Stormwater Management & Servicing Report
KMA Mosque and Community Centre
351 Sandhill Road, Ottawa, Ontario

Client:
Kanata Muslim Association
351 Sandhill Road
Ottawa, Ontario K2K 1X7

Project Number:
OTT-00238564-A0

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Date Submitted:
September 13, 2017
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1 Introduction

Exp Services Inc. was retained by **KMA** to prepare a Stormwater Management & Servicing Report in support of an application for Zoning By-law Amendment and Site Plan Control, to facilitate the use of the subject property as a place of worship (mosque and community centre) and for the conversion of the existing single detached dwelling for that use and the ultimate construction of an additional 2 storey, plus basement structure on the parcel of land municipally known as 351 Sandhill Road in Kanata North.

This report provides a stormwater management and servicing design brief in support of the proposed phased development. The 0.8ha site is generally flat with a gentle slope from west to east towards Sandhill Road. There is an existing single detached dwelling and a storage shed on the property. The dwelling will be converted for use as a mosque. The first phase of development will consist of converting the existing building into a place of worship and constructing a new parking lot on the west side of the existing building and a new access to the parking lot. The second phase of development will consist of a new 2 storey building constructed adjacent to the existing building. The 2 storey building which will include a basement will serve as a place of worship and a Community Centre.

The site is currently serviced by on-site water wells and a septic system. One of the three water wells is not in service and has been abandoned. The other two wells are currently in use to meet the domestic water supply and heating/cooling requirements of the existing dwelling.

The proposed development will be connected to the municipal services on Sandhill Road. The existing septic system will be removed. The two existing water wells will be used for the heating/cooling requirements of the existing building.

This servicing design brief will address SWM requirements for the phased development of the 0.8ha site and how the proposed phased development will be serviced with sanitary, storm and water services. Servicing, Grading and drainage and SWM plans for both phases of development are included with this report.

2 References

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

- City of Ottawa Sewer Design Guidelines Revision 2, October 2012 (SDG002)
    - Technical Bulletin ISTB-2018-01
- City of Ottawa Water Distribution Design Guidelines, July 2010 (WDG001)
    - Technical Bulletin ISTB-2018-02
- Stormwater Management Planning and Design Manual, Ontario Ministry of the Environment, March 2003 (MOE SMPDM)

3 Sanitary Sewer Design

The site is currently serviced by an on site septic system which will be decommissioned. The proposed development will be serviced by a new sanitary sewer connected to the existing 250mm municipal sanitary sewer on Sandhill Rd. The anticipated peak sanitary flow from the existing building and the proposed addition has been calculated as per the City of Ottawa Sewer Design Guidelines (SDG02, 2012) and Technical Bulletin ISTB-2018-01. The anticipated peak sanitary flow is calculated as follows:

Design Flows

Design Flow for Institutional Use:	28,000 L/day/ha (0.324 L/s/ha)
Peaking Factor:	1.5
Site Area:	0.818 hectares
Extraneous Flow:	0.33 L/s/ha
Peak Design Flow:	= (0.324L/s/ha)(0.818ha)(1.5)+(0.818ha)(0.33L/s/ha)
	=**0.67 L/s**

The new 2 storey building and the existing building will be serviced by a new 150mm diameter sanitary service. The proposed on-site sanitary sewer will be installed at a minimum grade of 0.60%. At this slope, the 150mm diameter sanitary sewer will have a capacity of 12.7 L/s and a full flow velocity of 0.94 m/s which will be sufficient to service the site under both phases of development. The City of Ottawa Sewer Design Guidelines recommend a flow velocity between 0.8m/s to 3m/s. Two new 150mm diameter sanitary services will be provided to service the existing and new buildings. Refer to Appendix C for detailed calculations and the Site Servicing and Grading Plans for Phase 1 & Phase 2 (dwg nos. SSGP1 & SSGP2) for layout details.

4 Watermain Design

The proposed development will be serviced by a new 200mm diameter water service which will be connected to the existing 305mm diameter municipal watermain on Sandhill Road to meet the domestic and fire flow requirements for the site.

will be serviced by a new 150mm diameter watermain connected to the 200mm diameter watermain servicing the site. The water supply to the existing building during phase 1 will be provided by a new 50mm diameter copper service connected to the 200mm water main. This service will be removed following construction of phase2 as the existing building will then be serviced internally from the new building.

Fire Water Demand
The fire flow demand calculations were prepared based on the Fire Underwriters Survey (FUS, 1999) criteria. For detailed fire flow demand calculations, refer to Appendix B. The new addition will be sprinklered and will be a non-combustible steel frame construction. The existing dwelling is combustible wood construction and is not sprinklered. Based on technical bulletin ISTB-2018-02 the classification for mixed construction rule of non-combustible construction applies, where 66.7% or over of the total wall, floor and roof area constructed are defined as non-combustible. Credits for sprinkler protection applied for the percentage of the floor area protected by sprinklers.

The fire flow demand for both buildings was calculated to be 67 L/s. A fire hydrant will be installed at the south corner of the proposed gymnasium to provide fire protection for the proposed development.

The domestic water demands for the proposed site were calculated as per the City of Ottawa Water Distribution Guidelines and Technical Bulletin 2018-02.

Institutional Water Demand

Average daily demand:
=28,000 L/ha/day
=0.818 ha x 28,000 L/ha/day x (1/86,400 s/day)
= 0.27 L/s

Maximum daily demand:
=1.5 x avg. day
=1.5 x 0.27 L/s
=0.41 L/s

Maximum hourly daily demand:
=1.8 x max.day
=1.8 x 0.41 L/s
=0.74 L/s

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

Peak Hour HGL = 125.0m

Maximum HGL = 131.2m

Max Day (0.75 L/s) + FireFlow (150L/s) = 123.2m

Based on the HGL of 123.2m for the max day + fire flow scenario, a pressure analysis was performed and a residual pressure of 51.8 psi (357 kPa) was estimated at the proposed 2 storey building. Refer to Appendix B for calculation details. The residual water pressure during the max day + fire scenario is greater than the minimum requirement of 20psi (140kPa) as per the City of Ottawa Design Guidelines. The existing water supply system will therefore have adequate capacity to meet the domestic and fire demands for the proposed development.

5 Stormwater Management

5.1 Storm Design Criteria

The storm sewer system was designed in conformance with the City of Ottawa Sewer Design Guidelines (SDG02, 2012). The stormwater servicing design criteria for the proposed development is as follows:

- The proposed on-site storm sewer network / minor system, is designed using Rational Method and Manning's Equation to convey runoff under free flow conditions for the 5-year return period.
- Maximum allowable ponding depth is 300 mm.

- No surface ponding during 2 year storm events
- Flows from storms events greater than the 100-year return period will be directed overland towards Sandhill Rd.
- Average runoff coefficients were calculated for each inlet drainage area using a runoff coefficient of 0.25 for pervious surfaces and 0.90 for impervious surfaces.
- Estimated storage volumes based on the Modified Rational Method.
- 100-year minor system flows to the sewer on Sandhill Rd must be controlled to the allowable release rate criteria of 85L/s/ha.
- Water quality will be provided by the existing off-site Briar Ridge Stormwater Management Facility.
- Minimum freeboard of 0.3m between the 100-year overland flow elevation and finished floor.

5.2 Pre-Development Conditions

The site is generally flat with a gentle slope from west to east towards Sandhill Road. There is no existing storm sewer system on the site. The stormwater runoff currently sheet drains overland towards Sandhill Road.

5.3 Allowable Release Rate

Minor system flows from the site to the 675mm diameter stormsewer on Sandhill Rd will be restricted to 85L/s/ha for up to the 100-year event. The allowable release rate criteria of 85L/s/ha was established as part of the Briar Ridge Phase 2 Subdivision. Refer to email from the City dated April 28, 2017 (copy included in appendix A). The ICDs have been sized to ensure that there is no surface ponding during 2-year storm events. The allowable post development release rates from the site up to the 100-year storm events will be controlled to a release rate of 48.74 L/s for phase 1 of development. The allowable release rate for phase 1 is based on a development area of 0.57ha since only 0.57ha will be developed during phase 1. The allowable post development release rate for phase 2 for up to the 100-year storm events will be controlled to 68.8 L/s based on the entire site area of f 0.81ha.

5.4 Post-Development Conditions

Stormwater will be controlled and released at a rate less than the allowable release rate for storms up to and including the 100-year storm event during both phases of development. An overland flow route is provided for storms greater than the 100-year event. Flow control devices will be installed in roof drains of the proposed 2 storey building and various catchbasins/manholes in order to control stormwater prior to its release from the site. The site under phase 1 post development conditions has been divided into 2 drainage areas, refer to Stormwater Management drawing SWM-1. The site under phase 2 post development conditions has been divided into 4 drainage areas, refer to Stormwater Management drawing SWM-2.

5.4.1 Storage Requirements and Allocation

Post development runoff will be detained on-site for storms up to and including the 100-year storm. The required SWM storage volumes will be achieved using the surface storage in the parking-lots and storage on the roof of the new building for storms up to the 100-year event. Underground pipe and structure storage volumes will be utilized to meet the criteria of no surface ponding during the 2-year storm events.

Surface ponding volumes over catch basins and catch basin manholes were determined by applying the pyramid volume equation of one-third of the depth multiplied by the surface area of the pond. Ponding depths for the subject site must be equal to or less than 300 mm for the 100-year storm event. There will be no surface ponding for 2-year storm events. Major overland flows from storms greater than the 100-year event will be directed to Sandhill Rd.

The volume of storage required was calculated for both the 2-year and 100-year storm events using the Rational Method. Since more storage is available than will be required in both the 2-year and 100-year events, the ponding level will be less than 300mm for the 100-year event. There will be no surface ponding during 2 year storm events.

For phase 1 of development, the 100-year on-site required and available storage volumes were estimated to be 182.5 m^3 and 218.8 m^3, respectively. The 2-year on-site required and available storage volumes were determined to be 34.3 m^3 and 35.1m^3, respectively.

For phase 2 of development, the 100-year on-site required and available storage volumes were determined to be 246.8 m^3 and 275.8 m^3, respectively. The 2-year on-site required and available storage volumes were determined to be 35.1 m^3 and 35.1m^3, respectively.

Detailed stormwater management calculations are shown in Appendix A, including storage requirements and storage quantities provided. Appendix A1 contains the stormwater management calculations for phase 1 of development and Appendix A2 contains the calculations for phase 2 of development. Ponding levels and drainage areas for the site are shown on the post-development storm drainage plan SWM-1 and SWM-2 for Phase 1 and Phase 2 respectively.

5.4.2 Flow Control Device Sizing

A simple plug-type insert is suitable if the orifice diameter is 75 mm or greater. the simple plug-type orifice sizing has been determined as per the following example:

Q = C(A)(2gh)0.5

A = Q / (C(2gh)0.5)

πr2 = Q / (C(2gh)0.5)

r = (Q / (π (C(2gh)0.5))0.5

r = $(0.0295 / (\pi (0.6(2*9.81*2.75)^{0.5}))^{0.5}$

r = 0. 0375m

diameter = 2r = 2(0.0375) = 0.075m = 75mm

Where;

C = 0.6 head loss coefficient for an orifice

Q = 29.5 L/s direct release rate from sub-area to sewer (= 0.0295 cub.m/s)

H = 2.75m head on orifice (top of grate + ponding depth – pipe invert - pipe radius

A = Area of orifice

r = radius of orifice

g = acceleration due to gravity

There are 2 proposed plug-type ICDs proposed at the site, a 75mm diameter orifice and a 90mm diameter orifice. ICDs and their locations are shown on the Site Servicing and Grading plans and the Stormwater Management Plan for both phase 1 and phase 2 of development. The maximum head during the 100-year storm event will be 3.05 m on the 90mm diameter ICD and 2.25 m on the 75mm diameter orifice.

Refer to Appendix A for orifice sizing calculations for the plug-type orifices.

5.4.3 Summary of Proposed SWM System

A summary of the release rates for the controlled and uncontrolled drainage areas and corresponding storage details for phase 1 of development is provided in Table 5.1 below. Refer to Appendix A1 for detailed stormwater management spreadsheet calculations. The Post development 100-year controlled release rate from the site is of 47.1 L/s, which is less than the allowable limit of 48.74 L/s.

Table 5-1: Phase 1 Summary of Proposed On-Site SWM System

Area ID	Area (ha)	Runoff Coefficient 'C'	2 Year Release (L/s)	2 Year storage required (m^3)	2 Year underground storage provided (m^3)	100 Year Release (L/s)	100 Year storage required (m^3)	100 Year surface storage provided (m^3)
PH1A	0.486	0.79	28.0	34.3	35.1	29.5	169.9	193.5
PH1B	0.088	0.73	11.0	0.0	0.0	17.6	12.7	25.3
TOTAL	**0.57**							
		Totals:	**39.0**	**34.3**	**35.1**	**47.1**	**182.5**	**218.8**
	Total Allowable Release (L/s):			**48.74**				

A summary of the release rates for the controlled and uncontrolled drainage areas and corresponding storage details for phase 2 of development is provided in Table 5.2 below. Refer to Appendix A2 for detailed stormwater management spreadsheet calculations and refer to Appendix C for the sewer design sheet. The Post development 100-year controlled release rate from the Site is of 68.85L/s, which is less than the allowable limit of 69.53L/s.

Table 5-2: Phase 2 Summary of Proposed On-Site SWM System

Area ID	Area (ha)	Runoff Coefficient 'C'	2 Year Release (L/s)	2 Year storage required (m^3)	2 Year underground storage provided (m^3)	100 Year Release (L/s)	100 Year storage required (m^3)	100 Year surface storage provided (m^3)
PH2A	0.498	0.78	28.0	35.1	35.1	29.5	173.3	193.5
PH2B	0.118	0.77	15.6	0.0	0.0	17.6	24.9	25.3
PH2C	0.145	0.90	7.0	13.9	57.0	10.0	48.7	57.0
PH2D	0.045	0.46	3.5	0.0	0.0	10.2	0.0	0.0
TOTAL	**0.81**							
		Totals:	**54.2**	**49.0**	**92.1**	**67.4**	**246.8**	**275.8**
		Total Allowable Release L/s		**68.85**				

5.4.4 Quality Control

Stormwater quality control will be provided by the existing Brookside Pond-C SWF, which receives flows from the 675mm storm sewer on Sandhill Road. Therefore, no other water quality measures are proposed. Refer to email from the City dated April 28, 2017 (copy included in appendix A).

6 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;
- Minimize the area to be cleared and disruption of adjacent areas;
- Filter cloth shall be installed between frame and cover of all catch basins, catch basin manholes, and storm manholes as 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 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) 805.

7 Conclusions

This report addresses the adequacy of the existing municipal services to service the proposed development at 351 Sandhill Rd, Ottawa, Ontario. Based on the analysis provided in this report, the conclusions are as follows:

- A 200mm diameter water service connected to the 300mm municipal water main on Sandhill Road will adequately service the proposed development.
- A Fire Hydrant will be installed to provide fire protection for the proposed development.
- A 150mm diameter sanitary service connected to the 250mm diameter municipal sanitary sewer on Sandhill Road system will adequately service the proposed development.
- SWM for the proposed phased development will be achieved by restricting all storms up to the 100-year post development flow to the allowable release rate. The allowable release rate criteria of 85L/s/ha was established as part of the Briar Ridge Phase 2 Subdivision.
- Required on-site SWM storage volumes will be achieved using the surface storage in the parking-lots, storage on the roof of the new building for storms up to the 100 year event.
- Quality control will be provided by the existing Brookside Pond-C SWF, which receives flows from the 675mm storm sewer on Sandhill Road. No other water quality measures are proposed.
- Temporary erosion and sediment control measures for the subject site have been identified.
- Overland flow routes have been provided for the subject site.
- During all construction activities, erosion and sedimentation shall be controlled

Appendix A1 – SWM Phase 1 Design Sheets

Table D1
CALCULATION OF AVERAGE RUNOFF COEFFICIENTS (POST-DEVELOPMENT)

Area No.	Area	Asphalt & Concrete 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				
PH1A	Parking Lot	3872.0	3484.8	245.0	220.5			738.0	147.6	3852.9	4855	0.79
PH1B	Access Rd	533.0	479.7	127.0	114.3			219.0	43.8	637.8	879	0.73
							Average Runoff Coeff =			C_{AVG} =	$\frac{4{,}531}{5{,}795}$	= 0.78

Table D2
SUMMARY OF POST DEVELOPMENT RUNOFF (UNCONTROLLED AND CONTROLLED)

Area No	Outlet Location	Area (ha)	Time of Conc. T_c (min)	Storm = 2-year				Storm = 5-year				Storm = 100-year			
				C_{AVG}	I_2 (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_5 (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_{100} (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)
PH1A	Parking Lot	0.486	15	0.79	61.77	66.2	**28.0**	0.79	83.56	89.5	**29.5**	0.99	142.89	191.3	**29.5**
PH1B	Access Rd	0.088	15	0.73	61.77	11.0	**11.0**	0.73	83.56	14.8	**17.6**	0.91	142.89	31.7	**17.6**
Total		0.573				77.1	**39.0**			104.3	**47.1**			223.0	**47.1**

Pre-Flows

Notes
1) Intensity, $I_2 = 732.951/(Tc+6.199)^{0.810}$ (2-year, City of Ottawa)
2) Intensity, $I_5 = 998.071/(Tc+6.035)^{0.814}$ (5-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 D3
Estimate of Storage Required for 2-yr and 100-yr Storms (Modified Rational Method)

Area No: PH1A Parking Lot
C_{AVG} = 0.79 (2-yr, 5-yr)
C_{AVG} = 0.99 (100-yr +25%)
Time Interval = 15 (mins)
Drainage Area = 0.4855 (hectares)

Duration, T_D (min)	Release Rate = 28.0 (L/sec); Return Period = 2 (years); IDF Parameters, A = 732.951, B = 0.810, C = 6.199 ($I = A/(T_D+C)^B$)					Release Rate = 29.5 (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	167.2	179.1	28.04	151.1	0	398.6	533.7	29.527	504.2	0.0
15	61.8	66.2	28.04	38.1	**34**	142.9	191.3	29.527	161.8	145.6
30	40.0	42.9	28.04	14.9	27	91.9	123.0	29.527	93.5	168.3
45	30.2	32.4	28.04	4.4	12	69.1	92.5	29.527	62.9	**169.9**
60	24.6	26.3	28.04	-1.7	-6	55.9	74.8	29.527	45.3	163.1
75	20.8	22.3	28.04	-5.7	-26	47.3	63.3	29.527	33.7	151.8
90	18.1	19.4	28.04	-8.6	-46	41.1	55.0	29.527	25.5	137.8
105	16.1	17.3	28.04	-10.8	-68	36.5	48.9	29.527	19.3	121.8
120	14.6	15.6	28.04	-12.4	-90	32.9	44.0	29.527	14.5	104.5
135	13.3	14.2	28.04	-13.8	-112	30.0	40.2	29.527	10.6	86.1
150	12.3	13.1	28.04	-14.9	-134	27.6	37.0	29.527	7.4	67.0
165	11.4	12.2	28.04	-15.9	-157	25.6	34.3	29.527	4.8	47.1
180	10.6	11.4	28.04	-16.7	-180	23.9	32.0	29.527	2.5	26.7
195	10.0	10.7	28.04	-17.3	-203	22.4	30.0	29.527	0.5	5.9
210	9.4	10.1	28.04	-18.0	-226	21.1	28.3	29.527	-1.2	-15.3
225	8.9	9.6	28.04	-18.5	-250	20.0	26.8	29.527	-2.7	-36.9
240	8.5	9.1	28.04	-19.0	-273	19.0	25.4	29.527	-4.1	-58.8
255	8.1	8.7	28.04	-19.4	-297	18.1	24.2	29.527	-5.3	-80.9
270	7.7	8.3	28.04	-19.8	-320	17.3	23.2	29.527	-6.4	-103.2
285	7.4	7.9	28.04	-20.1	-344	16.6	22.2	29.527	-7.4	-125.8
300	7.1	7.6	28.04	-20.4	-368	15.9	21.3	29.527	-8.3	-148.5
315	6.8	7.3	28.04	-20.7	-392	15.3	20.5	29.527	-9.1	-171.4
Maximum Storage Requried =					**34.3**					**169.9**

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 D4
Estimate of Storage Required for 2-yr and 100-yr Storms (Modified Rational Method)

Area No: PH1B Access Road
C_{AVG} = 0.73 (2-yr, 5-yr)
C_{AVG} = 0.91 (100-yr +25%)
Time Interval = 15 (mins)
Drainage Area = 0.0879 (hectares)

Duration, T_D (min)	Release Rate = 11.0 (L/sec); Return Period = 2 (years); IDF Parameters, A = 732.951, B = 0.810, C = 6.199 ($I = A/(T_D+C)^B$)					Release Rate = 17.6 (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	167.2	29.6	10.95	18.7	**0**	398.6	88.3	17.612	70.7	0.0
15	61.8	11.0	10.95	0.0	**0**	142.9	31.7	17.612	14.1	**12.7**
30	40.0	7.1	10.95	-3.9	-7	91.9	20.4	17.612	2.7	4.9
45	30.2	5.4	10.95	-5.6	-15	69.1	15.3	17.612	-2.3	-6.2
60	24.6	4.4	10.95	-6.6	-24	55.9	12.4	17.612	-5.2	-18.8
75	20.8	3.7	10.95	-7.3	-33	47.3	10.5	17.612	-7.1	-32.1
90	18.1	3.2	10.95	-7.7	-42	41.1	9.1	17.612	-8.5	-45.9
105	16.1	2.9	10.95	-8.1	-51	36.5	8.1	17.612	-9.5	-60.0
120	14.6	2.6	10.95	-8.4	-60	32.9	7.3	17.612	-10.3	-74.3
135	13.3	2.4	10.95	-8.6	-70	30.0	6.6	17.612	-11.0	-88.8
150	12.3	2.2	10.95	-8.8	-79	27.6	6.1	17.612	-11.5	-103.4
165	11.4	2.0	10.95	-8.9	-88	25.6	5.7	17.612	-11.9	-118.2
180	10.6	1.9	10.95	-9.1	-98	23.9	5.3	17.612	-12.3	-133.0
195	10.0	1.8	10.95	-9.2	-107	22.4	5.0	17.612	-12.6	-147.9
210	9.4	1.7	10.95	-9.3	-117	21.1	4.7	17.612	-12.9	-162.9
225	8.9	1.6	10.95	-9.4	-127	20.0	4.4	17.612	-13.2	-177.9
240	8.5	1.5	10.95	-9.4	-136	19.0	4.2	17.612	-13.4	-193.0
255	8.1	1.4	10.95	-9.5	-146	18.1	4.0	17.612	-13.6	-208.1
270	7.7	1.4	10.95	-9.6	-155	17.3	3.8	17.612	-13.8	-223.2
285	7.4	1.3	10.95	-9.6	-165	16.6	3.7	17.612	-13.9	-238.4
300	7.1	1.3	10.95	-9.7	-174	15.9	3.5	17.612	-14.1	-253.6
315	6.8	1.2	10.95	-9.7	-184	15.3	3.4	17.612	-14.2	-268.9
Maximum Storage Requried =					**0.0**					**12.7**

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 D5
Estimate of Provided Storage for 2-yr and 100-yr Storms

Total 2 Year Storage = ***35.1*** m^3
Total 100 Year Storage= ***218.8*** m^3

2 Year Catch Basin Storage

CB ID	Area (m2)	Depth (m)	Volume (m3)
CB 102	0.36	2.1	0.77
CB 103	0.36	2.1	0.77
CB 104	0.36	2.1	0.77
CB 105	0.36	2.1	0.77
CB 106	0.36	2.1	0.77
CB 107	0.36	2.1	0.77
CB 108	0.36	2.1	0.77
		Total Volume	***5.4***

2 Year Catch Basin Manhole Storage

MH ID	Diameter (mm)	Area (m2)	Depth (m)	Volume (m3)
CBMH 203	1200	1.13	3.00	3.39
CBMH 204	1200	1.13	3.20	3.62
CBMH 205	1200	1.13	3.00	3.39
			Total Volume	***7.0***

2 Year Underground Pipe Storage

Diameter (mm)	Area (m2)	Length (m)	Volume (m3)
200	0.031	119.6	3.8
600	0.283	67.0	18.9
		Total Volume	***22.7***

100 Year Surface Ponding Volumes PH1A

Ponding Location	Surface Area (m2)	Ponding Depth (m)	Volume (m3)
CB102	132	0.30	13.2
CB103	368.00	0.30	36.8
CB104	342.00	0.30	34.2
CB105	86.0	0.30	8.6
CB106	82.0	0.30	8.2
CB107	309.0	0.30	30.9
CB108	401.0	0.30	40.1
CBMH203	184	0.35	21.5
		Total Volume	***193.5***

100 Year Surface Ponding Volumes PH1B

Ponding Location	Surface Area (m2)	Ponding Depth (m)	Volume (m3)
CB101	217	0.35	25.3
		Total Volume	***25.3***

Table D6
Orifice Sizing

CBMH203

Event	Flow (L/s)	Head (m)	ORIFICE AREA(m^2)	SQUARE (1-side mm)	CIRC (mmØ)
2 Year	28.0	2.75	0.006	80	90
100 Year	29.5	3.05	0.006	80	90

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

Orifice Invert =	74.26 m
Ponding Elevation =	77.35 m
Top of CB Elevation =	77.00 m

Note: Orifice is located on the downstream invert of CBMH203

Table D7
Orifice Sizing

CB101

Event	Flow (L/s)	Head (m)	ORIFICE AREA(m^2)	SQUARE (1-side mm)	CIRC (mmØ)
2 Year	0.0	0.00	0.004	66	75
100 Year	17.6	2.25	0.004	66	75

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

Orifice Invert =	74.30	m
Ponding Elevation =	76.60	m
Top of CB Elevation =	76.25	m

Note: Orifice is located on the downstream invert of CB101

Appendix A2 – SWM Phase 2 Design Sheets

Table D8
CALCULATION OF AVERAGE RUNOFF COEFFICIENTS (POST-DEVELOPMENT)

Area No.	Area	Asphalt & Concrete 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				
PH2A	Parking Lot	3910.0	3519.0	245.0	220.5			826.4	165.3	3904.8	4981	0.78
PH2B	Access Rd	837.0	753.3	127.0	114.3			219.0	43.8	911.4	1183	0.77
PH2C	New Bld			1450.0	1305.0					1305.0	1450	0.90
PH2D	Free Flow	165.7	149.1					284.3	56.9	206.0	450	0.46
							Average Runoff Coeff =			C_{AVG} =	$\frac{6,368}{8,125}$	= 0.78

Table D9
SUMMARY OF POST DEVELOPMENT RUNOFF (UNCONTROLLED AND CONTROLLED)

Area No	Outlet Location	Area (ha)	Time of Conc. T_c (min)	Storm = 2-year				Storm = 100-year			
				C_{AVG}	I_2 (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_{100} (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)
PH2A	Parking Lot	0.498	15	0.78	61.77	67.1	**28.0**	0.98	142.89	193.9	**29.5**
PH2B	Access Rd	0.118	15	0.77	61.77	15.6	**15.6**	0.96	142.89	45.3	**17.6**
PH2C	New Bld	0.145	15	0.90	61.77	22.4	**7.0**	1.00	142.89	57.6	**10.0**
PH2D	Free Flow	0.045	15	0.46	61.77	3.5	**3.5**	0.57	142.89	10.2	**10.2**
Total		0.616				82.7	**54.2**			239.2	**67.4**

Notes
1) Intensity, $I_2 = 732.951/(Tc+6.199)^{0.810}$ (2-year, City of Ottawa)
2) Intensity, $I_5 = 998.071/(Tc+6.035)^{0.814}$ (5-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 D10
Estimate of Storage Required for 2-yr and 100-yr Storms (Modified Rational Method)

Area No: PH2A Parking Lot
C_{AVG} = 0.78 (2-yr, 5-yr)
C_{AVG} = 0.98 (100-yr +25%)
Time Interval = 15 (mins)
Drainage Area = 0.4981 (hectares)

Duration, T_D (min)	Release Rate = 28.0 (L/sec) Return Period = 2 (years) IDF Parameters, A = 732.951 ($I = A/(T_D+C)^B$			, B = 0.810 , C = 6.199		Release Rate = 29.5 (L/sec) Return Period = 100 (years) IDF Parameters, A = 1735.688 ($I = A/(T_D+C)^B$			0.820 , 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	167.2	181.5	28.04	153.5	0	398.6	540.9	29.527	511.4	0.0
15	61.8	67.1	28.04	39.0	**35**	142.9	193.9	29.527	164.4	147.9
30	40.0	43.5	28.04	15.4	28	91.9	124.7	29.527	95.1	171.2
45	30.2	32.8	28.04	4.8	13	69.1	93.7	29.527	64.2	**173.3**
60	24.6	26.7	28.04	-1.4	-5	55.9	75.8	29.527	46.3	166.7
75	20.8	22.6	28.04	-5.4	-24	47.3	64.1	29.527	34.6	155.7
90	18.1	19.7	28.04	-8.3	-45	41.1	55.8	29.527	26.3	141.8
105	16.1	17.5	28.04	-10.5	-66	36.5	49.5	29.527	20.0	126.0
120	14.6	15.8	28.04	-12.2	-88	32.9	44.6	29.527	15.1	108.8
135	13.3	14.4	28.04	-13.6	-110	30.0	40.7	29.527	11.2	90.5
150	12.3	13.3	28.04	-14.7	-133	27.6	37.5	29.527	7.9	71.4
165	11.4	12.3	28.04	-15.7	-155	25.6	34.7	29.527	5.2	51.7
180	10.6	11.5	28.04	-16.5	-178	23.9	32.4	29.527	2.9	31.4
195	10.0	10.8	28.04	-17.2	-201	22.4	30.4	29.527	0.9	10.6
210	9.4	10.2	28.04	-17.8	-224	21.1	28.7	29.527	-0.8	-10.5
225	8.9	9.7	28.04	-18.4	-248	20.0	27.2	29.527	-2.4	-32.0
240	8.5	9.2	28.04	-18.8	-271	19.0	25.8	29.527	-3.7	-53.8
255	8.1	8.8	28.04	-19.3	-295	18.1	24.6	29.527	-5.0	-75.9
270	7.7	8.4	28.04	-19.7	-318	17.3	23.5	29.527	-6.1	-98.2
285	7.4	8.0	28.04	-20.0	-342	16.6	22.5	29.527	-7.1	-120.7
300	7.1	7.7	28.04	-20.3	-366	15.9	21.6	29.527	-8.0	-143.4
315	6.8	7.4	28.04	-20.6	-390	15.3	20.7	29.527	-8.8	-166.2
Maximum Storage Requried =					**35.1**					**173.3**

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 D11
Estimate of Storage Required for 2-yr and 100-yr Storms (Modified Rational Method)

Area No: PH2B Access Road
C_{AVG} = 0.77 (2-yr, 5-yr)
C_{AVG} = 0.96 (100-yr +25%)
Time Interval = 15 (mins)
Drainage Area = 0.1183 (hectares)

Duration, T_D (min)	Release Rate = 15.6 (L/sec) Return Period = 2 (years) IDF Parameters, A = 732.951 ($I = A/(T_D+C)^B$			, B = 0.810 , C = 6.199		Release Rate = 17.6 (L/sec) Return Period = 100 (years) IDF Parameters, A = 1735.688 ($I = A/(T_D+C)^B$			, C =	0.820 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	167.2	42.4	15.65	26.7	**0**	398.6	126.2	17.612	108.6	0.0
15	61.8	15.6	15.65	0.0	**0**	142.9	45.3	17.612	27.6	**24.9**
30	40.0	10.1	15.65	-5.5	-10	91.9	29.1	17.612	11.5	20.7
45	30.2	7.7	15.65	-8.0	-22	69.1	21.9	17.612	4.3	11.5
60	24.6	6.2	15.65	-9.4	-34	55.9	17.7	17.612	0.1	0.3
75	20.8	5.3	15.65	-10.4	-47	47.3	15.0	17.612	-2.6	-11.9
90	18.1	4.6	15.65	-11.1	-60	41.1	13.0	17.612	-4.6	-24.8
105	16.1	4.1	15.65	-11.6	-73	36.5	11.6	17.612	-6.1	-38.1
120	14.6	3.7	15.65	-12.0	-86	32.9	10.4	17.612	-7.2	-51.8
135	13.3	3.4	15.65	-12.3	-99	30.0	9.5	17.612	-8.1	-65.7
150	12.3	3.1	15.65	-12.5	-113	27.6	8.7	17.612	-8.9	-79.8
165	11.4	2.9	15.65	-12.8	-126	25.6	8.1	17.612	-9.5	-94.1
180	10.6	2.7	15.65	-13.0	-140	23.9	7.6	17.612	-10.0	-108.4
195	10.0	2.5	15.65	-13.1	-154	22.4	7.1	17.612	-10.5	-122.9
210	9.4	2.4	15.65	-13.3	-167	21.1	6.7	17.612	-10.9	-137.5
225	8.9	2.3	15.65	-13.4	-181	20.0	6.3	17.612	-11.3	-152.2
240	8.5	2.1	15.65	-13.5	-194	19.0	6.0	17.612	-11.6	-166.9
255	8.1	2.0	15.65	-13.6	-208	18.1	5.7	17.612	-11.9	-181.7
270	7.7	2.0	15.65	-13.7	-222	17.3	5.5	17.612	-12.1	-196.6
285	7.4	1.9	15.65	-13.8	-236	16.6	5.2	17.612	-12.4	-211.5
300	7.1	1.8	15.65	-13.9	-249	15.9	5.0	17.612	-12.6	-226.4
315	6.8	1.7	15.65	-13.9	-263	15.3	4.8	17.612	-12.8	-241.4
Maximum Storage Requried =					**0.0**					**24.9**

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 D12
Estimate of Storage Required for 2-yr and 100-yr Storms (Modified Rational Method)

Area No: PH2C Roof
C_{AVG} = 0.90 (2-yr, 5-yr)
C_{AVG} = 1.00 (100-yr +25%)
Time Interval = 15 (mins)
Drainage Area = 0.1450 (hectares)

Duration, T_D (min)	Release Rate = 7.0 (L/sec); Return Period = 2 (years); IDF Parameters, A = 732.951, B = 0.810, C = 6.199 ($I = A/(T_D+C)^B$)					Release Rate = 10.0 (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	167.2	60.7	7.00	53.7	0	398.6	160.7	10.000	150.7	0.0
15	61.8	22.4	7.00	15.4	**14**	142.9	57.6	10.000	47.6	42.8
30	40.0	14.5	7.00	7.5	14	91.9	37.0	10.000	27.0	**48.7**
45	30.2	11.0	7.00	4.0	11	69.1	27.8	10.000	17.8	48.2
60	24.6	8.9	7.00	1.9	7	55.9	22.5	10.000	12.5	45.1
75	20.8	7.6	7.00	0.6	2	47.3	19.0	10.000	9.0	40.7
90	18.1	6.6	7.00	-0.4	-2	41.1	16.6	10.000	6.6	35.5
105	16.1	5.9	7.00	-1.1	-7	36.5	14.7	10.000	4.7	29.7
120	14.6	5.3	7.00	-1.7	-12	32.9	13.3	10.000	3.3	23.5
135	13.3	4.8	7.00	-2.2	-18	30.0	12.1	10.000	2.1	16.9
150	12.3	4.4	7.00	-2.6	-23	27.6	11.1	10.000	1.1	10.2
165	11.4	4.1	7.00	-2.9	-28	25.6	10.3	10.000	0.3	3.2
180	10.6	3.9	7.00	-3.1	-34	23.9	9.6	10.000	-0.4	-3.9
195	10.0	3.6	7.00	-3.4	-40	22.4	9.0	10.000	-1.0	-11.2
210	9.4	3.4	7.00	-3.6	-45	21.1	8.5	10.000	-1.5	-18.6
225	8.9	3.2	7.00	-3.8	-51	20.0	8.1	10.000	-1.9	-26.1
240	8.5	3.1	7.00	-3.9	-57	19.0	7.7	10.000	-2.3	-33.7
255	8.1	2.9	7.00	-4.1	-62	18.1	7.3	10.000	-2.7	-41.3
270	7.7	2.8	7.00	-4.2	-68	17.3	7.0	10.000	-3.0	-49.1
285	7.4	2.7	7.00	-4.3	-74	16.6	6.7	10.000	-3.3	-56.9
300	7.1	2.6	7.00	-4.4	-80	15.9	6.4	10.000	-3.6	-64.7
315	6.8	2.5	7.00	-4.5	-85	15.3	6.2	10.000	-3.8	-72.6
Maximum Storage Requried =					**13.9**					**48.7**

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 D13
Estimate of Provided Storage for 2-yr and 100-yr Storms

Total 2 Year Storage = ***35.1*** **m^3**
Total 100 Year Storage= ***275.8*** **m^3**

2 Year Catch Basin Storage

CB ID	Area (m2)	Depth (m)	Volume (m3)
CB 102	0.36	2.1	0.77
CB 103	0.36	2.1	0.77
CB 104	0.36	2.1	0.77
CB 105	0.36	2.1	0.77
CB 106	0.36	2.1	0.77
CB 107	0.36	2.1	0.77
CB 108	0.36	2.1	0.77
		Total Volume	***5.4***

2 Year Catch Basin Manhole Storage

MH ID	Diameter (mm)	Area (m2)	Depth (m)	Volume (m3)
CBMH 203	1200	1.13	3.0	3.39
CBMH 204	1200	1.13	3.2	3.62
CBMH 205	1200	1.13	3.0	3.39
			Total Volume	***7.0***

2 Year Underground Pipe Storage

Diameter (mm)	Area (m2)	Length (m)	Volume (m3)
200	0.031	119.6	3.8
600	0.283	67.0	18.9
		Total Volume	***22.7***

100 Year Surface Ponding Volumes PH1A

Ponding Location	Surface Area (m2)	Ponding Depth (m)	Volume (m3)
CB102	132	0.30	13.2
CB103	368.00	0.30	36.8
CB104	342.00	0.30	34.2
CB105	86.0	0.30	8.6
CB106	82.0	0.30	8.2
CB107	309.0	0.30	30.9
CB108	401.0	0.30	40.1
CBMH203	184	0.35	21.5
		Total Volume	***193.5***

100 Year Surface Ponding Volumes PH1C

Ponding Location	Surface Area (m2)	Ponding Depth (m)	Volume (m3)
Roof	1140	0.15	57
		Total Volume	***57.0***

100 Year Surface Ponding Volumes PH1B

Ponding Location	Surface Area (m2)	Ponding Depth (m)	Volume (m3)
CB101	217	0.35	25.3
		Total Volume	***25.3***

Table D14
Orifice Sizing

CBMH203

Event	Flow (L/s)	Head (m)	ORIFICE AREA(m^2)	SQUARE (1-side mm)	CIRC (mmØ)
2 Year	28.0	2.75	0.006	80	90
100 Year	29.5	3.05	0.006	80	90

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

Orifice Invert =	74.26	m
Ponding Elevation =	77.35	m
Top of CB Elevation =	77.00	m

Note: Orifice is located on the downstream invert of CBMH203

Table D15
Orifice Sizing

CB101

Event	Flow (L/s)	Head (m)	ORIFICE AREA(m^2)	SQUARE (1-side mm)	CIRC (mmØ)
2 Year	0.0	0.00	0.004	66	75
100 Year	17.6	2.25	0.004	66	75

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

Orifice Invert =	74.30	m
Ponding Elevation =	76.60	m
Top of CB Elevation =	76.25	m

Note: Orifice is located on the downstream invert of CBMH101

Appendix B – Water

TABLE 1: FIRE FLOW REQURIEMENTS BASED ON FIRE UNDERWRITERS SURVEY(FUS) 1999
Project No.: OTT-00238504-A0
Project Name: KMA Mosque & Community Center

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

F = 220 * C * SQRT(A)

where: F = required fire flow in litres per minute
A = total floor area in m^2 (including all storeys, but excluding basements at least 50% below grade)
C = coefficient related to the type of construction

Task	Options	Multiplier	Input	Value Used	Fire Flow Total (L/min)
Choose Building Frame (C)	Wood Frame	1.5	Non-combustible Construction	0.8	
	Ordinary Construction	1			
	Non-combustible Construction	0.8			
	Fire Resistive Construction	0.6			
Input Building Floor Areas (A)	Floor 3			2282.4 m²	
	Floor 2		586.4		
	Floor 1		1696		
	Basement (At least 50% below grade, not included)		0		
Fire Flow (F)	F = 220 * C * SQRT(A)				8,408
Fire Flow (F)	Rounded to nearest 1,000				**8,000**

Reductions/Increases Due to Factors Effecting Burning

Task	Options	Multiplier	Input	Value Used	Fire Flow Change (L/min)	Fire Flow Total (L/min)
Choose Combustibility of Building Contents	Non-combustible	-25%	Limited Combustible	-15%	-1,200	6,800
	Limited Combustible	-15%				
	Combustible	0%				
	Free Burning	15%				
	Rapid Burning	25%				
Choose Reduction Due to Sprinkler System	**Adequate** Sprinkler Conforms to NFPA13	-30%	Adequate Sprinkler Conforms to NFPA13	-23%	-1,591	5,209
	No Sprinkler	0%				
	Standard Water Supply for Fire Department Hose Line and for Sprinkler System	-10%	Standard Water Supply for Fire Department Hose Line and for Sprinkler System	-8%	-530	4,678
	Not Standard Water Supply or Unavailable	0%				
	Fully Supervised Sprinkler System	-10%	Fully Supervised Sprinkler System	-8%	-530	4,148
	Not Fully Supervised or N/A	0%				

Task	Exposures	Separ-ation Dist (m)	Cond	Separation Conditon	Exposing Wall type	Exposed Wall Length: Length (m)	No of Storeys	Lenth-height Factor	Sub-Conditon	Charge (%)	Total Charge (%)	Total Exposure Charge (L/min)	
Choose Structure Exposure Distance	Side 1	80.5	6	> 45.1	Type A	29	1	29	6	0%	0%	0	4,148
	Side 2	100	6	> 45.1	Type A	0	0	0	6	0%			
	Front	62.5	6	> 45.1	Type A	16.4	2	32.8	6	0%			
	Back	100	6	> 45.1	Type A	0	0	0	6	0%			
Obtain Required Fire Flow	Total Required Fire Flow, Rounded to the Nearest 1,000 L/min =												**4,000**
	Total Required Fire Flow, L/s =												**67**

Exposure Charges for Exposing Walls of Wood Frame Construciton (from Table G5)

Type A	Wood-Frame or non-conbustible
Type B	Ordinary or fire-resisitve with unprotected openings
Type C	Ordinary or fire-resisitve with semi-protected openings
Type D	Ordinary or fire-resisitve with nlank wall

Conditons for Separation

Separation Dist	Condition
0m to 3m	1
3.1m to 10m	2
10.1m to 20m	3
20.1m to 30m	4
30.1m to 45m	5
> 45.1m	6

351 Sandhill Road Boundary Conditions

Information Provided:

Date provided: April 2017

Scenario	Demand	
	L/min	L/s
Average Daily Demand	30	0.5
Maximum Daily Demand	45	0.75
Peak Hour	81	1.35
Fire Flow Demand	9000	150

Location:

Results:

Connection 1 - Sandhill Road

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	131.2	76.6
Peak Hour	125.0	67.6
Max Day plus Fire (9,000 l/min)	123.2	65.1

[1] Ground Elevation = 77.4 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.

KMA MOSQUE AND COMMUNITY CENTRE
Client:KANATA MUSLIM ASSOCIATION
Project: OTT-00238504-A0

Prepared By: M.Lafleur
Date: June 2018

Max day(0.75L/s) + FireFlow(150L/s) HGL= 123.2 m
Max HGL= 131.2 m
Peak Hour= 125 m

Table B1 Pressure Analysis

Description	From	To	Flow (L/sec)	Pipe Dia (mm)	Dia (m)	Q (m^3/sec)	Area (m2)	C	Velocity V (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	Building	150.75	200	0.200	0.15075	0.0314159	110	4.7985	0.1347	55	7.410624956	13.2	1.77855	9.18917	76.40	77.60	-1.20	459.0	(66.6)	357.1	(51.8)	14.8
Max HGL	Main	Building	0.5	200	0.200	0.0005	0.0314159	110	0.0159	0.0000	35	0.000120759	13.2	0.00005	0.00017	76.40	77.60	-1.20	537.4	(77.9)	525.7	(76.2)	1.7
Peak Hour	Main	Building	1.35	200	0.200	0.00135	0.0314159	110	0.043	0.0000	35	0.000759986	13.2	0.00029	0.00105	76.40	77.60	-1.20	476.6	(69.1)	464.8	(67.4)	1.7

V=Q/A

Slope of HGL= $\left(\frac{3.59}{C}\right)^{1.852} \frac{Q}{D^{4.87}}^{1.852}$

hf = Slope of HGL * Pipe Length

Resistance of Fittings and Valves for 200mm WM

Fittings	Loss in Equiv. Length in Pipe Diameters	Equiv. Length (metres)	Quantity (each)	Total Equiv. Length (m)
Standard 90^0 Elbow	32	6.40	1	6.4
11.25 Degree Elbow	8	1.60	1	1.6
45 Degree Elbow	16	3.20	0	0
Gate Valve Full -Open	13	2.60	2	5.2
		Total:	**4**	**13.2**

Appendix C – Sewer Design Sheets

exp.

SANITARY SEWER CALCULATION SHEET

LOCATION				RESEDENTIAL AREAS AND POPULAITONS												COMMERCIAL			INDUSTRIAL			INSTITUTIONAL		INFILTRATION				SEWER DATA						
					NUMBER OF UNITS							POPULATION				AREA (ha)			AREA (ha)					AREA (ha)										
Street	U/S MH	D/S MH	Desc	Area (ha)	Singles	Semis	Towns	1-Bed Apt.	2-Bed Apt.	3-Bed Apt.	4-Bed Apt.	INDIV	ACCU	Peak Factor	Peak Flow (L/sec)	INDIV	ACCU	Peak Flow (L/sec)	INDIV	ACCU	Peak Factor (per MOE)	AREA (Ha)	ACCU AREA (Ha)	INDIV	ACCU	INFILT FLOW (L/s)	TOTAL FLOW (L/s)	Nom Dia (mm)	Actual Dia (mm)	Slope (%)	Length (m)	Capacity (L/sec)	Q/Q_{CAP} (%)	Full Velocity (m/s)
Private		303		0.8180																		0.82	0.82	0.818	0.818	0.27	0.67	150	154.05	0.60	42.200	12.66524	0.05271	0.93970312
	303	302																										150	154.05	0.60	19.100	12.66524		0.93970312
	302	301																										150	154.05	0.60	10.100	12.66524		0.93970312
	301	Ex.MH																										150	154.05	0.60	12.800	12.66524		0.93970312
				0.818																				**0.818**										

Residential Avg. Daily Flow, q (L/p/day) = 280
Commercial Avg. Daily Flow (L/gross ha/day) = 28,000
or L/gross ha/sec = 0.324
Institutianal Avg. Daily Flow (L/s/ha) = 28,000
or L/gross ha/sec = 0.324
Light Industrial Flow (L/gross ha/day) = 35,000
or L/gross ha/sec = 0.40509
Light Industrial Flow (L/gross ha/day) = 55,000
or L/gross ha/sec = 0.637

Commercial Peak Factor = 1.5 (when area >20%)
1.0 (when area <20%)
Institutional Peak Factor = 1.5 (when area >20%)
1.0 (when area <20%)
Residential Correction Factor, K = 0.80
Manning N = 0.013
Peak extraneous flow, I (L/s/ha) = 0.33 (Total I/I)

Peak Population Flow, (L/sec) = P*q*M/86.4
Peak Extraneous Flow, (L/sec) = I*Ac
Residential Peaking Factor, M = 1 + (14/(4+P^0.5)) * K
A_c = Cumulative Area (hectares)
P = Population (thousands)
Sewer Capacity, Qcap (L/sec) = $1/N \; S^{1/2} R^{2/3} A_c$
(Manning's Equation)

Unti Type	Persons/Unit
Singles	3.4
Semi-Detached	5.7
Townhomes	2.7
Single Apt. Unit	1.4
2-bed Apt. Unit	2.1
3-bed Apt. Unit	3.1
4-bed Apt. Unit	3.8

Designed:	Project:
A. Elgayar, M.A.Sc.	KMA Mosque and Community Center
Checked:	Location:
A. Ansari, P.Eng.	Ottawa, Ontario
File Reference:	Page No:
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TABLE C-1: 100-YEAR STORM SEWER CALCULATION SHEET

Return Period Storm = **5** (5-year)
Default Inlet Time= 15 (minutes)
Manning Coefficient = 0.013 (dimensionless)

LOCATION			AREA (hectares)			FLOW (UNRESTRICTED)						SEWER DATA										
Location	From Node	To Node	Area No.	Area (ha)	Average R	Indiv. 2.78*A*R	Accum. 2.78*A*R	Tc (mins)	I (mm/h)	Return Period	Q (L/sec)	Dia (mm) Actual	Dia (mm) Nominal	Type	Slope (%)	Length (m)	Capacity (L/sec)	Velocity (m/s) Vf	Velocity (m/s) Va	Time in Pipe, Tt (min)	Hydraulic Ratios Qa/Qf	Hydraulic Ratios Va/Vf
351 Sandhill Road	CB105	CBMH205	PH2A	0.0300	0.78	0.07	0.07	15.00	83.56	5.00	5.4	201.16	200	PVC	0.50	18.60	23.6	0.74	0.49	0.63	0.23	0.67
	CB106	CBMH205	PH2A	0.0450	0.78	0.10	0.10	15.00	83.56	5.00	8.2	201.16	200	PVC	0.50	19.50	23.6	0.74	0.52	0.63	0.35	0.70
	CB104	CBMH205	PH2A	0.060	0.78	0.13	0.13	15.00	83.56	5.00	10.9	201.16	200	PVC	0.50	17.10	23.6	0.74	0.52	0.55	0.46	0.71
	CB107	CBMH205	PH2A	0.080	0.78	0.17	0.17	15.00	83.56	5.00	14.5	201.16	200	PVC	0.50	13.10	23.6	0.74	0.67	0.33	0.62	0.91
	CB103	CBMH205	PH2A	0.090	0.78	0.20	0.20	15.00	83.56	5.00	16.3	201.16	200	PVC	0.50	17.10	23.6	0.74	0.70	0.41	0.69	0.95
	CB108	CBMH205	PH2A	0.100	0.78	0.22	0.22	15.00	83.56	5.00	18.1	201.16	200	PVC	0.50	17.10	23.6	0.74	0.72	0.39	0.77	0.98
	CBMH205	CBMH204					0.88	15.63	81.59	5.00	71.6	610	600	PVC	0.50	45.30	453.7	1.54	0.91	0.83	0.16	0.59
	CB102	CBMH204	PH2A	0.047	0.78	0.10	0.10	15.00	83.56	5.00	8.5	201.16	200	PVC	0.50	17.10	23.6	0.74	0.52	0.55	0.36	0.71
	CBMH204	CBMH203					0.98	16.46	79.12	5.00	77.5	610	600	PVC	0.50	21.70	453.7	1.54	0.97	0.37	0.17	0.63
	CBMH101	CBMH203	PH2B	0.120	0.77	0.26	0.26	15.00	83.56	5.00	21.5	201.16	200	PVC	0.50	2.71	23.6	0.74	0.74	0.06	0.91	1.00
	ROOF	CBMH203	PH2C	0.150	0.90	0.38	0.38	15.00	83.56	5.00	31.4	201.16	200	PVC	1.00	15.00	33.3	1.04	1.09	0.23	0.94	1.04
	CBMH203	STMMH202					1.61	16.83	78.07	5.00	125.9	610	600	PVC	0.50	67.00	453.7	1.54	1.07	1.04	0.28	0.70
	STMMH202	STMMH201					1.61	17.87	75.29	5.00	121.4	610	600	PVC	0.50	15.00	453.7	1.54	1.07	0.23	0.27	0.70
TOTALS =																**286.31**						

Definitions:
Q = 2.78*AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Watershed Area (hectares)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficients (dimensionless)

Notes:
Ottawa Rainfall Intensity Values:
From Sewer Desing Guidelines, 2004

	5yr
a =	998.071
b=	0.814
c =	6.053

Designed:
A. Elgayar

Project:
351 Sand Hill Road

Checked:
A. Ansari, PEng.

Location:
Ottawa, Ontario

Dwg Reference:
SSGP-2

File Ref:
238504 - STM Design

Sheet No:
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