DesignFine

CONSULTING ENGINEERS – FORENSIC ENGINEERS – PROJECT MANAGERS
96 Kennedy Rd. South, Suite #207, Brampton, ON L6W 3E7

FUNCTIONAL SERVICING
&
STORMWATER MANAGEMENT REPORT

7211 & 7233 AIRPORT RD
PARTS 1,2, & 3
CITY OF MISSISSAUGA
REGION OF PEEL

MAY 2023

PREPARED BY:

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1.0 INTRODUCTION

Design Fine Ltd. Was retained by Airstar Holdings Inc. to complete a Functional Servicing and Storm Water Management Report in support of an Official Plan and Zoning By-law amendment for the property at 7211 and 7233 Airport Road. The 8656 m^2 property is legally described as Part of Lot 12, Registered Plan 43R-23708, Pin # 13272-0613 (LT) and 13272-0614 (LT), City of Mississauga, Regional Municipality of Peel. The property is bounded by Airport Road to the south, Collett Road to the west, Residential area to the west and east, and Victory Park to the north. The site location is illustrated in Figure 1.

The proponent plans to construct a multi-unit senior's residence complex with a total of 128 units, as well 4 commercial units located on the main floor. A large underground parking structure will encompass most of the site area to support the proposed building.

2.0 SITE DESCRIPTION

The site is currently vacant. Access to the site is provided from one driveway onto Airport Road at the South limits of the property and another from Collett Road at the west limits of the property (refer to attached Drawing A-100 for more detail). The site predominantly drains South-West to North-East overland towards Victory Park. The site is approximately 0.87 hectares in size.

3.0 SITE PROPOSAL

This site will be developed into a Senior's residence which will be comprised of 128 units in a total of five floors. Additionally, on the main floor 4 units will be commercial in nature. Parking will be provided underground for the residents and a separate parking area will be designated for commercial and employee. Detailed site statistics can be found in drawing A-100.

4.0 STORMWATER MANAGEMENT & SITE DRAINGE

Management of storm water and site drainage for the proposed development policies and standards of various agencies including:

- City of Mississauga
- Ministry of Environment (MOE)
- Toronto Region Conservation Authority (TRCA)
- Region of Peel

A description of the existing and proposed drainage conditions as well as proposed storm water quantity and quality controls are described in the sections to follow.

4.1 EXISTING DRAINGE CONDITIONS

The subject land is located at the North of the intersection of Airport Road and Victory Crescent. Land is presently vacant and consists of an undeveloped green field. An existing 600mm diameter storm service in the adjacent lot collects the drainage from the subject property. Subject land also drains towards Airport Road into roadside catch basin and a secondary roadside catch basin is located North on Airport Road. The slope for the subject land is about 1-2% is Existing.

4.2 PROPOSED DRAINAGE CONDITIONS

The site will be developed into a residential/commercial structure in 'L' shape which will consist of one building, with the side perpendicular to Airport Road containing two floors. Building side parallel to Airport Road will contain a total of six floors, part of the first floor will be commercial and the rest residential.

Internal drainage within the proposed development will be collected in the parking lot area with a drain and subsurface storm sewers sized to convey the 100-year event. This storm sewer will be connected to manhole and stormceptor; which will release the water to main storm sewer line with controlled flow located on Collett road.

The preliminary grading of the site has been designed to direct all storm water generated on-site to the proposed internal drainage system. Driveway and parking lot sloped range from 1-2% in accordance with City of Mississauga standards. Low points at the drains have been graded such that the maximum depth of ponding will be 0.25 meters in the event of drain blockage.

4.3 STORMWATER QUANTITY CONTROL

Due to an increase in the site imperviousness because of the proposed development, peak flows from the site will increase. As such, an analysis of the required storage volume was completed to ensure post-development peak flows rates emanating from the site pre-development levels (i.e., quantity control).

The storm water quantity storage requirements for the site were determined using the Modified Rational Method. Rainfall data was collected from the City of Mississauga IDF Standard 2111.010. Refer to Appendix B for detailed storm water management calculations.

Given that the peak flow has substantially increased due to the increase in site imperviousness and drainage area contribution, quantity control measures will be required.

The total storage volume on-site to achieve the above-noted peak flow targets is a maximum of storage of 114.65 m^3 (increased from 114.65 m^3 to 127.56 m^3 due to the reduction in flow caused by orifice tube, please refer to Table 2 in Appendix A for more detail.

Drainage from parking areas will be collected in storage, after which it will pass through Stormceptor EFO6 before being released into the main storm sewer. The specification of the most suitable quantity control method(s) is provided in the Appendix C.

4.4 STORM QUALITY & EROSION CONTROL

It will be easier to implement storm water management practices to address the water quality and erosion control requirements of the regulatory agencies. Since Lake Ontario is the ultimate receiver of drainage, the development will incorporate measure to provide “enhanced protection” per the MOE (2003) guidelines. “Enhanced” water quality protection involves the removal of at least 80% of suspend solids from 90% of the annual runoff volume.

Typical water quality and erosion controls for the treatment of runoff from area size feature a treatment oil/grit separators and infiltration galleries. Storm water quality objectives for can be achieved using a stormceptor EFO6 (or equivalent). The storm water can be collected through 3 drains located on the ground level in parking area and discharge to Collett Rd outlets through EFO6. As per TRCA requirements only credits 50% efficiency. EFO6 is designed based on the drainage area of 1.74 ha (double the size of existing property (0.87 ha)).

5.0 SANITARY SEWAGE SYSTEM

The trunk sanitary sewer is approximately 2m below road surface. We propose 254 mm (10") dia. service connection will be made from the building to existing trunk sanitary sewer below Collett Rd. The proposed development consists of one Long-term facility with some commercial entities present on the main floor of the building. The combined floor area of 15,457 m^2 produces an estimated average day and peak sewage flow of 0.73 L/s and 3.01 L/s respectively.

6.0 WATER DISTRIBUTION SYSTEM

We propose one service connection be made of 152 mm (6") dia. water main on Collett Rd. The building services will include flow meters and connection requirements according to Region of Peel Standards.

Fire protection will be provided by a new hydrant, which will be located on the north side of the site so that it is less than 50m away from the building and provides easy access to each side of the building.

7.0 ROAD AND DRIVEWAY ACCESS

The development plan shows one right-in, right-out access from Airport Road.

8.0 EROSION & SEDIMENTATION CONTROL DURING CONSTRUCTION

Erosion and sediment controls will be implemented on-site prior to construction. The controls will consist of dams.

- Sediment control fence

Sediment control fence will be installed were required to intercept sheet flow. It should be noted that additional silt fencing maybe added during construction based on field decisions by the Engineer and Owner prior to, during and following, the earth works.

- Topsoil Stockpiles

It will be necessary to strip topsoil prior to earth moving. Temporary topsoil stockpiles will be located such that sediment does not enter the adjacent roadside ditches.

- Dust Suppression

During earthwork activities, the Contractor will ensure that measures for dust suppression are provided as required, such as the application of lime water.

A complete sediment and erosion control plan will be developed during the detailed design / approvals process.

8.1 CONSTRUCTION SEQUENCING

The following is the scheduling of construction activities with respect to sediment controls:

- Installation of all silt fences prior to any other activities on the site.
- Construct temporary mud mat for construction access.
- Demolish existing buildings and dispose of waste material off site.
- Excavate the site for the construction of the building foundations and
- dispose of surplus material off site.
- Install the site servicing and all underground utilities.
- Construct the building, underground Parking garage and buildings.
- Restore / re-vegetate all disturbed areas with temporary measures.
- such as mulch or seeding or with final landscape and paving materials.
- Upon stabilization of all disturbed areas, remove sediment controls.

8.2 *INSPECTIONS & MAINTENANCE*

To ensure that the sediment control measures operate effectively, they are to be regular monitored during construction. Inspections of all the erosion and sediment control measures on the construction site should be undertaken with the following frequency:

- On a weekly basis
- After every rainfall
- After significant snow melt
- Prior to forecasted rainfall events.

If damaged is found, the damage should be repaired or replaced within 48 hours. Site inspection staff and construction managers should refer to the Erosion and Sediment Control Inspection Guide (2008) prepared by the Greater Golden Horseshoe Area Conservation

Authority. The guide provides information on inspection reporting, how to respond to variety of problems, and proper installation techniques.

9.0 UTILITIES

As the surrounding area contains large number of commercial and residential properties access to all the major utilities including Enbridge Gas, Hydro One, Rogers, and Bell Canada.

10.0 CONCLUSIONS & RECOMMENDATIONS

We conclude that the proposed development of the subject lands can be readily serviced and meet the storm water management objectives of the regulatory agencies.

1. Access to the site will be provided from Northbound Airport Road adjacent to the site.
2. On-site storm water quantity control will be required. The total storage volume required while accounting orifice tube reduction is 127.56 m^3. Storage is provided via underground tank with a capacity of 45.00 m^3. Furthermore, additional 68.84 m^3 storage can be achieved utilizing, ponding around the drain in the parking area, and around the catch basin (CBMH1). First 5mm of rain (43.28 m^3) shall be managed by way of infiltration and evapotranspiration for the site via landscape areas. There total storage required on site will be 127.56 m^3 – 43.28 m^3 = 84.28 m^3 < 113.84 m^3.
3. On-site storm water quality controls are required and will be achieved using treatment train approach. MOE storm water quality objectives using a Stormceptor EFO6.
4. The expected average domestic water consumption will be approximately 3.18 L/sec.
5. The fire flow required for the site is estimated to be 140.23 L/sec.
6. Internal drainage for the development will convey storm event and emergency overland flows in accordance with City of Mississauga design standards.
7. One sanitary sewer connection will be made to the existing sewer via a proposed 254 mm (10”) Ø service lateral.
8. Domestic water for the commercial uses will be provided by a connection to the existing 152 mm (6”) Ø watermain on Collett Rd.
9. Existing utility plants are located on Airport Rd and Collett Rd can service the proposed site.

Therefore, we recommend approval of the planning applications for the subject lands from the perspective of site grading, storm water management, and engineering servicing requirements.

Regards,

Design Fine Ltd.

Aryan Sharma, P. Eng

APPENDIX A

STORMWATER MANAGEMENT CALCULATIONS

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<u>RUNOFF COEFFCIENT</u>

Pre-Development Peak Flows – As per City of Mississauga design requirements, manual maximum runoff coefficient can be used for the predevelopment condition is 0.5, and for undeveloped land 0.25 should be used. The site is described as underdeveloped greenfield therefore 0.25 is to be used.

Post-Development Peak Flows

Land Use	Area(ha)	Runoff Coef.	A x C
Parks	0.3757	0.25	0.0939
Multiple & Institutional	0.0831= (0.2171-0.1340)	0.90	0.0748
Green Roof area	0.1340	0.45	0.0603
Roadways	0.1965	0.90	0.1768
<u>Total</u>:	0.8656		

Total Area (ha) = 0.8656

Average Runoff Coef. = (0.0939+0.0748+0.0603+0.1768) = 0.4058/0.8656 ha = 0.4688 ~ 0.47

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RATIONAL METHOD FLOWS

Sample Calculation (Post-Development) – 2 years

Intensity (2 years): $i = \frac{610}{(Tc+4.6)^{0.78}}$ (As per City of Mississauga IDF Standard 2111.010)

Peak Flow: $Q_{post} = 0.0028 \times C_{post} \times i_{(T_d)} \times$ Area

Factors:

T_c = 15 minutes as per City of Mississauga Design Criteria
$C_{post} = 0.41$
$Q_{pre} = 0.0595\ m^3/s$
T_d = Time in minutes

2 years

Cpost	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity	59.89	50.16	43.42	38.45	34.60	31.54	29.03	26.94
Td (sec)	900	1200	1500	1800	2100	2400	2700	3000
Mins	15	20	25	30	35	40	45	50
Peak flow (Post)	0.0682	0.0571	0.0495	0.0438	0.0394	0.0359	0.0331	0.0307

5 years

Cpost	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity	80.51	67.43	58.37	51.68	46.52	42.40	39.02	36.21
Td (sec)	900	1200	1500	1800	2100	2400	2700	3000
Mins	15	20	25	30	35	40	45	50
Peak flow (Post)	0.0917	0.0768	0.0665	0.0589	0.0530	0.0483	0.0445	0.0412

<u>10 years</u>

Cpost		0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity		99.17	83.06	71.90	63.66	57.30	52.22	48.07	44.60
Td (sec)		900	1200	1500	1800	2100	2400	2700	3000
	Mins	15	20	25	30	35	40	45	50
Peak flow (Post)		0.1130	0.0946	0.0819	0.0725	0.0653	0.0595	0.0548	0.0508

<u>25 years</u>

Cpost		0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity		113.89	95.40	82.58	73.11	65.80	59.98	55.21	51.22
Td (sec)		900	1200	1500	1800	2100	2400	2700	3000
	Mins	15	20	25	30	35	40	45	50
Peak flow (Post)		0.1297	0.1087	0.0941	0.0833	0.0750	0.0683	0.0629	0.0583

<u>50 years</u>

Cpost		0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity		127.13	106.57	92.30	81.75	73.60	67.10	61.77	57.32
Td (sec)		900	1200	1500	1800	2100	2400	2700	3000
	Mins	15	20	25	30	35	40	45	50
Peak flow (Post)		0.1448	0.1214	0.1051	0.0931	0.0838	0.0764	0.0704	0.0653

<u>100 years</u>

Cpost		0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
Intensity		140.69	118.12	102.41	90.77	81.77	74.58	68.68	63.75
Td (sec)		900	1200	1500	1800	2100	2400	2700	3000
	Mins	15	20	25	30	35	40	45	50
Peak flow (Post)		0.1603	0.1352	0.1173	0.1039	0.0936	0.0854	0.0786	0.0730

Table 1: Maximum Storage required.

Time Min (t)	Intensity Mm/hr. (I)	Max. Discharge m^3/sec. Q(release)	Peak flow 100 yr. Event m^3/sec Q(peak)	Inflow Volume m^3 V(in)	Outflow Volume m^3 V(out)	Storage Required m^3
10	176.31	0.036	0.2008	120.48	30.36	90.12
15	140.69	0.036	0.1603	144.27	40.64	103.63
20	118.12	0.036	0.1352	162.24	50.97	111.27
25	**102.41**	**0.036**	**0.1173**	**175.95**	**61.30**	**114.65**
30	90.77	0.036	0.1039	171.43	71.65	99.78

The maximum storage volume required to control 100-year rainfall event for grade level area is 114.65 m^3.

Water balance (WWFMG Table 3 and 7 section 2.2.1.1 and 2.2.1.2) is estimated for erosion control, ground water recharge and downstream habitat protection though run-off control, infiltration, and evapotranspiration.

STORAGE VOLUME PROVIDED

The maximum storage volume required to control 100-year rainfall event for grade level area: 114.65 m^3.

Pipe Diameter = $\sqrt{\frac{4*Q}{\pi*V}}$

Pipe diameter used = 50 mm

V = 9.81 m^2/s (Gravity)

Where Q = 5 L/s x 7 pipes = 35 L/s (0.035 m^3/s)

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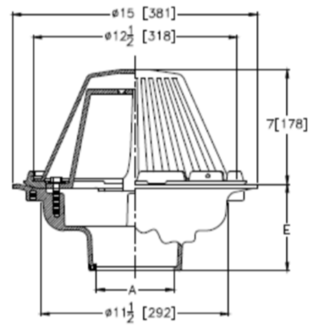
SPECIFICATION DATA



ENGINEERING SPECIFICATION: ZURN Z-105 "Control-Flo" roof drain for dead -level or sloped roof construction, Dura-Coated cast iron body. "Control-Flo" weir shall be linear functioning with integral membrane flashing clamp/ gravel guard and Poly-Dome. All data shall be verified proportional to flow rates.

Roof will control the runoff to a fixed discharge with 7 hoppers. The discharge table for the Zurn hoppers is attached in the appendix.

Proposed storm water sewer system is shown on grading plan. There is 1 drain, and 1 catch basin.

<u>Volume of the Catch Basin</u>:

= (L X W X D) = (0.61 X 0.61 X 1.9) = 0.71 m^3

Where:

L = Length of the catch basin
W = Width of the catch basin
D = Average depth of the catch basin

<u>Surface Ponding storage</u>:

There are two ponding areas provided: one in the parking lot and another in the landscape area.

Total volume provided: 58.93 m^3 + 2.55 m^3 = 61.48 m^3
Ramp ponding (5%) of the ramp surface = 7m x 19m x 5% = 6.65 m^3
<u>Note: Ramp to drain to storage tank provide (pipe inverts show on plan SG1)</u>

<u>Basement storage tank</u>:

The underground storage tank will have following dimensions 4m x 7.5m x 1.5m. Hence, the storage volume for up to 100-year storms can be collected here. A total volume of 45 m^3.

0.3757 ha (landscape area) is 0.8656 ha is 43.40% of the proposed site. Therefore, it is proposed that 5mm of rainfall shall be managed by way of infiltration and evapotranspiration for the site via landscape areas.

Total volume provided = 61.48+6.65+0.71+45= 113.84 m^3 > required 114.65 m^3 – 43.28m^3 = 70.56 m^3

<u>Note</u>: Storage is increased from 114.65 m^3 to 127.56 m^3 due to the reduction in flow caused by orifice tube, please refer to the part below for more detail.

Therefore, required 127.56 m^3 – 43.28m^3 = 84.28 m^3 < 113.84 m^3 (provided)

<u>Quality control</u>:

The discharge flow rate from the site will be controlled by installing an orifice pipe at the upstream of the control catch basin.

Following formula is used to calculate the size of orifice pipe.

$Q = CA\,(2gH)^{1/2}$

Where

C = coefficient of discharge (sharp orifice)
A = Orifice area (m^2)
H = Head on orifice (m)
g = 9.81 m/sec^2

Based on maximum allowable release rate from the site (0.0360 m^3/sec), the Calculations for orifice plate size are shown here:

Q = 0.0360 m^3/sec
C = 0.25
H = 168.65(Top of CBMH 1) – 166.60(Invert of stormceptor EFO6 manhole) = 2.05 m
Q = CA $(2gH)^{½}$

A = 0.0227 m^2
D = 170.00 mm

Therefore, an orifice pipe of 150 mm diameter shall be used after the catch basin to control the flow to the Storm control manhole.

Based on orifice pipe diameter following flow (Q) is calculated below:

A = 0.01767 m^2
C = 0.25
H = 168.65 (Top of CBMH 1) – 166.60(Invert of stormceptor EFO6 manhole) = 2.05 m
Q = $CA (2gH)^{½}$
Q = 0.028 m^3/sec

Table 2: Increased Maximum storage

Time Min (t)	Intensity Mm/hr. (I)	Max. Discharge m^3/sec. Q(release)	Peak flow 100 yr. Event m^3/sec Q(peak)	Inflow Volume m^3 V(in)	Outflow Volume m^3 V(out)	Storage Required m^3
25	**102.41**	**0.028**	**0.1173**	**175.95**	**48.39**	**127.56**

Total storage provided 68.84 m^3 > required 127.56 m^3 – 43.28m^3 (5mm rainfall) = 84.28 m^3. For more detail, please refer to Drawing SG1.

<u>FIRE FLOW</u>:

F = 220 C(A)$^{0.5}$ = 220 x 0.8 x (2171 + (50% x 2085))$^{0.5}$ = 10,787.09 L/min

<u>Where</u>:
F = the required fire flow in liters per minute
C = Coefficient related to the type of construction
= 1.5 for wood frame construction (structure essentially all combustible)
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non-combustible construction (unprotect metal structural components, masonry or metals walls)
= 0.6 for fire-resistive construction (fully protected frames, floors, roof)
A = Area in Square meters

Further reduction for non-combustible building – 25%

F = 10,787.09 L/min x 0.75 = 8,090.32 L/min

Further reduction of 20% for automated sprinkler system = 8,090.32 x 80% = 6472.26 L/min

Increase in "F" value is considered for structures exposed within 45 meters as recommended by FUS (Fire Underwriters Survey)

Project Northwest exposure = 10% (20.1 m – 30 m)
Project Southeast exposure = 20% (3.1 m – 10 m)

Required fire flow = 6472.26 x 1.30 = 8413.94 L/min = 140.23 L/sec

<u>Reference</u>: Fire Underwriters survey-1999 Water supply for public fire protection

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WATER DEMANS CALCULATIONS

Building & site use

- Land Area = 0.8656 ha
- Building Area = 15,457.09 m^2
    - Commercial Use – 228.70 m^2
    - Long term facility Use – 15,228.39 m^2

Sewage flows

Ontario Building Code - Table 8.2.1.3.B and Table 3.1.17.1

- Commercial Use = 75 L/Day / 9.3 m^2 (OBC Table 3.1.17.1-office use (area/per person))
  = 75L/Day x (228.70 m^2/9.3 m^2)
  = 1844.35 L/Day

- Long term facility Use – 450 L/Day/per bed
    - Total Beds in Entire Facility – 138 Beds
      = 450 L/Day x 138 Beds = 61,200 L/Day

Subtotal average daily = 1844.35 L/Day + 61,200 L/Day = 63,044.35 L/Day

Reference: Region of Peel Public works Design Criteria Manual- Sanitary Sewer, Revised July 2009, Modified March 2017 Rev 0.9 (CS)

50 p.p. ha x 0.87 ha = 43.5 ~ 43 People avg. daily
302.8 L/C Day x 43 = 13,020 L/Day

Long term facility Occupancy:
(118 (1-bedroom) x1.68) + (20 (2-bedroom) x 2.54) = 224 residents
Assume office Building = 1844.35/302.8 = 6 People

Total = 224 People-peak

Therefore, Daily Sewage flow = 63,044.35 L/Day => 0.73 L/sec

Peak flow based on Harmon formula
M = 1 + (14 / (4+ $p^{0.5}$); p = the tributary equivalent population in thousands

M = 4.13; Therefore, peak flow = 4.13 x 0.73 L/sec = 3.01 L/sec

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WATER DEMANDS

Reference: Region of Peel Public works watermain design criteria (Revised June 2010), Table 1, and Table 2

Peak Use = 224 People

Therefore, Total water demand = 63,044.35 L/Day

Maximum/Day = 224 x 409 x 2.0 = 183,232 L/Day => 2.12 L/sec

Peak hr. = 224 x 409 x 3.0 => 3.18 L/sec

Connection Multi Use Demand Table

WATER CONNECTION

Connection Point[3)]	
152MM (6") DIA WTM ON COLLETT RD	
Pressure zone of connection point	
Total equivalent population to be serviced[1)]	224
Total lands to be serviced	0.8656 ha
Hydrant flow test	
Hydrant flow test location	
Test was conducted on May 19, please see attached report.	

No.	Water demands			
	Demand type	Demand (l/sec)		
		Use 1[5)]	Use 2[5)]	Total
1	Average day flow	0.73		0.73
2	Maximum day flow	2.12		2.12
3	Peak hour flow	3.18		3.18
4	Fire flow[2)]	140.23		140.23
Analysis				
5	Maximum day plus fire flow	142.35		142.35

WASTEWATER CONNECTION

				Total
Connection point [4)]		EX. 254MM SAN ON COLLETT RD		
Total equivalent population to be serviced [1)]				224
Total lands to be serviced				0.8656 ha
6	Wastewater sewer effluent (l/sec)			3.01

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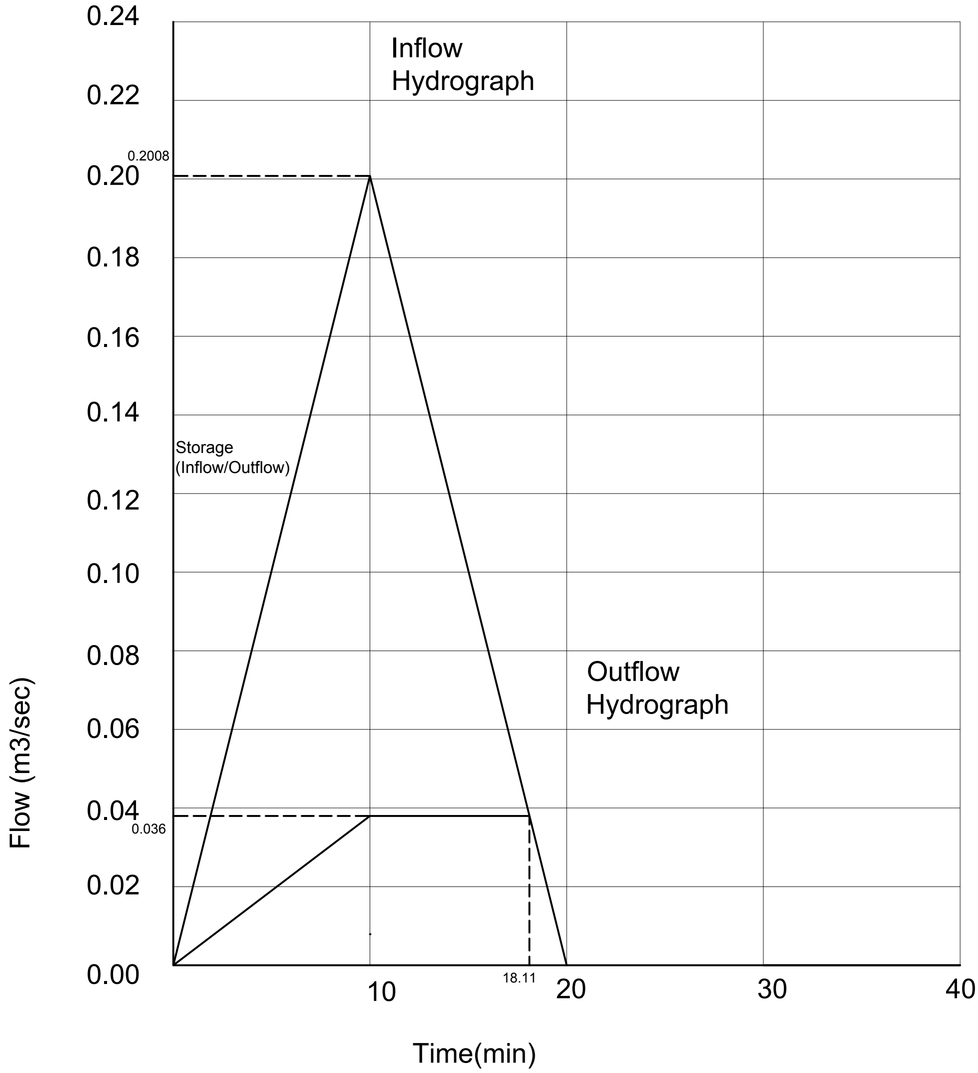
1) The calculations should be based on the development estimated population (employment and/or residential).
2) Please reference the Fire Underwriters Survey Document
3) Please specify the connection point ID
4) Please specify the connection point (wastewater line or manhole ID)
   Also, the total equivalent population to be serviced" and the "total lands to be serviced" should reference the connection point. (The FSR should contain onecopy of Site Servicing Plan)
5) Please complete as many uses are necessary for the development. (Please specify the use)

Please include the graphs associated with the hydrant flow test information table Please provide Professional Engineer's signature and stamp on the demand table. All required calculations must be submitted with the demand table submission.

Location	From M.H.	To M.H.	AREA (ha)	DENSITY (ppha)	POPULATION	CUMULATIVE AREA (ha)	CUMULATIVE POPULATION	SEWAGE FLOW 1 (m^3/sec)	INFILTRATION FLOW 2 (m^3/sec)	FOUNDATION DRAIN 3 (m^3/sec)	TOTAL FLOW (m^3/sec)	LENGTH (m)	PIPE DIAMETER (mm)	GRADIENT (%)	CAPACITY (m^3/sec)	VELOCITY (m/sec)
	BLDG	R.O.W.	0.8656	230	224	0.8656	230	0.0028	0.0002	--	0.0030	31.43	254	1.99	0.084	1.66



Storage Calculations for maximum storage at 10 minutes

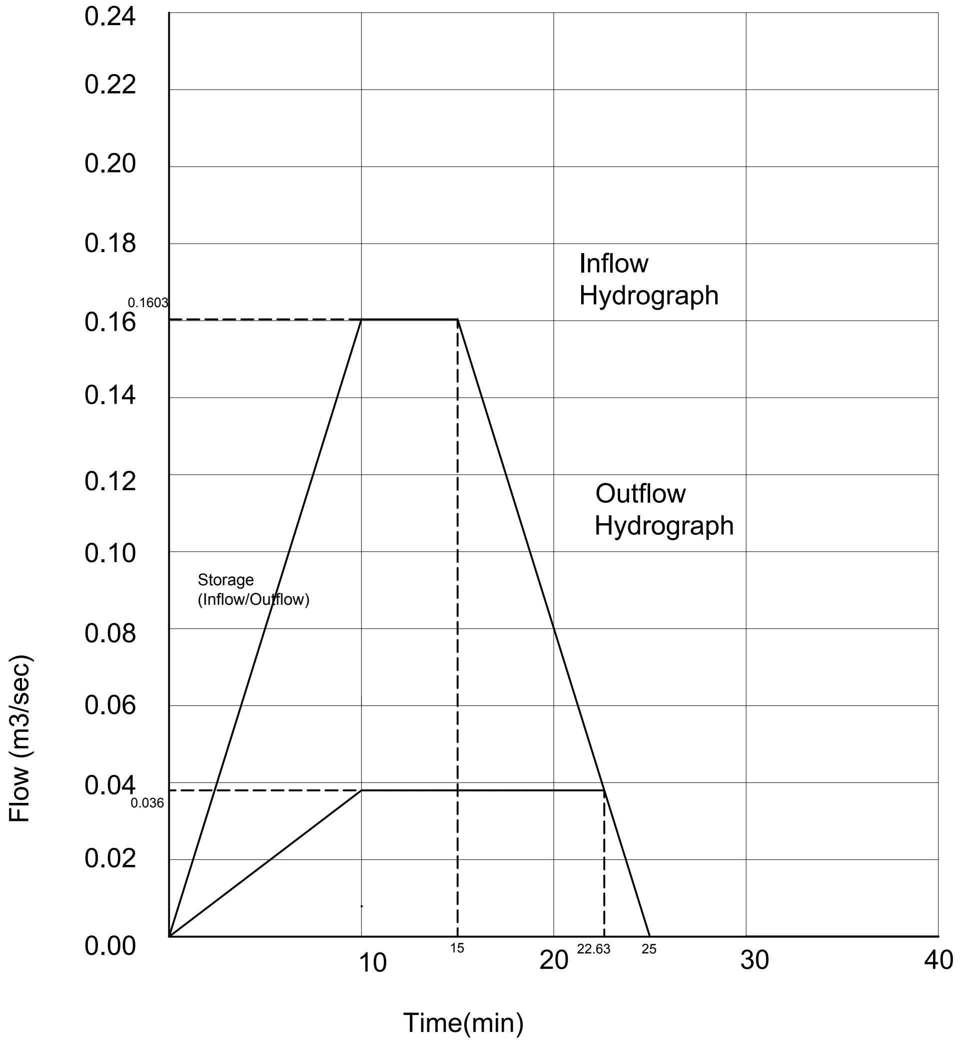
Q (peak) = 0.2008 m^3/sec.

Q (release) = 0.0360 m^3/sec

V (in) = (20/2) x 0.2008 x 60 = 120.48 m^3

V (out) = (8.11 + 20)/2 x 0.0360 x 60 = 30.36 m^3

Storage = V(in) - V(out) = 90.12 m^3



Storage Calculations for maximum storage at 15 minutes

Q (peak) = 0.1603 m^3/sec.

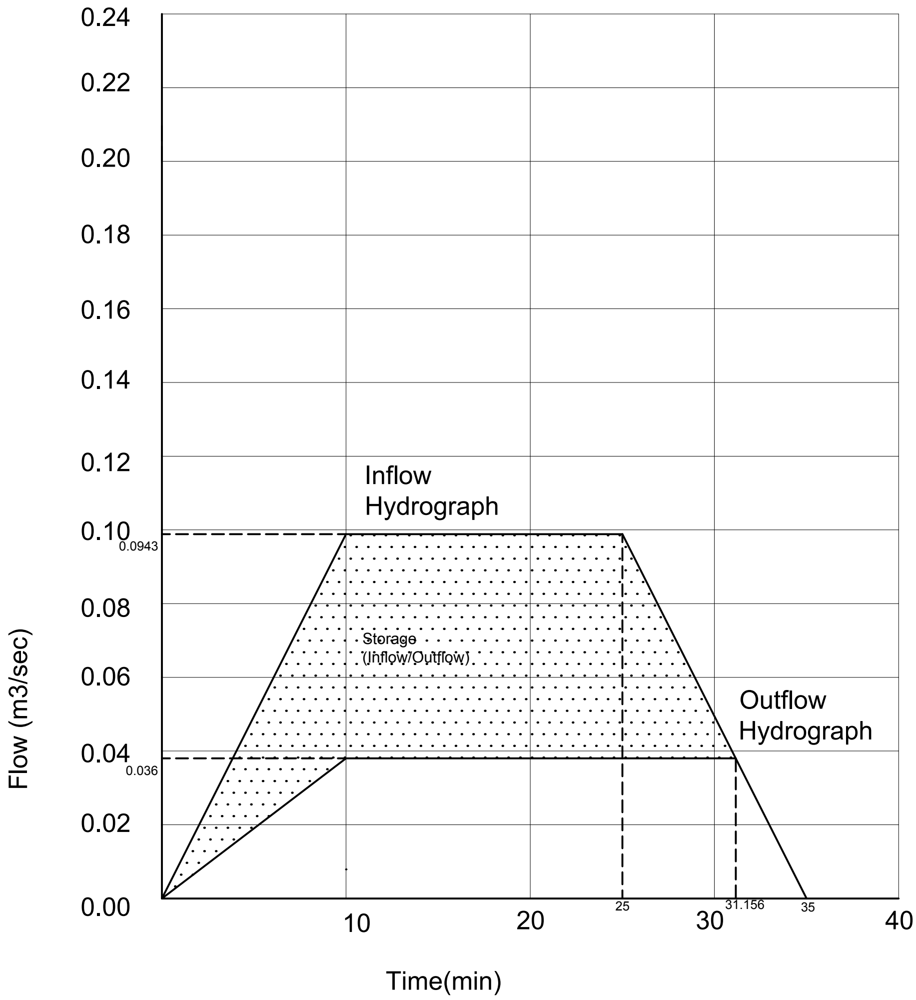
Q (release) = 0.0360 m^3/sec

V (in) = (5+25)/2 x 0.1603 x 60 = 144.27 m^3

V (out) = (12.63 + 25)/2 x 0.0360 x 60 = 40.64 m^3

Storage = V(in) - V(out) = 103.63 m^3

Maximum storage volume occurs at 25 minutes



Storage Calculations for maximum storage at 25 minutes

Q (peak) = 0.0943 m³/sec.

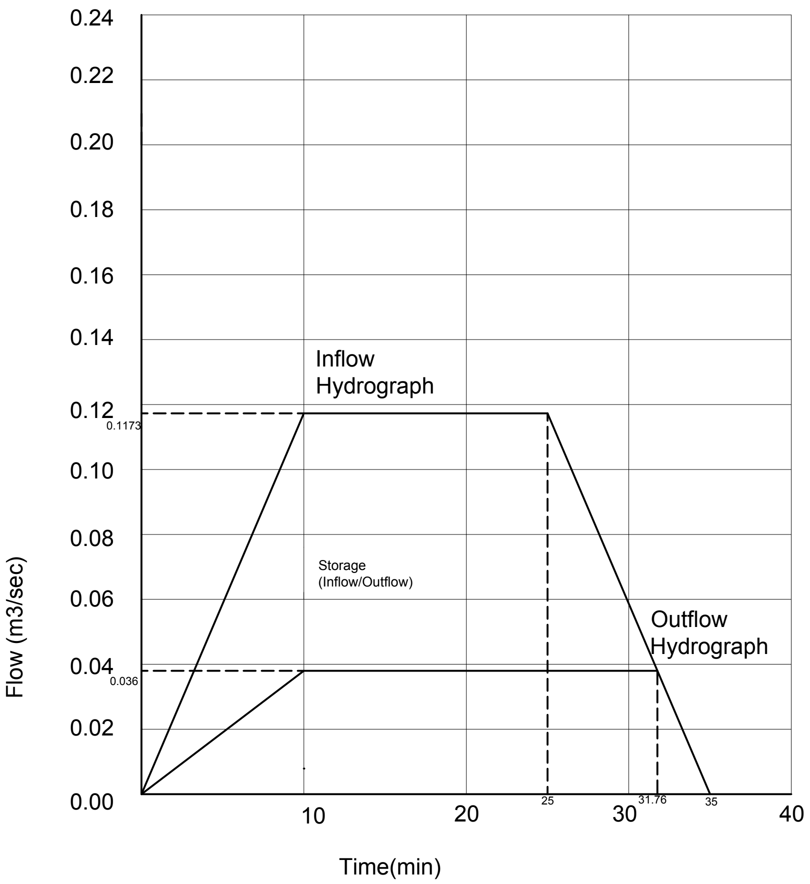
Q (release) = 0.0360 m³/sec

V (in) = (15+35)/2 x 0.0943 x 60 = 141.45 m³

V (out) = (21.156 + 35)/2 x 0.0360 x 60 = 60.65 m³

Storage = V(in) - V(out) = 80.80 m³

Maximum storage volume occurs at 25 minutes



Storage Calculations for maximum storage at 25 minutes

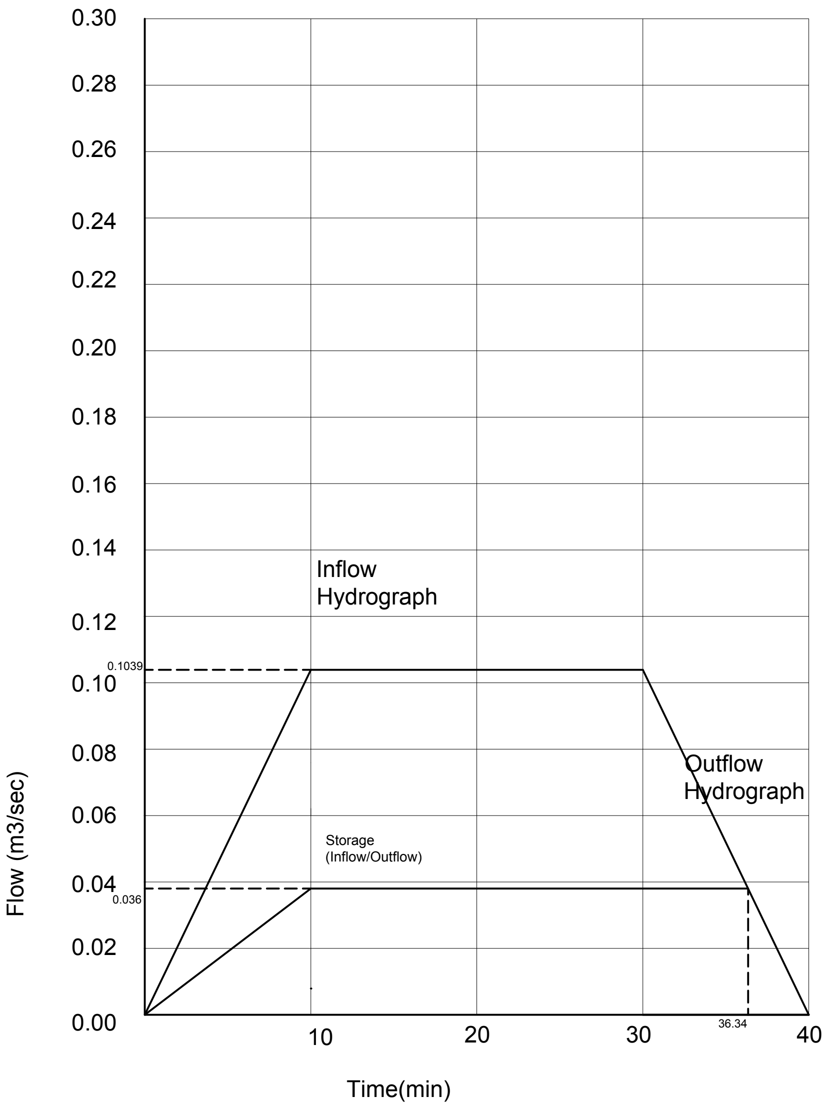
Q (peak) = 0.1173 m^3/sec.

Q (release) = 0.0360 m^3/sec

V (in) = (15+35)/2 x 0.1173 x 60 = 175.95 m^3

V (out) = (21.76 + 35)/2 x 0.0360 x 60 = 61.30 m^3

Storage = V(in) - V(out) = 114.65 m^3



Storage Calculations for maximum storage at 30 minutes

Q (peak) = 0.1039 m^3/sec.

Q (release) = 0.0360 m^3/sec

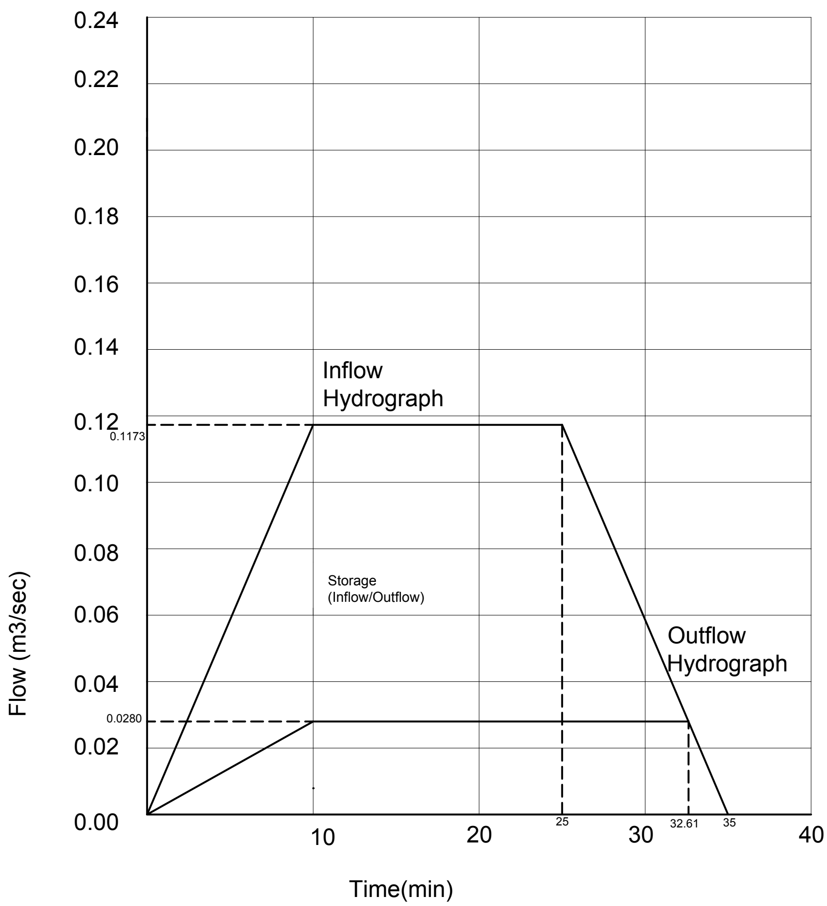
V (in) = (15+40)/2 x 0.1039 x 60 = 171.43 m^3

V (out) = (26.34 + 40)/2 x 0.0360 x 60 = 71.65 m^3

Storage = V(in) - V(out) = 99.78 m^3

ORIFICE PIPE FLOW REDUCTION

Maximum storage volume occurs at 25 minutes



Storage Calculations for maximum storage at 25 minutes

Q (peak) = 0.1173 m³/sec.

Q (release) = 0.0280 m³/sec

V (in) = (15+35)/2 x 0.1173 x 60 = 175.95 m³

V (out) = (22.61 + 35)/2 x 0.0280 x 60 = 48.39 m³

Storage = V(in) - V(out) = 127.56 m³

APPENDIX B

FIGURES:

1. Site Location Plan
2. Site Survey
3. Mississauga IDF chart
4. Stormceptor Sizing report
5. Site Plan (A – 100)
6. Proposed Grading and Servicing Plan (SG1, SG2)