Orleans Family Health Hub

Stormwater Management & Servicing Report

Type of Document:
Site Plan Application

Project Name:
Orleans Family Health Hub (OFHH)

Project Number
OTT-00240132-A0

Prepared By:
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Canada

Date Submitted
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1 Introduction

Exp Services Inc. (exp) was retained by HDR to prepare site servicing and stormwater management report or the proposed Orleans Family Health Hub. The subject site is approximately 9 hectares in area and is e located at 225 Mer Bleue Road in the City of Ottawa (Orleans), Ontario. This report outlines the servicing and stormwater management strategy for the subject property.

2 Site Overview

The proposed development is located north-east of the intersection of Mer Bleue Road and Brian Coburn Blvd. in the City of Ottawa. The site is within the Neighborhood 5 of the Mer Bleue Community Design Plan. Access to the site will be provided from both Mer Bleue Road to the West and Brian Coburn Blvd. to the south. The proposed development comprises of a healthcare building with access roads, surface parking and outdoor landscape areas.

The development has been designed in accordance with the recommendations and guidelines provided in the following documents:

- Infrastructure Servicing Brief – Chaperal Phase 4, July 19, 2012 revisions 2 (exp Services Inc.)
- Avalon West (Neighbourhood 5), Western Trunk Storm Sewer and Interim Stormwater Management Report, Revision 2, February 2012 (IBI Group)
- Avalon West (Neighbourhood 5), Stormwater Management Facility Design, Revision 2, October 2012 (IBI Group)
- Sewer Design Guidelines, Second Edition, Document SDG002, October 2012, City of Ottawa (Guidelines) including:
- Technical Bulletin ISDTB-2012-4 (20 June 2012)
- Technical Bulletin ISDTB-2014-01 (05 February 2014)
- Technical Bulletin PIEDTB-2016-01 (September 6, 2016)
- Ontario Ministry of Transportation (MTO) Drainage Manual, 1995-1997
- Stormwater Management Planning and Design Manual, Ontario Ministry of the Environment and Climate Change, March 2003 (SMPDM)

3 Water Servicing

The development will be serviced by a new 200mm diameter looped water main which will be connected to the existing 300mm diameter water main on Brian Coburn Blvd to the south and the existing 400mm diameter water main on Gerry Lalonde Drive to the east. Two additional 200mm diameter water service stubs will be provided to service the future development on the north side of the property and the future park at the south-east corner.

The building will be provided with a sprinkler system for firefighting. Sprinkler water demand provided by the Mechanical Engineer is 32 L/s. The Institutional water demands for the facility were calculated in accordance with City of Ottawa Water Distribution Design Guidelines and are as follows:

Institutional Water Demand

Average daily demand:

=50 000 l/ha/day
=9.16 ha x 50 000 L/ha/day x (1/86,400 s/day)
= 5.3 L/s

Maximum daily demand:

=1.5 x avg. day
=1.5 x 5.3 L/s
=8.0 L/s

Maximum hourly daily demand:

=1.8 x avg. day
=1.8 x 8.0 L/s
=9.5 L/s

The following boundary conditions were provided by the City of Ottawa (refer to Appendix B)

Brian Coburn Blvd. (ground Elevation = 88.34m):

Maximum HGL = 130.5m

Peak Hour HGL = 126.4 m

Max Day (8 L/s) + FireFlow (32L/s) = 127.1m

Gerry Lalonde Dr. (ground Elevation = 88.43m):

Maximum HGL = 130.4m

Peak Hour HGL = 125.7 m

Max Day (8 L/s) + Fire Flow (32L/s) = 126.8m

Based on the HGL for max day + fire flow, a pressure analysis was performed to calculate the residual pressure at the building. Based in these calculations residual pressure of 52 psi (358kPa) was estimated for the connection off Brian Coburn Blvd and 50.7 psi (349 kPa) for the connection off Gerry Lalonde Drive. Refer to Appendix B for calculations. The estimated residual water pressure is greater than the minimum requirement of 20psi (140kPa) as per the City of Ottawa Design Guidelines. Therefore, the existing municipal water supply system will have adequate capacity to meet the domestic and fire demands for the proposed development.

4 Sanitary Sewer Design

The development will be serviced by a new 200mm sanitary service which will be connected to the existing 375mm diameter sewer main on Gerry Lalonde Drive. A 200mm diameter sanitary stub will be also provided to service the future development towards the north of the property.

The design flows of the site have been determined as follows: Design Flow for

Commercial Use:	50,000 L/day/ha or 0.5787 L/s/ha
Peaking Factor:	1.5
Site Area:	9.16 hectares
Extraneous Flow:	0.28 L/s/ha
Peak Design Flow:	= (0.5787L/s/ha)(9.16ha)(1.5)+(9.16ha)(0.28L/s/ha)
	= **7.9 L/s**

The proposed on-site sanitary sewer will be installed at a minimum grade of 0.5%. At this slope, a 200 mm diameter sanitary sewer has a capacity of 23 L/s and a full flow velocity of 0.74 m/s. The proposed sanitary sewer will therefore have sufficient capacity to service the site.

The sanitary sewer system was designed to accommodate flows from future development on the property on the north side of the site. The flows from the site will ultimately drain to the Tenth Line Sanitary Trunk Sewer and Pump Station.

5 Storm Sewer Design

The proposed development is within the Avalon West Neighbourhood 5 which drains to McKinnon's Creek approximately 800m south of the site. The development will be serviced by the existing trunk sewer along Gerry Lalonde Drive. During the development of the adjacent subdivision, a 1350mm diameter storm sewer stub was installed for servicing the development. This service stub will be utilized for conveying the storm water flows from the site and the flows from the external drainage area to the north of the site.

The on site storm sewer system has been designed for ultimate build out conditions and includes the future development on the north side of the site and flows from the external drainage area comprising of a section of the future transit way and a section of the Hydro corridor to the north of the site.

6 Stormwater Management Design

An on-site storm water system has been designed to convey the storm water flows from the site to the existing 1350mm diameter storm sewer stub which is connected to the 1650mm diameter trunk sewer on Gerry Lalonde Blvd. Quantity control will be achieved by limiting the stormwater runoff to the allowable release rate determined during the Avalon West Neighbourhood 5 Western Trunk Storm Sewer Design. Flows from the external drainage on the north side of the site have been accounted for in the Stormwater Management design. Storm water flows from the site will be restricted to allowable release rates by installation of inlet control devices (ICDs) and using flow control roof drains. Storm water quality control will be provided by the Neighbourhood 5 SWM facility. A 900mm diameter storm sewer will be provided to service the future development on the property towards the north side of the site. A 375mm diameter stub will be provided to service the future park on the south-east side of the property.

The site grading has been designed to limit the maximum ponding under 100-year storm events to 300mm. Overland flow paths have been provided to convey the excess run off towards the municipal roads. Overland flow from the parking lot on the north side will sheet drain towards Mer Bleue Road. Overland flow from the rest of the current phase of development will sheet drain towards Brian Coburn Boulevard. Overland flows from the areas earmarked for future expansion of the Facility and from the external drainage area to the north will sheet drain towards Gerry Lalnonde Drive through the existing easement. Until such time the future development area is fully developed, there will be minimal or no overland flow from it. Following the development of this area, overland flow will be directed towards Brian Coburn Boulevard.

6.1 Allowable Release Rate

The allowable release rate for the site was established during the design of Avalon West (Neighborhood 5) Western Trunk Sewer and Stormwater Management Facility. Reference Table 4.4 and Figure 4 of the IBI report (Revision 2, 2012) included in Appendix A. The release rate for the OFHH site in IBI's report for up to the 100-year events is **778.6 L/s.** This release rate has been used for the SWM design of OFHH.

6.2 Design Criteria

The storm sewer system was designed in conformance with the latest version of the City of Ottawa Design Guidelines (October 2012).

Minor System Design Criteria

- The storm sewers and service laterals have been designed and sized based on the Rational Method and the Manning's Equation under free flow conditions for the 5-year storm using a 10-minute inlet time.
- Inflow rates into the minor system are limited to the allowable release rates determined in the Avalon West (Neighborhood 5) IBI Group Western Trunk Sewer and Stormwater Management Facility reports.

Major System Design Criteria

- The major system has been designed to accommodate onsite detention with sufficient capacity to attenuate the 100-year design storm. Excess runoff above the 100-year event will flow overland offsite.
- Onsite storage is provided for up to the 100-year design storm with maximum allowable ponding depth of 300mm on the ground surface and 150mm on the building roof. Calculations for the required onsite storage volumes are provided in Appendix A.
- Calculations of the required storage volumes have been prepared based on the Modified Rational Method as identified in Section 8.3.10.3 of the City's Sewer Guidelines. The depth and extent of surface storage is illustrated on the Site Servicing and Grading plan.

6.3 Runoff Coefficients

Runoff coefficients used for post-development conditions were based on actual areas measured in AutoCAD. Runoff coefficients for impervious surfaces (roofs, asphalt, and concrete) were taken as 0.90, whereas pervious surfaces (grass/landscaping) were taken as 0.20. Runoff coefficients were increased by 25% during the 100-year storm event, with a maximum of 1.0.

6.4 Calculation of Post-Development Runoff

Refer to the storm drainage plan drawing C003 for the post-development storm drainage areas. There will be small portions of the site that will be subject to free flow conditions due to site constraints and for matching existing grades at the perimeter of the site. There will be some uncontrolled flow from the western side of the site towards the Mer Bleue ROW and from the southern edge of the site towards the Brian Coburn Blvd ROW. The drainage from the loading dock area will also flow uncontrolled into the storm sewer network.

Based on the storm drainage areas presented on the storm drainage plan drawing C002, the 5-year and 100-year post-development peak flows are calculated based on the Rational Method and are summarized in the Table 6.1 below with detailed calculations provided in Appendix A.

Table 6.1: Summary of Post-Development Flows

Area No	Outlet Location	Area (ha)	Storm = 5-year			Storm = 100-year		
			C_{AVG}	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	Q (L/sec)	Q_{CAP} (L/sec)
Post-1	Roof	0.79	0.90	206.1	**23.0**	1.00	392.2	**23.0**
Post-2	Parking/Roads	2.89	0.80	673.7	**104.2**	1.00	1434.1	**104.2**
Post-3	Future Dev Area	5.20	0.20	301.5	301.5	0.25	645.3	**502.1**
Post-4	ROW Free Flow	0.21	0.34	20.6	20.6	0.42	44.0	44.0
Post-5	Loading Dock	0.07	0.90	18.3	18.3	1.00	34.7	34.7
Total		9.16		1220.2	**467.5**		2550.3	**708**

Flows in **bold** under Q_{CAP} denotes flows that are controlled.

Flow control devices will be used to restrict these runoff rates from the site to 708 L/sec up to the 100-year events which is less than the allowable 778.6 L/s. Further details regarding the onsite detention and storage methods are provided in the proceeding section.

6.5 Flow Control Method

It will be necessary to control runoff to the allowable rate; therefore, runoff will be detained using inlet control devices (ICDs) within the storm system as well as flow control roof drains. The following Table 6.2 summarizes the ICDs that are proposed.

Table 6.2: Summary of ICDs

Area No	ICD Location	Controlled Rate (L/sec)	Orifice Centre Elev. (m)	Max Elev. (m)	Head (m)	ICD Type / Model
Post-2	CBMH221	104.2	85.34	88.70	3.36	165mm diameter Plug Type Orifice
Post-3	CBMH226	502.1	85.04	88.00	2.96	375mm diameter Plug Type Orifice
Post-5	Roof Drains	23	N/A	N/A	0.15	By Mechanical

The discharge rate for the two ICDs was calculated based on the Orifice Equation, assuming it was fully submerged, as follows:

$Q_{ORF} = C * A * \sqrt{(2gH)}$

where:

Q_{ORF}	=	Flow through orifice, m^3/sec
C	=	Discharge Coefficient [0.61]
A	=	Area of orifice (m^2)
g	=	Acceleration due to gravity, m/sec^2 [9.81]
H	=	Head above centerline of orifice, m

6.6 Storage Requirements

Stormwater storage requirements and associated controlled release rates within the site are summarized below in Table 6.3. Detailed calculations using the Modified Rational Method of the onsite storage requirements are provided in Appendix A.

Table 6.3: Summary of Storage Requirements and Release Rates

Area No	Controlled Rate (L/sec)	Storage Volume Required (m^3)	Storage Volume Provided (m^3)
Post-2	104.2	1242.6	1287.9
Post-3	502.1	112.5	683.6
Post-5	23	363.9	375

The storage provided on the surface areas were estimated using the prism formula as follows:

$$V = 1/3 \times A \times d$$

where:

V = storage volume (cu.m.)
A = storage area (sq.m.)
d = maximum storage depth (m)

The depth is the difference in elevation between the low point elevation and the maximum water level. Refer to Appendix A for storage volume calculation details.

6.7 City Stress Test

As per Technical Bulletin ISDTB-2012-1 issued by the City of Ottawa, it is now a requirement that all drainage systems be stress tested using design storms calculated on the basis of a 20% increase of the City's IDF curves rainfall values. Modifications to the drainage system would be required if severe flooding to properties is identified.

As indicated previously, stormwater is to be stored on site for storms up to and including the 100 year storm events. An increase of 20% of the 100 year storm event will result in excess water following the overland flow route and spilling onto the City Right-of-Way before impacting the building. The building finished floor elevation has been established to maintain a minimum of 300mm freeboard from the spill elevation for each ponding area to the Finished Floor Elevation of for 100 year and larger storm events.

6.8 Stormwater Quality Control

Water quality control requirements for the Neighborhood 5 development area was determined in the approved "Mer Bleue Community Design Plan – Infrastructure Servicing Study (2006 CDP). Stormwater quality control for the proposed health hub will be provided by the existing stormwater management facility designed to provide 80% TSS removal for Neighborhood 5. Therefore, no on-site water quality control measures will be provided.

7 Erosion and Sediment Control

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

- extent of exposed soils shall be limited at any given time,
- exposed areas shall be re-vegetated as soon as possible,
- filter cloth shall be installed between frame and cover of all new catch basins and catch basin manholes,
- filter cloth shall be installed between frame and cover of the existing catch basins and catch basin manholes as identified on the site grading and erosion control plan,
- light duty silt fencing will be used to control runoff around the construction area. Silt fencing locations are identified on the site grading and erosion control plan.
- visual inspection shall be completed daily on sediment control barriers and any damage repaired immediately. Care will be taken to prevent damage during construction operations,
- In some cases barriers may be removed temporarily to accommodate the construction operations. The affected barriers will be reinstated at night when construction is completed,
- Sediment control devices will 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 the engineer believes that additional prevention methods are required to control erosion and sedimentation, the contractor will install additional silt fences or other methods as required to the satisfaction of the engineer, and
- Construction and maintenance requirements for erosion and sediment controls are to comply with Ontario Provincial Standard Specification (OPSS) OPSS 805, and City of Ottawa specifications.

8 Conclusions

The following conclusions are provided:

- A 200mm diameter looped water service connected to the 300mm diameter municipal watermain on Brian Coburn Blvd and the 400mm diameter watermain on Gerry Lalonde Drive will adequately service the proposed development.
- A fire Hydrant will be installed to provide fire protection for the proposed development.
- A 200mm diameter sanitary service connected to the municipal sanitary sewer system will adequately service the proposed building.
- SWM for the proposed development will be achieved by restricting all storms up to the 100 year post development flows to the allowable release rate determined during the design of Avalon West (Neighborhood 5) Western Trunk Sewer and Stormwater Management Facility.
- Onsite stormwater storage is provided up to the 100-year design storms with maximum allowable ponding depths of 300mm on the ground surface and 150mm on the roof.
- Quality control of stormwater will be provided by the existing Stormwater Management Facilty for Neighbourhood 5 to provide 80% TSS removal
- Temporary erosion and sediment control measures for the subject site have been identified.
- Overland flow routes have been provided for the subject site.

Appendix A –

SWM Design Sheets

IBI Group Report Exerpts (Table 4.4 – Summary of Minor System Capture & Figure 4 – Avalon West Drainage Area Plan

Storm Sewer Design Sheet

Table A1
CALCULATION OF AVERAGE RUNOFF COEFFICIENTS (POST-DEVELOPMENT)

Area No.	Area Description	Asphalt Areas		Roof Areas		Gravel Areas		Grassed Areas		Sum AC	Total Area (m^2)	C_{AVG}
		Area (m^2)	A * C	Area (m^2)	A * C	Area (m^2)	A * C	Area (m^2)	A * C			
		C=0.90		C=0.90		C=0.90		C=0.20				
Post-1	Roof			7900.0	7110.0					7110.0	7900	0.90
Post-2	Parking	24941.0	22446.9					3949.0	789.8	23236.7	28890	0.80
Post-3	Future DEV							52000.0	10400.0	10400.0	52000	0.20
Post-4	ROW Free Flow	410.0	369.0					1700.0	340.0	709.0	2110	0.34
Post-5	Loading Dock	700.0	630.0							630.0	700	0.90
							Average Runoff Coeff =			C_{AVG} =	$\frac{42,086}{91,600}$	= 0.46

Table A2
SUMMARY OF POST DEVELOPMENT RUNOFF (UNCONTROLLED AND CONTROLLED)

Area No	Outlet Location	Area (ha)	Time of Conc. T_c (min)	Storm = 5-year				Storm = 10-year				Storm = 100-year			
				C_{AVG}	I_5(mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_{10} (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_{100} (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)
Post-1	Roof	0.790	10	0.90	104.29	206.1	**23.0**	0.90	122.14	241.4	**23.0**	1.00	178.56	392.2	**23.0**
Post-2	Parking	2.889	10	0.80	104.29	673.7	**104.2**	0.80	122.14	789.0	**104.2**	1.00	178.56	1434.1	**104.2**
Post-3	Future DEV	5.200	10	0.20	104.29	301.5	301.5	0.20	122.14	353.1	353.1	0.25	178.56	645.3	**502.1**
Post-4	ROW Free Flow	0.211	10	0.34	104.29	20.6	20.6	0.34	122.14	24.1	24.1	0.42	178.56	44.0	44.0
Post-5	Loading Dock	0.070	10	0.90	104.29	18.3	18.3	0.90	122.14	21.4	21.4	1.00	178.56	34.7	34.7
Total		9.160				1220.2	**467.5**			1429.0	**525.8**			2550.3	**708.0**

Notes
1) Intensity, $I_5 = 998.071/(Tc+6.035)^{0.814}$ (5-year, City of Ottawa)
2) Intensity, $I_{10} = 1174.184/(Tc+6.035)^{0.816}$ (10-year, City of Ottawa)
3) Intensity, $I_{100} = 1735.688/(Tc+6.014)^{0.820}$ (100-year, City of Ottawa)
4) Time of Concentration: T_c=10min (5.4.5.2, City of Ottawa)
4) Flows under column Q_{CAP} which are **bold**, denotes flows that are controlled.

Table A3
Estimate of Storage Required for 5-yr and 100-yr Storms (Modified Rational Method)

Area No: Post 1 Roof
C_{AVG} = 0.90 (2-yr, 5-yr)
C_{AVG} = 1.00 (100-yr +25%)
Time Interval = 5 (mins)
Drainage Area = 0.7900 (hectares)

Duration, T_D (min)	Release Rate = 23.0 (L/sec) Return Period = 5 (years) IDF Parameters, A = 998.071, B = 0.814 ($I = A/(T_D+C)^B$), C = 6.053					Release Rate = 23.0 (L/sec) Return Period = 100 (years) IDF Parameters, A = 1735.688 0.820 ($I = A/(T_D+C)^B$), C = 6.014				
	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	230.5	455.6	23.00	432.6	0	398.6	875.4	23.000	852.4	0.0
5	141.2	279.1	23.00	256.1	77	242.7	533.0	23.000	510.0	153.0
10	104.2	205.9	23.00	182.9	110	178.6	392.2	23.000	369.2	221.5
15	83.6	165.2	23.00	142.2	128	142.9	313.8	23.000	290.8	261.7
20	70.3	138.9	23.00	115.9	139	120.0	263.4	23.000	240.4	288.5
25	60.9	120.4	23.00	97.4	146	103.8	228.1	23.000	205.1	307.6
30	53.9	106.6	23.00	83.6	150	91.9	201.8	23.000	178.8	321.8
35	48.5	95.9	23.00	72.9	153	82.6	181.4	23.000	158.4	332.6
40	44.2	87.3	23.00	64.3	154	75.1	165.0	23.000	142.0	340.9
45	40.6	80.3	23.00	57.3	**155**	69.1	151.6	23.000	128.6	347.4
50	37.7	74.4	23.00	51.4	154	64.0	140.5	23.000	117.5	352.4
55	35.1	69.4	23.00	46.4	153	59.6	130.9	23.000	107.9	356.2
60	32.9	65.1	23.00	42.1	152	55.9	122.8	23.000	99.8	359.1
65	31.0	61.4	23.00	38.4	150	52.6	115.6	23.000	92.6	361.2
70	29.4	58.1	23.00	35.1	147	49.8	109.3	23.000	86.3	362.7
75	27.9	55.1	23.00	32.1	145	47.3	103.8	23.000	80.8	363.5
80	26.6	52.5	23.00	29.5	142	45.0	98.8	23.000	75.8	**363.9**
85	25.4	50.1	23.00	27.1	138	43.0	94.3	23.000	71.3	363.8
90	24.3	48.0	23.00	25.0	135	41.1	90.3	23.000	67.3	363.4
95	23.3	46.1	23.00	23.1	131	39.4	86.6	23.000	63.6	362.6
100	22.4	44.3	23.00	21.3	128	37.9	83.2	23.000	60.2	361.5
105	21.6	42.7	23.00	19.7	124	36.5	80.2	23.000	57.2	360.1
Maximum Storage Requried =					**154.7**					**363.9**

Notes
1) Peak flow is equal to the product of 2.78 x C x I x A
2) Rainfall Intensity, $I = A/(T_D+C)^B$, where T_D = 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
7) A,B,C are IDF Parameters for City of Ottawa. From Ottawa Sewer Design Guidelines, Section 5.4.2.

Table A4
Estimate of Storage Required for 5-yr and 100-yr Storms (Modified Rational Method)

Area No: Post 2 Parking
C_{AVG} = 0.80 (2-yr, 5-yr)
C_{AVG} = 1.00 (100-yr +25%)
Time Interval = 5 (mins)
Drainage Area = 2.8890 (hectares)

Duration, T_D (min)	Release Rate = 104.2 (L/sec); Return Period = 5 (years); IDF Parameters, A = 998.071, B = 0.814, C = 6.053 ($I = A/(T_D+C)^B$)					Release Rate = 104.2 (L/sec); Return Period = 100 (years); IDF Parameters, A = 1735.688, B = 0.820, C = 6.014 ($I = A/(T_D+C)^B$)				
	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	230.5	1488.9	104.20	1384.7	0	398.6	3201.5	104.200	3097.3	0.0
5	141.2	912.0	104.20	807.8	242	242.7	1949.3	104.200	1845.1	553.5
10	104.2	673.1	104.20	568.9	341	178.6	1434.1	104.200	1329.9	797.9
15	83.6	539.8	104.20	435.6	392	142.9	1147.6	104.200	1043.4	939.1
20	70.3	453.8	104.20	349.6	420	120.0	963.4	104.200	859.2	1031.0
25	60.9	393.4	104.20	289.2	434	103.8	834.0	104.200	729.8	1094.8
30	53.9	348.4	104.20	244.2	**439**	91.9	737.8	104.200	633.6	1140.5
35	48.5	313.4	104.20	209.2	439	82.6	663.2	104.200	559.0	1173.9
40	44.2	285.4	104.20	181.2	435	75.1	603.5	104.200	499.3	1198.4
45	40.6	262.5	104.20	158.3	427	69.1	554.6	104.200	450.4	1216.0
50	37.7	243.2	104.20	139.0	417	64.0	513.6	104.200	409.4	1228.3
55	35.1	226.9	104.20	122.7	405	59.6	478.9	104.200	374.7	1236.4
60	32.9	212.8	104.20	108.6	391	55.9	448.9	104.200	344.7	1241.0
65	31.0	200.5	104.20	96.3	376	52.6	422.8	104.200	318.6	**1242.6**
70	29.4	189.7	104.20	85.5	359	49.8	399.9	104.200	295.7	1241.9
75	27.9	180.2	104.20	76.0	342	47.3	379.5	104.200	275.3	1239.0
80	26.6	171.6	104.20	67.4	323	45.0	361.3	104.200	257.1	1234.3
85	25.4	163.9	104.20	59.7	304	43.0	345.0	104.200	240.8	1228.0
90	24.3	156.9	104.20	52.7	285	41.1	330.2	104.200	226.0	1220.3
95	23.3	150.5	104.20	46.3	264	39.4	316.7	104.200	212.5	1211.3
100	22.4	144.7	104.20	40.5	243	37.9	304.4	104.200	200.2	1201.3
105	21.6	139.4	104.20	35.2	222	36.5	293.1	104.200	188.9	1190.2
Maximum Storage Requried =					**439.5**					**1242.6**

Notes

1) Peak flow is equal to the product of 2.78 x C x I x A
2) Rainfall Intensity, $I = A/(T_D+C)^B$, where T_D = 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
7) A,B,C are IDF Parameters for City of Ottawa. From Ottawa Sewer Design Guidelines, Section 5.4.2.

Table A5

Estimate of Storage Required for 5-yr and 100-yr Storms (Modified Rational Method)

Area No: Roof West Parking Lot

C_{AVG} = 0.20 (2-yr, 5-yr)

C_{AVG} = 0.25 (100-yr +25%)

Time Interval = 5 (mins)

Drainage Area = 5.2000 (hectares)

Duration, T_D (min)	Release Rate = 301.5 (L/sec); Return Period = 5 (years); IDF Parameters, A = 998.071, B = 0.814; ($I = A/(T_D+C)^B$), C = 6.053					Release Rate = 502.1 (L/sec); Return Period = 100 (years); IDF Parameters, A = 1735.688, 0.820; ($I = A/(T_D+C)^B$), C = 6.014				
	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	230.5	666.4	301.52	364.9	0	398.6	1440.6	502.100	938.5	0.0
5	141.2	408.2	301.52	106.7	**32**	242.7	877.1	502.100	375.0	**112.5**
10	104.2	301.2	301.52	-0.3	0	178.6	645.3	502.100	143.2	85.9
15	83.6	241.6	301.52	-59.9	-54	142.9	516.4	502.100	14.3	12.9
20	70.3	203.1	301.52	-98.4	-118	120.0	433.5	502.100	-68.6	-82.3
25	60.9	176.1	301.52	-125.5	-188	103.8	375.3	502.100	-126.8	-190.2
30	53.9	155.9	301.52	-145.6	-262	91.9	332.0	502.100	-170.1	-306.2
35	48.5	140.3	301.52	-161.2	-339	82.6	298.4	502.100	-203.7	-427.7
40	44.2	127.7	301.52	-173.8	-417	75.1	271.6	502.100	-230.5	-553.3
45	40.6	117.5	301.52	-184.1	-497	69.1	249.5	502.100	-252.6	-681.9
50	37.7	108.9	301.52	-192.7	-578	64.0	231.1	502.100	-271.0	-812.9
55	35.1	101.5	301.52	-200.0	-660	59.6	215.5	502.100	-286.6	-945.8
60	32.9	95.2	301.52	-206.3	-743	55.9	202.0	502.100	-300.1	-1080.3
65	31.0	89.8	301.52	-211.8	-826	52.6	190.3	502.100	-311.8	-1216.2
70	29.4	84.9	301.52	-216.6	-910	49.8	179.9	502.100	-322.2	-1353.1
75	27.9	80.6	301.52	-220.9	-994	47.3	170.8	502.100	-331.3	-1490.9
80	26.6	76.8	301.52	-224.7	-1079	45.0	162.6	502.100	-339.5	-1629.6
85	25.4	73.3	301.52	-228.2	-1164	43.0	155.2	502.100	-346.9	-1769.0
90	24.3	70.2	301.52	-231.3	-1249	41.1	148.6	502.100	-353.5	-1909.0
95	23.3	67.4	301.52	-234.1	-1335	39.4	142.5	502.100	-359.6	-2049.6
100	22.4	64.8	301.52	-236.7	-1420	37.9	137.0	502.100	-365.1	-2190.7
105	21.6	62.4	301.52	-239.1	-1506	36.5	131.9	502.100	-370.2	-2332.2
Maximum Storage Requried =					**32.0**					**112.5**

Notes

1) Peak flow is equal to the product of 2.78 x C x I x A

2) Rainfall Intensity, $I = A/(T_D+C)^B$, where T_D = 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

7) A,B,C are IDF Parameters for City of Ottawa. From Ottawa Sewer Design Guidelines, Section 5.4.2.

Table A6
CALCULATION OF AVAILABLE SURFACE STORAGE

Drainage Area	Location	Min W/L or T/G (m)	Indiv Spill Elev (m)	System Spill Elev (m)	[1]Max Depth (m)	Area (m^2)	Max Volume (m^3)	Volume Required (m^3)	Adequate Storage Provided
Post 1	Roof				0.15	7500	375.0	363.9	Yes
Post 2	Parking	88.40	88.70	88.70	0.30	12879	1287.9	1242.6	Yes
Post 3	Future Dev	87.70	88.00	88.00	0.30	6836	683.6	112.5	Yes
Total							**2346.5**	**1719.0**	

[1]The Max Depth is is the distance from the Min W/L (T/G) and the lower of the Indiv Spill or System Spill Elev

Storage volume is: V= Area x depth x 1/3

Orifice Sizing

CBMH221					
Event	**Flow (L/s)**	**Head (m)**	**ORIFICE AREA(m^2)**	**SQUARE 1-side mm**	**CIRC (mmØ)**
100 Year	104.2	3.36	0.021	146	165

Orifice Control Sizing

Q = 0.6 x A x (2gh)1/2

Where:

Q is the release rate in m^3/s

A is the orifice area in m^2

g is the acceleration due to gravity, $9.81 m/s^2$

h is the head of water above the orifice centre in m

d is the diameter of the orifice in m

Pipe Invert =	84.74	m
Ponding Elevation =	88.7	m
Top of CB Elevation =	88.4	m

Note: Orifice #1 is located on the downstream invert of CBMH221

Orifice Sizing

CBMH226					
Event	**Flow (L/s)**	**Head (m)**	**ORIFICE AREA(m^2)**	**SQUARE 1-side mm**	**CIRC (mmØ)**
100 Year	502.1	2.26	0.126	354	400

Orifice Control Sizing

Q = 0.6 x A x (2gh)1/2

Where:

Q is the release rate in m^3/s

A is the orifice area in m^2

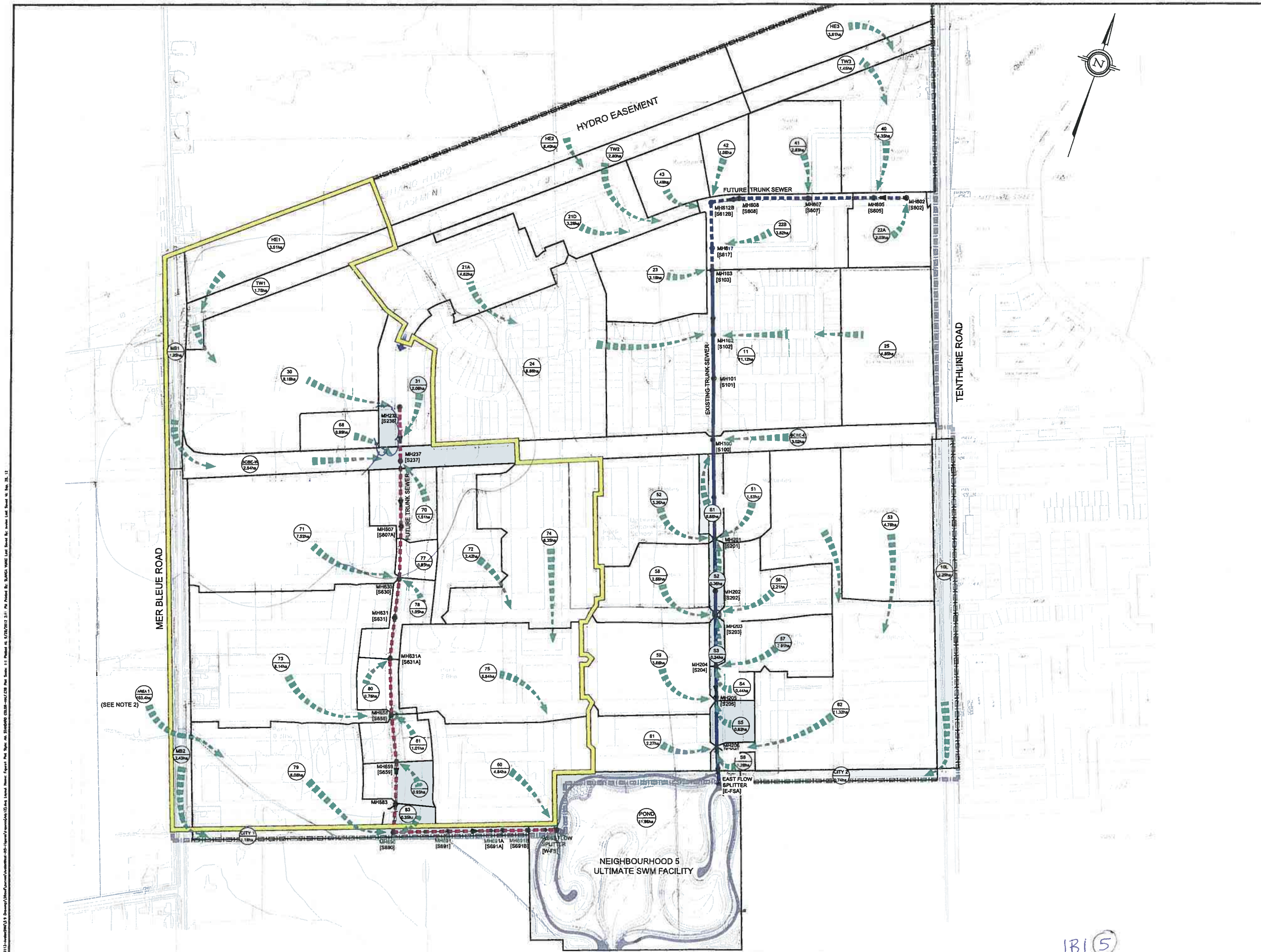
g is the acceleration due to gravity, $9.81 m/s^2$

h is the head of water above the orifice centre in m

d is the diameter of the orifice in m

Pipe Invert =	85.29	m
Ponding Elevation =	88.7	m
Top of CB Elevation =	87.7	m

Note: Orifice #2 is located on the downstream invert of CBMH226



LEGEND:

- NEIGHBOURHOOD 5 BOUNDARY
- EXISTING EASTERN TRUNK SEWER
- FUTURE EASTERN TRUNK SEWER
- FUTURE WESTERN TRUNK SEWER
- DRAINAGE BOUNDARIES
- WESTERN TRUNK DRAINAGE BOUNDARY
- AREA CONTRIBUTED MAJOR FLOW TO WESTERN TRUNK (MINOR TO EASTERN TRUNK)
- MAJOR FLOW
- 70 / 1.51ha — AREA ID / AREA (ha)
- MH630 — TRUNK STORM SEWER MANHOLE
- [S607] — XPSWMM MANHOLE ID
- CONNECTIVITY OF SUBCATCHMENT MANHOLE

NOTE:

1. FOR ADDITIONAL DETAILS REFER TO THE SWMHYMO SCHEMATIC IN FIGURE 5
2. AREA 1 RUN OFF WILL BE CONVEYED TO A DITCH INLET AT MH 690 AND ENTER THE SWM FACILITY.

No.	REVISIONS	By	Date
14			
13			
12			
11			
10			
9			
8			
7			
6			
5			
4			
3	REVISED AS PER CITY COMMENTS	P.S.	12/10/01
2	REVISED AS PER CITY COMMENTS	P.S.	12/08/14
1	ISSUED TO CITY	P.S.	12/05/30

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Project Title
AVALON WEST (N5) STORMWATER MANAGEMENT FACILITY DESIGN

Drawing Title
AVALON WEST DRAINAGE AREA PLAN

Scale
1:3000

Design	Date
R.B.	FEB. 2012
Drawn	Checked
S.V.	P.S.
Project No.	Drawing No.
30113	FIGURE 4

Table 4.4 Summary of Minor System Capture
(SWMHYMO Output 30113-D6.dat/out)

Drainage Area		Weighted Runoff Coefficient	Flow (l/s)			Manhole (XPSWMM ID)	Peak Flow from Saved Hydrograph (SWMHYMO) – 3 hour Chicago (l/s)	
ID	Area (ha)		220 l/s/ha	Rational Method 10 Year 10 Min	Other Restriction (refer to Table 4.2)*		5 Year†	100 year†
Eastern Trunk Storm Sewer								
HE3	3.61	n/a	0	0	30	MH 605 (S605)	1015	1017
TW3	1.45	0.70	0	344.7	0			
40	4.35	0.76	0	0	645.7			
41	2.83	0.76	0	0	420.1	MH 607 (S607)	419	419
HE2	6.40	n/a	0	0	30	MH 612B (S612B)	965	1002
TW2	2.80	0.70	0	665.5	0			
43	1.49	0.62	0	0	180.4			
42	1.08	0.62	0	0	130.8			
22A	2.03	0.76	0	0	172.6	MH 602 (S602)	172	173
22B	3.62	0.59	0	0	307.7	MH 617 (S617)	306	308
23	3.16	n/a	0	0	268	MH 103 (S103)	106	267
21A	4.62	0.59	0	0	392.7	MH 102 (S102)	2778	2782
21D	3.28	0.59	0	0	278.8			
24	8.86	0.57	0	0	753.1			
25	4.85	0.80	0	0	412.3			
11	11.12	0.57	0	0	945.2			
BCB-E	3.02	0.70	0	717.8	0	MH 100 (S100)	595	867
S1	0.68	0.76	149.6	0	0			
51	1.53	0.76	335.5	0	0	MH 201 (S201)	807	1131
52	3.26	0.63	717.2	0	0			
S2	0.36	0.66	79.2	0	0			
56	2.21	0.66	486.2	0	0	MH 203 (S203)	797	1123
58	2.66	0.71	585.2	0	0			
S3	0.24	0.66	52.8	0	0			
57	2.01	0.41	442.2	0	0	MH 204 (S204)	222	538
S4	0.44	0.66	96.8	0	0			
59	3.86	0.61	849.2	0	0	MH 205 (S205)	577	985
S5	0.62	0.62	136.4	0	0			
61	2.27	0.63	499.4	0	0	MH 206 (S206)	2391	4094
62	11.32	0.68	2489.3	0	0			
53	4.78	0.66	1051.6	0	0			
S6	0.29	0.62	63.8	0	0			
10L	2.25	0.70	0	534.8	0	E-FS	369	535
Western Trunk Storm Sewer								
31	2.06	0.59	0	0	353.2	236 (S236)	1442	1767
HE1	3.51	0.27	0	0	220.1			
TW1	1.75	0.70	0	416.0	0			
30	9.16	0.66	0	0	778.6			
68	0.99	0.76	0	0	217.9	237	1033	1474

Table 4.4 cont...

Drainage Area		Weighted Runoff Coefficient	Flow (l/s)			Manhole (XPSWMM ID)	Peak Flow from Saved Hydrograph (SWMHYMO) – 3 hour Chicago (l/s)	
ID	**Area (ha)**		**220 l/s/ha**	**Rational Method 10 Year 10 Min**	**Other Restriction (refer to Table 4.2)***		**5 Year†**	**100 year†**
MB1	1.35	0.70	0	320.9	0	(S237)		
BCB-W	2.54	0.70	0	603.7	0			
70	1.51	0.66	332.2	0	0			
71	7.08	0.69	1654.4	0	0	630 (S630)	1191	1874
78	1.05	0.66	231.0	0	0			
77	0.53	0.65	116.6	0	0	607 (S607A)	92	167
80	0.76	0.66	167.2	0	0	631A (S631A)	128	167
73	8.14	0.63	1790.8	0	0	658 (S658)	1119	2005
81	1.01	0.63	222.2	0	0			
82	0.93	0.62	204.6	0	0	659 (S659)	146	204
MB2	2.45	0.70	0	582.3	0	690 (S690)	1070	2049
79	6.08	0.59	1337.6	0	0			
83	0.35	0.62	77.0	0	0			
Area 1	103.4	n/a	Unrestricted					
72	2.42	0.68	531.9	0	0	770 (S770)	2388	4272
74	6.35	0.66	1397.0	0	0			
75	5.84	0.44	1284.8	0	0			
60	4.84	0.62	1064.8	0	0			

Note: * Refer to **Table 4.2** for reference to other restriction rates.
† Results from the 5 and 100 year 3 hour Chicago evaluation from 30113-D6.dat/out as presented in **Appendix B**.

The inlet control restriction was set to match flow as indicated in **Table 4.4**. The results of the SWMHYMO evaluation indicate that when applying the inlet control restriction and storage requirements as indicated in **Table 4.4**, there is cascading flow from existing areas and some future areas north of BCB. The future development areas south of BCB are self-contained with some instances where major flow is conveyed via some drainage areas to the stormwater facility. It is anticipated that detail evaluation of the major system for the future development will be completed as part of detailed subdivision design submission. The evaluation should account for the conveyance of major flow from the existing upstream areas. As part of the detailed evaluation of the major system, the total ponding in low points is not to exceed a depth of 0.3 m under static or dynamic conditions. The surface ponding design has to ensure that emergency overflow paths are maintained and flows are conveyed safely to the end-of-pipe stormwater management facility. A safety board of 0.3 m above the emergency overflow high point and lowest house opening should be provided.

Major System

Based on the hydrological modeling undertaken for the entire site on a semi-lumped basis, there are some instances of cascading major flow from already constructed areas north of BCB and from arterial roadways within the subdivision area. The following table summarizes the estimated major flow within the site based on semi-lumped areas. It should be noted that as part of detailed evaluation of the subdivision areas, the total ponding in low points is not to exceed a depth of 0.3 m under static or dynamic conditions. The surface ponding design has to ensure that emergency overflow paths are maintained and flows are conveyed safely to the

5 year Free Flow Storm Sewer Design Sheet

LOCATION FROM	LOCATION TO	Area (ha)	R=	INDIV 2.78 AR	ACCUM 2.78 AR	TIME OF CONC.	RAINFALL INTENSITY I	BLDG FLOW Q (l/s)	PEAK FLOW Q (l/s)	PIPE SIZE (mm)	PIPE SLOPE (%)	LENGTH (m)	CAPACITY (l/s)	FULL FLOW VELOCITY (m/s)	TIME OF FLOW (min.)	EXCESS CAPACITY (l/s)	Q/Qfull
CB101	CBMH203	0.05	0.90	0.14	0.14	10.00	104.19		14.08	300.0	1.00	26.0	96.80	1.37	0.32	82.72	0.15
CBMH203	CBMH202	0.04	0.90	0.11	0.24	10.32	102.55		24.89	250.0	0.50	26.0	42.09	0.86	0.51	17.20	0.59
CB102	CBMH201	0.04	0.90	0.11	0.11	10.00	104.19		11.47	250.0	0.50	26.0	42.09	0.86	0.51	30.62	0.27
CBMH201	CBMH202	0.05	0.90	0.13	0.24	10.51	101.59		23.89	250.0	0.50	26.0	42.09	0.86	0.51	18.20	0.57
CBMH202	CBMH205	0.05	0.90	0.12	0.60	11.01	99.14		59.03	900.0	0.10	18.3	573.05	0.90	0.34	514.02	0.10
CB103	CBMH204	0.04	0.90	0.10	0.10	10.00	104.19		10.69	250.0	0.50	26.0	42.09	0.86	0.51	31.40	0.25
CBMH204	CBMH 205	0.03	0.90	0.08	0.18	10.51	101.59		18.56	250.0	0.50	26.0	42.09	0.86	0.51	23.54	0.44
CB104	CBMH206	0.04	0.90	0.09	0.09	10.00	104.19		9.65	250.0	0.50	26.0	42.09	0.86	0.51	32.45	0.23
CBMH206	CBMH205	0.04	0.90	0.09	0.19	10.51	101.59		18.81	250.0	0.50	26.0	42.09	0.86	0.51	23.28	0.45
CBMH 205	CBMH208	0.03	0.90	0.09	1.05	11.35	97.56		102.28	900.0	0.10	15.3	573.05	0.90	0.28	470.77	0.18
CB105	CBMH209	0.06	0.90	0.15	0.15	10.00	104.19		15.64	250.0	0.50	26.0	42.09	0.86	0.51	26.45	0.37
CBMH209	CBMH208	0.05	0.90	0.13	0.28	10.51	101.59		27.96	250.0	0.50	26.0	42.09	0.86	0.51	14.13	0.66
CB106	CBMH207	0.05	0.90	0.13	0.13	10.00	104.19		13.03	250.0	0.50	26.0	42.09	0.86	0.51	29.06	0.31
CBMH207	CBMH208	0.05	0.90	0.13	0.25	10.51	101.59		25.42	250.0	0.50	26.0	42.09	0.86	0.51	16.67	0.60
CBMH208	CBMH211	0.05	0.90	0.13	1.70	11.63	96.29		163.58	900.0	0.10	19.1	573.05	0.90	0.35	409.47	0.29
CB107	CBMH210	0.06	0.90	0.15	0.15	10.00	104.19		15.64	250.0	0.50	26.0	42.09	0.86	0.51	26.45	0.37
CBMH210	CBMH211	0.05	0.90	0.13	0.28	10.51	101.59		27.96	250.0	0.50	26.0	42.09	0.86	0.51	14.13	0.66
CB108	CBMH212	0.05	0.90	0.13	0.13	10.00	104.19		13.03	250.0	0.50	26.00	42.09	0.86	0.51	29.06	0.31
CBMH212	CBMH211	0.05	0.90	0.13	0.25	10.51	101.59		25.42	250.0	0.50	26.00	42.09	0.86	0.51	16.67	0.60
CBMH211	CBMH213	0.05	0.90	0.13	2.35	11.99	94.75		222.60	900.0	0.10	19.20	573.05	0.90	0.36	350.45	0.39

CMMH216	CBMH215	0.14	0.90	0.35	0.35	10.00	104.19		36.50	450.0	0.20	60.00	127.63	0.80	1.25	91.14	0.29
CB110	CBMH214	0.04	0.90	0.10	0.10	10.00	104.19		10.43	250.0	0.50	27.50	42.09	0.86	0.54	31.66	0.25
CBM H214	CBMH25	0.17	0.90	0.43	0.53	10.54	101.45		53.30	450.0	0.20	52.00	127.63	0.80	1.08	74.33	0.42
CBMH215	CBMH213	0.19	0.90	0.48	1.35	11.62	96.37		130.20	525.0	0.20	100.80	192.52	0.89	1.89	62.32	0.68
CBMH213	CBMH222	0.23	0.90	0.58	4.28	13.51	88.71		379.33	1050.0	0.10	82.50	864.40	1.00	1.38	485.07	0.44
CBMH222	CBMH221	0.05	0.90	0.13	4.40	14.89	83.93		369.37	1050.0	0.10	50.00	864.40	1.00	0.84	495.03	0.43
CBMH217	CBMH218	0.28	0.70	0.54	0.54	10.00	104.19		56.77	450.0	0.20	38.00	127.63	0.80	0.79	70.86	0.44
CBMH218	CBMH219	0.20	0.60	0.33	0.88	10.79	100.20		88.02	525.0	0.20	62.00	192.52	0.89	1.16	104.50	0.46
CBMH219	CBMH220	0.12	0.60	0.20	1.62	11.95	94.90		154.07	525.0	0.20	91.60	192.52	0.89	1.72	38.46	0.80
CBMH220	CBMH221	0.35	0.60	0.58	2.21	13.67	88.11		194.49	600.0	0.20	68.00	274.87	0.97	1.17	80.38	0.71
BUILDING	CBMH229	0.79	0.90	1.98	1.98	10.00	104.19	**23.00**	205.95	450.0	0.20	25.00	127.63	0.80	0.52	-78.31	1.61
CBMH229	CBMH332	0.00	0.90	0.00	1.98	10.52	101.53		200.67	450.0	0.20	7.50	127.63	0.80	0.16	-73.04	1.57
CBMH232	CBMH230	0.00	0.90	0.00	1.98	10.68	100.75		199.15	525.0	0.20	23.50	192.52	0.89	0.44	-6.63	1.03
CBMH230	CBMH221	0.00	0.90	0.00	1.98	11.12	98.64		194.98	525.0	0.20	17.80	192.52	0.89	0.33	-2.45	1.01
CBMH221	CBMH223	0.27	0.60	0.45	9.04	15.72	81.30		734.54	1200.0	0.10	88.50	1234.13	1.09	1.35	499.59	0.60
CBMH223	CBMH224	0.18	0.60	0.30	9.34	17.07	77.40		722.60	1200.0	0.10	45.00	1234.13	1.09	0.69	511.53	0.59
EXTERNAL HE1	CBMH227	3.51	0.27	2.63	2.63	15.00	83.56		220.14	750.0	0.20	100.00	498.38	1.13	1.48	278.23	0.44
EXTERNAL TW1	CBMH226	1.75	0.70	3.41	6.04	15.00	83.56		504.69	825.0	0.20	100.00	642.59	1.20	1.39	137.90	0.79
FUTURE PHASE	CBMH225	5.21	0.20	2.90	8.94	10.00	104.19		931.16	900.0	0.30	100.00	992.55	1.56	1.07	61.39	0.94
CBMH225	CBMH224	0.00	0.20	0.00	8.94	15.00	83.56		746.74	900.0	0.30	100.00	992.55	1.56	1.07	245.81	0.75
EXISTING	Gerry Lalonde	0.00	0.00	0.00	17.97	17.76	75.58		1358.31	1350.0	0.10	10.00	1689.54	1.18	0.14	331.23	0.80
TOTAL		**14.42**		**17.73**								**1737.6**					

Notes:

Rainfall Intensity = $998.071/(T+6.053)^{-0.814}$ T= time in minutes

Appendix B –

Email-Boundary Conditions Provided by the City of Ottawa

Pressure Check Under Max Day + FireFlow

2225 Mer Bleue Road - Boundary Conditions

Information Provided:

Date provided: November 2017

Scenario	Demand	
	L/min	L/s
Average Daily Demand	318	5.3
Maximum Daily Demand	480	8
Peak Hour	570	9.5
Fire Flow Demand # 1	1920	32.0

Location:

Results:

Connection 1 - Brian Coburn Blvd

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	130.5	59.9
Peak Hour	126.4	54.0
Max Day plus Fire (1,920 l/min)	127.1	55.1

[1] Ground Elevation = 88.34 m

Connection 2 - Gerry Lalonder Dr

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	130.4	59.6
Peak Hour	125.7	53.0
Max Day plus Fire (1,920 l/min)	126.8	54.6

[1] Ground Elevation = 88.43 m

Notes:

Disclaimer

The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

Water Pressure Analysis at Building for Max Day + Fireflow

Max day8L/s) + FireFlow(32L/s) HGL=	**127.1**	**m**	**Brian Coburn Blvd**
Max day8L/s) + FireFlow(32L/s) HGL=	**126.8**	**m**	**Gerry Lalonde Dr**

Description	From	To	Flow (L/sec)	Pipe Dia (mm)	Dia (m)	Q (m^3/sec)	Area (m2)	C	Vel (m/s)	Slope of HGL (m/m)	Pipe Length (m)	Frictional Head Loss h_f (m)	Equivalent Pipe Length of Fittings (m)	Minor Loss of Fittings h_b (m)	Total Losses (m) $h_b + h_f$	Start Ground Elev(m)	End Ground Elev (m)	Static Head (m)	Pressure From kPa	(psi)	Pressure To kPa	(psi)	Pressure Drop (psi)
Max Day + Fire Flow	Main Terminal Ave	0+210	40	200	0.200	0.04	0.0314159	125	1.2732	0.0091	201.3	1.834004406	84.8	0.77260	2.60660	88.34	89.20	-0.86	380.1	(55.1)	346.1	(50.2)	4.9
Max Day + Fire Flow	Main Sandford Flemming	0+210	135.65	200	0.200	0.13565	0.0314159	125	4.3179	0.0875	201.3	17.60462331	21.8	1.90651	19.51113	88.43	89.20	-0.77	376.3	(54.6)	177.4	(25.7)	28.8

Resistance of Fittings and Valves for 200mm WM from Terminal to 0+200

Fittings	Loss in Equiv. Length in Pipe Diameters	Equiv. Length (metres)	Quantity (each)	Total Equiv. Length (m)
Standard 90^0 Elbow	32	6.40	9	57.6
11.25 Degree Elbow	4	0.80	8	6.4
45 Degree Elbow	16	3.20	0	0
Gate Valve Full -Open	13	2.60	8	20.8
		Total:	**25**	**84.8**

Resistance of Fittings and Valves for 200mm WM from Sanford Fleming to 0+200

Fittings	Loss in Equiv. Length in Pipe Diameters	Equiv. Length (metres)	Quantity (each)	Total Equiv. Length
Standard 9(	32	6.40	2	12.8
11.25 Degre	4	0.80	0	0
45 Degree E	16	3.20	2	6.4
Gate Valve	13	2.60	1	2.6
		Total:	**5**	**21.8**

Marc Alain Lafleur

From: Darrell Noseworthy <Darrell.Noseworthy@smithandandersen.com>
Sent: Tuesday, November 21, 2017 10:30 AM
To: Marc Alain Lafleur; Havers, Christopher
Cc: Alam Ansari; Kevin Sharples
Subject: RE: OHH

Hi Marc/Chris,

The sprinkler demand for this building will be approximately 360 gpm.

Note that this value will be confirmed at the beginning of construction once the fire protection engineer completes his/her hydraulic calculations.

Regards,

Smith + Andersen

Darrell Noseworthy P.Eng., LEED AP
Associate
d 613 691 0272 m 613 325 7166

From: Marc Alain Lafleur [mailto:MarcAlain.Lafleur@exp.com]
Sent: November-16-17 3:47 PM
To: Havers, Christopher <Christopher.Havers@hdrinc.com>
Cc: Darrell Noseworthy <Darrell.Noseworthy@smithandandersen.com>; Alam Ansari <alam.ansari@exp.com>
Subject: OHH

Goo afternoon Chris,

Please note that we will require the sprinkler demand for the OHH facility. I have already spoken to Darrel regarding this request.

Furthermore, we will require the roof plan layout showing roof drain locations.

Thank you,
Marc

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