BARRHAVEN FELLOWSHIP CHRISTIAN REFORMED CHURCH

3058 JOCKVALE ROAD

OTTAWA, ON

SITE SERVICING REPORT

STORMWATER MANAGEMENT

EASTERN ENGINEERING GROUP INC.

APEX BUILDING

100 STROWGER BLVD, SUITE 207

BROCKVILLE, ON

K6V 5J9

NOVEMBER 2017

REVISION RECORD					
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1	CITY OF OTTAWA COMMENTS	CJ	2018-03-20		
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May 30/18

PROVINCE OF ONTARIO

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1.0 PROJECT BACKGROUND

The project consists of the construction of a new church on the site of the current Barrhaven Fellowship Christian Reformed Church. The site is located at 3058 Jockvale Road, Ottawa. The project will include site works, grading and drainage, new stormwater management facilities and parking areas. There is planned for a future expansion as well of the building and the parking.

The current site is bounded by Jockvale Road to the north, CN rail to the west, a residential subdivision (Townsend Drive) to the south and east. The site has been serviced with sanitary, storm and water through an easement from Townsend Drive and are in place to the existing church.

2.0 SUPPORTING INFORMATION

The Stormwater Management Report was developed using background information provided by the Owner, City of Ottawa and Vandenberg & Wildeboer Architects Inc.

The following documents were referenced in preparing the site design for the Barrhaven Fellowship Christian Reformed Church:

- Sewer Design Guidelines – City of Ottawa, 2012
- Design Guidelines for Drinking Water Systems, Ministry of Environment, 2008
- Design Guidelines for Sewage Works, Ministry of the Environment, 2008
- Stormwater Management Planning and Design Manual, Ministry of the Environment, 2003

3.0 LOCATION

The property is located at 3058 Jockvale Road, Ottawa, Ontario.

4.0 SERVICING

4.1 Water Demand

The existing church was supplied with new servicing in 1999 from the south (Townsend Drive). This servicing includes a 50 mm water service, sanitary service and storm service. This is shown on the Site Servicing Plan C2 Site Servicing Plan. The existing services will be re-routed on site to the new mechanical room on the main floor of the church.

Water demands for the proposed church development were provided to the City of Ottawa in order to determine boundary conditions for the site. Hydraulic gradeline (HGL) values for each of the demand scenarios were obtained from the City. They are summarized below:

- Average Day Demand of 0.75 L/s; corresponding HGL of 155.7 m (Existing Pressure)
- Peak Hour Demand of 1.20 L/s; corresponding HGL of 144.7 m (Existing Pressure)
- Max Day Demand of 1.85 L/s plus an additional Fire Flow Demand of 183.3 L/s; corresponding HGL of 121.0 m.

The City has indicated that for the requested fire flow of 183.3 L/s, the resulting HGL is 121.0 m. This corresponds to a residual pressure of 32.7 psi, above the minimum required pressure of 20 psi for fire flow connections. According to the boundary condition provided, there is significant fire flow capacity along Townsend Drive.

With respect to the Peak Hour flow conditions, the resulting boundary condition HGL of 144.7 m corresponds to a Peak Hour pressure of 66.3 psi which is within the desired normal operation range of 40-80 psi as per the City of Ottawa 2010 Water Distribution Design Guidelines.

With respect to the Maximum pressure check, the resulting boundary condition HGL of 155.7 m corresponds to a pressure of 82.1 psi which is above the objection maximum pressure value of 80 psi as per the City of Ottawa 2010 Water Distribution Design Guidelines.

As per Ontario Building Code, pressure reducing valves to be installed immediately downstream of the isolation valve in the building located just downstream of the meter.

4.2 Sanitary Demand

Sanitary Sewer Flow

The sanitary sewage flow for the proposed church is calculated based on City of Ottawa Sewer Design Guidelines (Section 4.4.1.2).

Institutional Flow

Church Flow assumed to be 20 L/seat over 16 hour day. Based on Building Code Matrix, the maximum occupancy is 899 persons.

899 x 20 L/d/pers x 1/16 hrs x 1/60 min x 1/60 s = 0.31 L/s

Infiltration Flow

Peak Extraneous Flow = 0.28 L/s/effective gross ha

Site Area = 1.75 ha x 0.28 L/s = 0.49 L/s

The Peak sanitary Sewage flow to be discharged from the site = 0.80 L/s

Sewage discharges will be domestic in type and in compliance with the City of Ottawa Sewer Use By-Law. The proposed service connection to the existing sewer will be as per the drawings on the east side of the church, where it currently enters the building. The existing sanitary service, 200 mm dia at slope of 7.2%, has a capcity of 88.0 L/s. The maximum hydraulic load is 0.80 L/s.which is 0.9% capacity of the pipe.

The calculated hydraulic load for the sanitary service is 65 fixture units (SFU). Based on Ontario Building Code, the approximate probably sanitary drainage rate based on SFU is 45 gpm. The minimum required sanitary service is 100 mm dia at 1%.

5.0 DRAINAGE AREAS

The overall drainage area is shown on Drawing No. C2 Site Servicing Plan and C7 Drainage Area Plan. The total area of the site is 1.75 ha.

The general slope of the site is from north to south. The natural grade of the existing site is mainly from the existing church towards the southeast property line. A portion of the site to the north which will remain green space drains towards Jockvale Road and this will not be modified

6.0 EXISTING CONDITIONS

The current site does not have any stormwater management facilities in place. The site drains naturally to a swale along the south property line. There is a storm sewer and catch basins located in the servicing easement to Townsend Drive.

The intent of the stormwater management plan presented herein is to mitigate any negative impact that the proposed development will have on the existing storm sewer system and neighbouring properties.

7.0 POST DEVELOPMENT DRAINAGE

The intent is to limit runoff for the 5 year to 100 year storm events to the 5 year predevelopment rate as per City of Ottawa SWM guidelines. The flow from the site will be controlled to meet the 5 year flow using stormwater management facilities. Storage will be provided on site for the 5 and 100 year event, with the 5 year release rate.

The east portion of the site will drain overland through depressed curb areas, through a rip rap outlet, to a new flat bottom swale. The flat bottom ditch will have rip rap check dams installed to retain water behind acting as above ground storage. Below the bottom of the ditch will be a buried 200mm dia. subdrain which will convey water into a new OGS manhole. The OGS will also act as quantity and quality control for the entire site. The subdrain and the trench in which it sits will aid in infiltration back into the ground.

The west portion of the site drains overland through depressed curb areas, through rip-rap outlets into a bio retention swale area. The bioretention facilities will promote filtration and infiltration of the runoff, thereby providing some quality and quantity control. Outlet from the bioretention areas will be controlled through outlet storm sewers prior to discharging into the new storm system.

8.0 QUALITY CONTROL

The current church site does not incorporate any stormwater quality control measures. Runoff is overland to grassed swales along the south property line, and into existing catch basin structures in the easement.

The Ministry of Environment Stormwater Management Planning and Design Manual, March 2003, provides requirements for stormwater quality control. For this project a normal level of protection of 80% TSS removal will be targeted through the implementation of Low Impact Development strategies as follows:

Fit Design to Terrain – The area will be drained to use the natural slope of the ground and slope to the new ditches.

Conveyance Control – The use of low gradient grassed swales with flattened side slopes is one of the best conveyance controls available. The flat grades (min 0.5%) help to reduce flow velocities, reducing erosion potential.

At the lot, the effects of runoff reduction measures are enhanced by minimizing lot grades to promote natural infiltration. The flat bottomed retention ditches and bioretention swale will retain normal rainwater events and filter through the ground as the slope through the basin is minimal. During larger events, the water will slowly fill up in the basin/swales, with sediment releasing to the bottom of the basin and water eventually flowing through the outlet structure.

As stated in MOE Stormwater Management Planning and Design Manual, grassed swales are most effective for stormwater treatment when the depth of flow is minimized, bottom width is maximized and channel slope is minimized. The grass should be allowed to grow higher than 75mm to enhance the filtration of suspended solids.

For runoff from the parking lot in Drainage Area 1, it will drain into the easterly flat bottom ditch, which is 2.75 m wide with a 650 mm deep by 500 mm wide clear stone infiltration trench below the surface. There is a 200 mm dia. subdrain located in the clear stone trench which will convey stormwater when the basin has reached saturation. The water will flow into this area and the first flush will help to settle the sediment out of the storm flow from the storage area.

For runoff from the storage area in Drainage Area 2, it will drain overland into the bioretention area along the west and south side of the future proposed parking lot. The bioretention ditch is constructed to allowe the sediment and stormwater to settle into the basin and naturally filter itself. The basin is approximately 84 m long and also outlets into the OGS at the site outlet to the south. The water will flow into this area and the first flush will help to settle the sediment out of the storm flow from the storage area.

The outlet of the stormwater facilities will be an OGS structure connected to the existing storm pipe. The OGS will be sized to control the outgoing storm flow to meet the 5 year predevelopment rate as well as to treat the water to meet the 80% removal of suspended solids.

The basin will be vegetated with hydroseed to help the filter media grow quicker. The hydroseed will be placed with mulch that will aid in the growth of seed. The seed mix will contain native species that are suitable to the local soil, moisture and lighting conditions.

The site drainage treatment to achieve the 80% TSS removal will be accomplished with the use of an OGS unit. The OGS will be connected to the existing storm sewer south through the unopened easement and drain to City of Ottawa storm sewer system.

Bioswales are landscape elements designed to remove silt and pollution from surface runoff water. They consist of a drainage course with sloped sides and filled with vegetation, compost and/or riprap. The water's flow path, along with the wide and shallow ditch, is designed to maximize the time water spends in the swale, which aids the trapping of pollutants and silt. Biological factors also contribute to the breakdown of certain pollutants.

The basin will be vegetated with hydroseed to help the filter media grow quicker. The hydroseed will be placed with mulch that will aid in the growth of seed. The seed mix will contain native species that are suitable to the local soil, moisture and lighting conditions.

9.0 SEDIMENT AND EROSION CONTROL PLAN

To control sediment and erosion during construction the following shall be adhered to:

1. Before proceeding with any area grading the silt fence must be constructed where indicated.
2. Silt control fence shall be installed where shown and maintained until the completion of the landscaping.
3. Catchbasin silt traps are to be installed at all catchbasins.
4. Accumulated silt to be removed off site prior to removal of the silt control fence.
5. Contractor to clean adjacent roads on a regular basis to the satisfaction of the City of Ottawa.
6. The silt fence must be inspected weekly and immediately after rainfall events for rips or tears, broken stakes, blow outs (structural failure) and accumulation of sediment. The silt fence must be fixed and/or replaced immediately when damaged. Sediment must be removed from silt fence when accumulation reaches 50% of the height of the fence.
7. Upon completion of landscaping all sediment and erosion control measures shall be removed.
8. No construction activity or machinery shall be beyond the silt fence.
9. All earth or topsoil stockpiles shall be surrounded with a sediment control fence.

The Contractor shall be responsible for monitoring and maintaining the sediment and erosion control facilities until re-vegetation is complete.

The Sediment and Erosion Control Plan shall be considered a 'living document' that may need to be changed or adjusted during the life of the project to be effective.

10.0 PROPOSED STORMWATER MANAGEMENT

The stormwater management for the new church site has been divided into 3 drainage areas. These drainage areas are shown on Drawing C6.

Drainage Area 1 is the south eastern drainage area consisting of the new entrance from Jockvale Road, a portion of the parking lot, a portion of the building and some of the grassed surface on the east of the church.
Drainage Area 2 is the south wester drainage area consisting of large parking lot, a portion of the building and the grassed area to the north and west of the parking lot.
Drainage Area 3 is the north eastern portion of the site. It consists of a large grassed area, a small walkway and a portion of the building.

The majority of the drainage will flow southerly to the ditch in DA 1 and DA 2. The flow from DA 3 is will flow overland through grassed swales to the bioretention area at the south of the property.

11.0 QUANTITY – PRE DEVELOPMENT

The water quantity objective for the storage area is to not exceed the existing stormwater flows from this area. The flow is limited to the pre-development runoff rates.

11.1 Drainage Area 1

Drainage Area 1 is the south eastern drainage area consisting of the new entrance from Jockvale Road, a portion of the parking lot, a portion of the building and some of the grassed surface on the east of the church. The area of DA1 is 0.397 ha. The pre-development average runoff coefficient for the 5 year storm is 0.34, and the 100 year storm is 1.25 x c_5 = 0.425. The proposed area is the same, with 1588 m2 of hard surface and 2384 m2 of soft surface. Cavg for post development is 0.54 for 5 year event and 0.675 for 100 year event.

We use the Modified Rationale Method to calculate the pre and post development runoff.

The time of concentration is estimated at 15 mins based on accepted municipal design standards for determination of allowable flow. The rainfall intensity is taken using the City of Ottawa values. I_{5year} is calculated to be 83.56 and $i_{100year}$ is 142.89.

The target runoff is calculated using the modified rationale method and the 5 year pre:

Q= 2.78x A x I x C = 2.78*0.397*.34*83.56 = <u>31.36 L/s</u>

For the 100 year event:

Q= 2.78x A x I x C = 2.78*0.397*.425*142.89 = <u>67.0 L/s</u>

The 5 year pre development flow was calculated to be 31.4 L/s and the 100 year pre development flow is 67.0 L/s.

<u>11.2 Drainage Area 2</u>

Drainage Area 2 is the south western drainage area consisting of the new and future parking lots, grassed area along the CN property line and grassed/treed area at the western side. The area of DA2 is 0.867 ha. The pre-development average runoff coefficient for the 5 year storm is 0.34, and the 100 year storm is 1.25 x c_5 = 0.425. The proposed area is the same, with 4500 m2 of hard surface and 4197 m2 of soft surface. Cavg for post development is 0.61 for 5 year event and 0.762 for 100 year event.

We use the Modified Rationale Method to calculate the pre and post development runoff.

The time of concentration is estimated at 15 mins based on accepted municipal design standards for determination of allowable flow. The rainfall intensity is taken using the City of Ottawa values. I_{5year} is calculated to be 83.56 and $i_{100year}$ is 142.89.

The target runoff is calculated using the modified rationale method and the 5 year pre:

Q= 2.78x A x I x C = 2.78*0.867*.34*83.56 = <u>68.48 L/s</u>

For the 100 year event:

Q= 2.78x A x I x C = 2.78*0.867*.425*142.89 = 146.37 L/s

The 5 year pre development flow was calculated to be 68.5 L/s and the 100 year pre development flow is 146.37 L/s.

11.3 Drainage Area 3

Drainage Area 3 consists of the north eastern corner of the site. The existing conditions of DA 3 consist of the existing entrance way (compacted hard gravel), the church as well as large grassed area. The proposed areas for DA 3 include less hard surface than the existing. There will be more grassed surface due to the removal of the existing entrance.

This area currently drains uncontrolled towards Jockvale Road overland. The area will be re-graded to contain the runoff in the drainage area, and direct it towards a new swale along the northern property line, ultimately into the storm retention basin. There will be a slight reduction in runoff due to the reduced runoff coefficient.

The area of DA3 is 4700 m^2. Existing conditions are comprised of 1880 m^2 of hard surface (c=0.9) and 2820^2 of soft surface (c=0.3). The average runoff coefficient for the 5 year storm is 0.54. The proposed area is the same, with 1546 m2 of hard surface and 3154 m2 of grass surface. Cavg for post development is 0.50.

The time of concentration is estimated at 15 mins based on accepted municipal design standards for determination of allowable flow. The rainfall intensity is taken using the City of Ottawa values. I_{5year} is calculated to be 83.56 and $i_{100year}$ is 142.89.

The target runoff is calculated using the modified rationale method and the 5 year pre:

Q= 2.78x A x I x C = 2.78*0.470*.54*83.56 = 58.96 L/s

For the 100 year event:

Q= 2.78x A x I x C = 2.78*0.470*.675*142.89 = 126.02 L/s

The 5 year pre development flow was calculated to be 58.96 L/s and the 100 year pre development flow is 126.02 L/s.

12.0 QUANTITY - POST DEVELOPMENT

The post development flows are calculated using Modified Rationale method for various times and rainfall intensities, to determine how much storage is required for each drainage area.

12.1 Drainage Area 1

The post development runoff coefficient is 0.54 for 5 year event and 0.675 for 100 year event.

Intensity Duration Frequency calculated using City of Ottawa values.

i_5=a / (t+b)^c a=998.071 b=6.053 c=0.814

i_{100}=a / (t+b)^c a=1735.688 b=6.014 c = 0.820

The allowable release rate from Drainage Area 1 is 31.36 L/s (5 year), and 67.0 L/s (100 year).

5 Year Storage – A=0.397 ha, c=0.54, Q allowable 31.36 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	141.8	84.51	31.36	53.15	15.94
10	104.19	62.09	31.36	30.73	**18.44**
15	83.56	49.80	31.36	18.44	16.60

100 Year Storage – A=0.397 ha, c=0.675, Q allowable 67.0 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	242.7	180.80	67	113.80	34.14

10	178.56	133.02	67	66.02	**39.61**
15	142.89	106.45	67	39.45	35.50

Therefore, based on Modified Rationale Method, the storage requirement for Area 1 for 5 year is 18.44 m3 and for 100 year 39.61 m3.

12.2 Drainage Area 2

The post development runoff coefficient is 0.61 for 5 year event and 0.762 for 100 year event.

Intensity Duration Frequency calculated using City of Ottawa values.

i_5=a / (t+b)^c a=998.071 b=6.053 c=0.814

i_{100}=a / (t+b)^c a=1735.688 b=6.014 c = 0.820

The allowable release rate from Drainage Area 2 is 68.48 L/s (5 year), and 146.37 L/s (100 year).

5 Year Storage – A=0.867 ha, c=0.61, Q allowable 68.48 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	141.8	208.48	68.48	140.00	42.00
10	104.19	153.19	68.48	84.71	**50.82**
15	83.56	122.85	68.48	54.37	48.94

100 Year Storage – A=0.867 ha, c=0.762, Q allowable 146.37 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	242.7	445.75	146.37	2199.38	89.81
10	178.56	327.95	146.37	181.58	**108.95**
15	142.89	262.46	146.37	116.06	104.46

Therefore, based on Modified Rationale Method, the storage requirement for Area 2 for 5 year is 50.82 m3 and for 100 year 108.95 m3.

12.3 Drainage Area 3

The post development runoff coefficient is 0.50 for 5 year event and 0.625 for 100 year event.

Intensity Duration Frequency calculated using City of Ottawa values.

i_5=a / (t+b)^c a=998.071 b=6.053 c=0.814

i_{100}=a / (t+b)^c a=1735.688 b=6.014 c = 0.820

The allowable release rate from DA 3 is 58.96 L/s (5 year), and 126.02 L/s (100 year).

5 Year Storage – A=0.470 ha, c=0.50, Q allowable 65.3 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	141.8	92.64	58.96	33.68	**10.10**
10	104.19	68.07	58.96	9.11	5.46
15	83.56	54.59	58.96	0	0

100 Year Storage – A=0.47 ha, c=0.625, Q allowable 126.02 L/s

Tc (min.)	I (mm/hr.)	Q (L/s)	Qallow (L/s)	Net Runoff (L/s)	Storage (m^3)
5	242.7	198.19	126.02	72.17	**21.65**
10	178.56	145.82	126.02	19.80	11.88
15	142.89	116.69	126.02	0	0

Therefore, based on Modified Rationale Method, the storage requirement for Area 3 for 5 year is 10.10 m3 and for 100 year 21.65 m3.

13.0 STORAGE PROVIDED

13.1 Area 1 Stormwater Storage

The runoff from the parking lot in Drainage Area 1 flows overland into the easterly flat bottom ditch through depressed curb sections. The ditch is 2.75 m wide with a 650 mm deep by 500 mm wide clear stone infiltration trench below the surface. There is a 200 mm dia. subdrain located in the clear stone trench which will convey stormwater when the basin has reached saturation. The water will flow into this area and the first flush will help to settle the sediment out of the storm flow from the storage area.

The storage available in this ditch is 2 parts. The first part will be the voids in the clear stone infiltration trench. The available storage here is 0.65 x 0.50 m = 0.325 m2/m x 0.40 voids for 0.13 m3 per m of trench. The trench is 77 m long on the east side of structure 1. The underground storage here is equivalent to 10.0 m3.

The second part of the storage is above ground and is accounted for in a 2.75 m wide ditch with rock check dams designed to hold water behind during large rain events. The eastern most ditch has an average cross-sectional area of 0.775 m2 and is 25 m long for a volume of 19.4 m3. The second part of this ditch is 39 m long and has a cross-sectional area of 0.66 m2 for a volume of 25.7 m3.

The total storage available on the eastern portion of the site for Drainage Area 1 is 55.1 m3 and more than the required 39.61 m3 for the 100 year storm event.

13.2 Area 2 Stormwater Storage

The runoff from the parking lot and building in Drainage Area 2 flows overland southwesterly into the stormwater retention area which surrounds the future expansion of the parking lot and along the south property line to connect in with Structure 1.

This stormwater storage area is bioretention swale. The bio-retention swale will function as follows:

- Runoff from the parking lot will reach the swale via the rip-rapped spillways from the curb cuts.
- As the bio-retention swales are low sloped with a 300 mm high dam at the end of each section the runoff will pond in the swale and be infiltrated by the sandy planting medium and the underlying clear stone layer.

- If the bioretention area has filled to a depth of 200 mm above the swale, the runoff will enter the catch basins and be directed down into the storm sewer by the linear sewer. The stormwater runoff will infiltrate into the adjacent soil or be conveyed via the storm outlet ditches, swales or sewers.
- The storage capacity of the bioretention swales is such that the post development outflow will be less than or equal to the pre-development condition. The storage requirements of the bio-swales will meet the 100 year storage requirements for the areas at both the 5 and 100 year release rates.

Storage and quality control for Drainage Area 2 is provided in the bio-retention swale constructed on the south side of the church property. The bioswale is 1.6 m deep and 2.25 m wide. The length of the bioswale is approximately 85m. There are voids in the clear stone layer of stone which is 2.25 m x 1.0 m. The voids are approximately 40% which equals 0.9 m3/m. This equates to approximately 76 m^3 in storage in the stone layer alone. The swale on the top of the bioswale is also designed to contain stormwater. There are 3 sections of above ground storage. The westerly one is smaller and provides 5.8 m3 of storage. The middle section provides 21.0 m3 of storage and the easterly section allows for 17.5 m3 of storage. The total storage in the stone layer and above ground is 120.3 m3. The filter media layer of the bioswale will also contain water and we estimate 10% voids in this layer which is equal to 2.5m x 0.5 x 0.1 x 85 m) = 10.6 m3 for a total of <u>130.9 m3</u>. This is the amount required for the 100 year storm event.

The water filters into the ground naturally and in the case of oversaturation, there is a 300mm dia. storm sewer connected to the catch basin (Structure 2) at the low end of the basin and to the Structure 1. Build up of stormwater will release via the catch basin into the storm system and through the control manhole SC.

The Owner will be responsible for maintaining the works.

13.3 Area 3 Stormwater Storage

The required stormwater storage for area 3 is 21.65 m3. The Drainage Area 3 has been graded to have a small berm along the north and north west which contains the surface runoff. There is a new swale shown which will direct the runoff through long, low sloped ditches to encourage infiltration. The storage area is 616 m2 and a depth of 0.35 m before overflow the path. The storage area has 44m3 of surface storage before it gets to the new 250mm dia storm pipe crossing under the new asphalt path. This pipe will control the flow to the allowable rate of 59 L/s which matches the 5 year release rate. A small portion of the Drainage area will continue to drain to the northern corner of the lot, near the existing walking path. The runoff from Jockvale Road, also from the right of way, will continue to drain overland northerly to the walking path. At this location, the runoff drains directly into the existing drainage ditch along the southeast side of the railway. This is a drainage ditch which follows the CN tracks in both directions.

14.0 OUTLET CONTROL STRUCTURE

The outlet control structure consists of the following:

Drainage Area 1, 2 and 3 will be controlled by an OGS structure (SC) installed at the south property limit of the site, at the existing utility easement. There currently is a manhole connected to the existing storm service. The new OGS structure will be installed at property line and connect into the existing structure with a 300 mm dia PVC storm pipe.

There will be a Stormceptor Model STC2000 manhole installed as shown on the drawings. The STC2000 is designed to remove 80% TSS and will also be sized appropriately for the storm water flowing through the pipe from the Barrhaven Reformist Church site. The flow through the stormceptor will be controlled during 5 year event to 158.84 L/s. There is storage in the site for up to the 100 year event. The Stormceptor Detailed Sizing Report by Forterra and Imbrium Systems is attached in the appendix.

15.0 CONSTRUCTION ACTIVITIES

The construction of the project will entail:

- Construction of erosion and sediment control protection for the site.
- Installation of new sewers (storm) and structures.
- Construction of bio-retention swales and protection of adjacent property.
- Construction of parking lot, including grading of granular sub-base, granular base, asphalt, curbing, sidewalks.
- Hydroseeding and landscaping of the site.

16.0 WINTER OPERATION

During the winter months, snow will be stored in areas designated as snow storage area. If accumulation is above normal, snow will removed from the site and parking lot. There will be no snow storage permitted in any of the bio-retention areas. All catch basins and manholes will be monitored and kept free of snow and ice buildup to avoid any localized flooding on the site. The insulation effect of the snow will limit the depth of frost penetration such that when temperatures rise, moderate flow will occur in the bottom of the ditches and swales. Regular maintenance of the parking lot in spring to remove accumulated sand will be required.

17.0 MAINTENANCE

The owner will have maintenance staff review the site periodically during routine maintenance. Staff will remove any leaves, debris, etc from swales and retention basin. The rip rap area is to be inspected visually yearly to determine if there is a high buildup of sediment in the rip rap.

Manholes and catchbasins in the parking lot will need to be cleaned out as required in the sumps.

The maintenance plans and forms must address the following:

• inspection frequency

• maintenance frequency
• data collection/ storage requirements (i.e. during inspections)
• detailed cleanout procedures (main element of the plans) including:
   − equipment needs
   − maintenance techniques
   − occupational health and safety
   − public safety
   − environmental management considerations
   − disposal requirements (of material removed)
   − access issues

18.0 BIORETENTION MAINTENANCE

From: Low Impact Development Stormwater Management Planning and Design Guide, 2010 by CVC and TRCA.

Ideally, bioretention sites should remain outside the limit of disturbance until construction of the Bioretention begins to prevent soil compaction by heavy equipment. Bioretention locations should not be used as the site of sediment basins during construction, as the concentration of fines will prevent post-construction infiltration. They should also not be used for storing materials. To prevent sediment from clogging the surface of a bioretention cell, stormwater should be diverted away from the bioretention site until the drainage area is fully stabilized. Due to the locations of many bioretention practices in the road right-of-way or tight urban spaces, considerations of traffic control and utility conflicts must be part of the plans and inspections.

The following is a typical construction sequence to properly install a bioretention practice. The steps may be modified to reflect different bioretention applications or expected site conditions.

1. Bioretention areas should be fully protected by silt fence or construction fencing to prevent compaction by construction traffic and equipment.
2. Installation may only begin after entire contributing drainage area has been either stabilized or flows have been safely routed around the area. The designer should check the boundaries of the contributing drainage area to ensure it conforms to original design.

3. The pretreatment forebay should be excavated first and sealed until full construction is completed.
4. Excavators or backhoes working adjacent to the proposed bioretention area should excavate the cell to the appropriate design depth.
5. It may be necessary to rip the bottom soils to promote greater infiltration or excavate any sediment that may have built up during construction.
6. There are three options at this step depending on the design:

  a. No infiltration: Place an impermeable liner on the bed of the bioretention area with 150 mm overlap on sides. Lay the perforated underdrain pipe, Pack 50 mm diameter clear stone to 75 mm above top of underdrain, an optional 75 mm choking coarse of pea gravel, and then lay the non-woven geotextile drainage fabric over the stone and underdrain.

  b. Partial infiltration: Place desired depth of stone for the infiltration volume on bed and then lay the perforated underdrain pipe over it. Pack 50 mm diameter clear stone to 75 mm above the top of the underdrain, an optional 75 mm choking coarse of pea gravel and then lay the non-woven geotextile drainage fabric over the stone and underdrain.

  c. Full infiltration: Stone can be placed to provide added stormwater volume storage or the bioretention media can be added directly to the bottom of the excavation.

7. Bioretention filter media should be obtained premixed from a vendor. Apply in 300 mm lifts until desired top elevation of bioretention area is achieved. Thoroughly wet each lift before adding the next and wait until water has drained through the soil before adding the next lift. Wait a few days to check for settlement, and add additional media as needed.
8. Prepare planting holes for any trees and shrubs, install vegetation, and water accordingly. Install any temporary irrigation.
9. Plant landscaping materials as shown in the landscaping plan, and water them weekly in the first two months.
10. Lay down surface cover in accordance with the design (mulch, riverstone, or turf).
11. Conduct final construction inspection, checking inlet, pretreatment cell, bioretention cell and outlet elevations.

19.0 CONSTRUCTION INSPECTION

Common construction pitfalls can be avoided by careful construction supervision that focuses on the following aspects:

Erosion and Sediment Control

- Bioretention locations should be blocked from construction traffic and should not be used for erosion and sediment control.
- Proper erosion and sediment controls should be in place for the drainage area.

Materials

- Gravel for the underdrain should be clean and washed; no fines should be present in the material.
- Underdrain pipe material should be perforated and of the correct size.
- A cap should be placed on the upstream (but not the downstream) end of the underdrain.
- Filter media should be tested to confirm that it meets specifications.
- Mulch composition should be correct.

Elevations of the following items should be checked for accuracy:

- Depth of the gravel and invert of the underdrain
- Inverts for inflow and outflow points
- Filter depth after media is placed
- Ponding depth provided between the surface of the filter bed and the overflow structure
- Mulch depth

Landscaping and Stabilization

- Correct vegetation should be planted.
- Pretreatment area should be stabilized.
- Drainage area should be stabilized prior to directing water to the bioretention.

The following items should be checked after the first rainfall event, and adjustments should be made as necessary:

• Outfall protection/energy dissipation at concentrated inflow should be stable.
• Flow should not concentrate and should spread evenly over the filter bed.
• Ponded water at the surface of the bioretenton facility should drain within 24 hours of the end of the storm event. The filter media bed should fully drain within a maximum period of 72 hours.
• Excessive sediment accumulation should not be present.

Inspection and Maintenance

Bioretention requires routine inspection and maintenance of the landscaping as well as periodic inspection for less frequent maintenance needs or remedial maintenance. Generally, routine maintenance will be the same as for any other landscaped area, weeding, pruning, and litter removal. Routine operation and maintenance tasks are key to public acceptance of highly visible bioretention units.

Periodic inspections after major storm events will determine whether corrective action is necessary to address gradual deterioration or abnormal conditions. For the first two years following construction the facility should be inspected at least quarterly and after every major storm event (> 25 mm). Subsequently, inspections should be conducted in the spring and fall of each year and after major storm events.

While maintenance can be performed by landscaping contractors who are already providing similar landscape maintenance services on the property, they will need some additional training on bioretention needs. This training should focus on elevation differences needed for ponding, mulching requirements, acceptability of ponding after a rainstorm, and fertilizer requirements. The planting plan should be kept for maintenance records and used to help maintenance staff identify which plants are weeds or invasive.

Aside from homeowner initiated rain garden projects, legally binding maintenance agreements are a necessity for bioretention facilities on private property. Agreements should specify the property owner's responsibilities and the municipality's right to enter the property for inspection or corrective action. Agreements must require regular inspection and maintenance and should refer to an inspection checklist. The construction contract should include a care and replacement warranty to ensure vegetation is properly established and survives during the first growing season following construction.

The expected lifespan of infiltration practices is not well understood, however, it can be expected that it will vary depending on pretreatment practice maintenance frequency, and the sediment texture and load coming from the catchment.

Routine Maintenance and Operation

Routine inspection and maintenance activities as shown in Table 4.5.6 are necessary for the continued operation of bioretention areas.

Table 4.5.6 Suggested routine inspection and maintenance activities for bioretention

Activity	Schedule
Inspect for vegetation density (at least 80% coverage), damage by foot or vehicular traffic, channelization, accumulation of debris, trash and sediment, and structural damage to pretreatment devices.	After every major storm event (>25 mm), quarterly for the first two years, and twice annually thereafter.
Regular watering may be required during the first two years until vegetation is established;	As needed for first two years of operation.
Remove trash and debris from pretreatment devices, the bioretention area surface and inlet and outlets.	At least twice annually. More frequently if desired for aesthetic reasons.
• Remove accumulated sediment from pretreatment devices, inlets and outlets; • Trim trees and shrubs; • Replace dead vegetation, remove invasive growth; • Repair eroded or sparsely vegetated areas; • Remove accumulated sediment on the bioretention area surface when dry and exceeds 25 mm depth (PDEP, 2006); • If gullies are observed along the surface, regrading and revegetating may be required.	Annually or as needed

Annual Inspection and Maintenance

The annual spring cleaning should consist of an inspection and corrective maintenance tasks described in Table 4.5.7

Table 4.5.7 Suggested inspection items and corrective actions for bioretention

Inspection Item	Corrective Actions
Vegetation health, diversity and density	• Remove dead and diseased plants. • Add reinforcement planting to maintain desired vegetation density. • Prune woody matter. • Check soil pH for specific vegetation. • Add mulch to maintain 75 mm layer.
Sediment build up and clogging at inlets	• Remove sand that may accumulate at the inlets or on the filter bed surface following snow melt. • Examine drainage area for bare soil and stabilize. Apply erosion control such as silt fence until the area is stabilized. • Check that pretreatment is properly functioning. For example, inspect grass filter strips for erosion or gullies. Reseed as necessary.
Ponding for more than 48 hours	• Check underdrain for clogging and flush out. • Apply core aeration or deep tilling • Mix amendments into the soil • Remove the top 75 mm of bioretention soil • Replace bioretention soil

The owner will have maintenance staff review the site periodically during routine maintenance.

Prepared by:

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May 30/18
PROVINCE OF ONTARIO

Colin A. Jardine, P. Eng

May 30, 2018

APPENDIX 1

STORMCEPTOR DETAILED SIZING REPORT BY FORTERRA AND IMBRIUM SYSTEMS

Detailed Stormceptor Sizing Report – Barrhaven Reformist Church - 3058 Jockvale Rd.

Project Information & Location			
Project Name	Barrhaven Reformist Church - 3058 Jockvale Rd.	**Project Number**	-
City	Nepean	**State/ Province**	Ontario
Country	Canada	**Date**	11/15/2017
Designer Information		**EOR Information (optional)**	
Name	Brandon O'Leary	**Name**	Colin Jardine
Company	Forterra	**Company**	Eastern Engineering Group Inc.
Phone #	905-630-0359	**Phone #**	
Email	brandon.oleary@forterrabp.com	**Email**	cjardine@easteng.com

Stormwater Treatment Recommendation

The recommended Stormceptor Model(s) which achieve or exceed the user defined water quality objective for each site within the project are listed in the below Sizing Summary table.

Site Name	Barrhaven Reformist Church - 3058 Jockvale Rd.
Recommended Stormceptor Model	STC 2000
Target TSS Removal (%)	80.0
TSS Removal (%) Provided	80
PSD	Fine Distribution
Rainfall Station	OTTAWA MACDONALD-CARTIER INT'L A

The recommended Stormceptor model achieves the water quality objectives based on the selected inputs, historical rainfall records and selected particle size distribution.

Stormceptor Sizing Summary		
Stormceptor Model	**% TSS Removal Provided**	**% Runoff Volume Captured Provided**
STC 300	65	75
STC 750	75	88
STC 1000	76	88
STC 1500	77	88
STC 2000	80	94
STC 3000	81	94
STC 4000	85	97
STC 5000	85	97
STC 6000	87	99
STC 9000	90	100
STC 10000	90	100
STC 14000	92	100
StormceptorMAX	Custom	Custom

Stormceptor

The Stormceptor oil and sediment separator is sized to treat stormwater runoff by removing pollutants through gravity separation and flotation. Stormceptor's patented design generates positive TSS removal for each rainfall event, including large storms. Significant levels of pollutants such as heavy metals, free oils and nutrients are prevented from entering natural water resources and the re-suspension of previously captured sediment (scour) does not occur.
Stormceptor provides a high level of TSS removal for small frequent storm events that represent the majority of annual rainfall volume and pollutant load. Positive treatment continues for large infrequent events, however, such events have little impact on the average annual TSS removal as they represent a small percentage of the total runoff volume and pollutant load.

Design Methodology

Stormceptor is sized using PCSWMM for Stormceptor, a continuous simulation model based on US EPA SWMM. The program calculates hydrology using local historical rainfall data and specified site parameters. With US EPA SWMM's precision, every Stormceptor unit is designed to achieve a defined water quality objective. The TSS removal data presented follows US EPA guidelines to reduce the average annual TSS load. The Stormceptor's unit process for TSS removal is settling. The settling model calculates TSS removal by analyzing:

• Site parameters
• Continuous historical rainfall data, including duration, distribution, peaks & inter-event dry periods
• Particle size distribution, and associated settling velocities (Stokes Law, corrected for drag)
• TSS load
• Detention time of the system

Hydrology Analysis

PCSWMM for Stormceptor calculates annual hydrology with the US EPA SWMM and local continuous historical rainfall data. Performance calculations of Stormceptor are based on the average annual removal of TSS for the selected site parameters. The Stormceptor is engineered to capture sediment particles by treating the required average annual runoff volume, ensuring positive removal efficiency is maintained during each rainfall event, and preventing negative removal efficiency (scour).
Smaller recurring storms account for the majority of rainfall events and average annual runoff volume, as observed in the historical rainfall data analyses presented in this section.

Rainfall Station			
State/Province	Ontario	**Total Number of Rainfall Events**	4819
Rainfall Station Name	OTTAWA MACDONALD-CARTIER INT'L A	**Total Rainfall (mm)**	20978.1
Station ID #	6000	**Average Annual Rainfall (mm)**	567.0
Coordinates	45°19'N, 75°40'W	**Total Evaporation (mm)**	1760.2
Elevation (ft)	370	**Total Infiltration (mm)**	10248.9
Years of Rainfall Data	37	**Total Rainfall that is Runoff (mm)**	8969.0

Notes

• Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor, which uses the EPA Rainfall and Runoff modules.
• Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal defined by the selected PSD, and based on stable site conditions only, after construction is completed.
• For submerged applications or sites specific to spill control, please contact your local Stormceptor representative for further design assistance.

Drainage Area	
Total Area (ha)	1.75
Imperviousness %	51

Up Stream Storage	
Storage (ha-m)	Discharge (cms)
0.000	0.000

Water Quality Objective	
TSS Removal (%)	80.0
Runoff Volume Capture (%)	90.00
Oil Spill Capture Volume (L)	
Peak Conveyed Flow Rate (L/s)	99.84
Water Quality Flow Rate (L/s)	

Up Stream Flow Diversion	
Max. Flow to Stormceptor (cms)	0.00000

Design Details	
Stormceptor Inlet Invert Elev (m)	95.33
Stormceptor Outlet Invert Elev (m)	95.30
Stormceptor Rim Elev (m)	95.66
Normal Water Level Elevation (m)	
Pipe Diameter (mm)	300
Pipe Material	PVC - plastic
Multiple Inlets (Y/N)	No
Grate Inlet (Y/N)	No

Particle Size Distribution (PSD)

Removing the smallest fraction of particulates from runoff ensures the majority of pollutants, such as metals, hydrocarbons and nutrients are captured. The table below identifies the Particle Size Distribution (PSD) that was selected to define TSS removal for the Stormceptor design.

Fine Distribution

Particle Diameter (microns)	Distribution %	Specific Gravity
20.0	20.0	1.30
60.0	20.0	1.80
150.0	20.0	2.20
400.0	20.0	2.65
2000.0	20.0	2.65

Site Name	Barrhaven Reformist Church - 3058 Jockvale Rd.

Site Details

Drainage Area	
Total Area (ha)	1.75
Imperviousness %	51

Surface Characteristics	
Width (m)	265.00
Slope %	2
Impervious Depression Storage (mm)	1.57
Pervious Depression Storage (mm)	4.67
Impervious Manning's n	0.015
Pervious Manning's n	0.25

Maintenance Frequency	
Maintenance Frequency (months) >	12

Infiltration Parameters	
Horton's equation is used to estimate infiltration	
Max. Infiltration Rate (mm/hr)	76.2
Min. Infiltration Rate (mm/hr)	13.2
Decay Rate (1/sec)	0.00115
Regeneration Rate (1/sec)	0.01

Evaporation	
Daily Evaporation Rate (mm/day)	2.54

Dry Weather Flow	
Dry Weather Flow (lps)	0

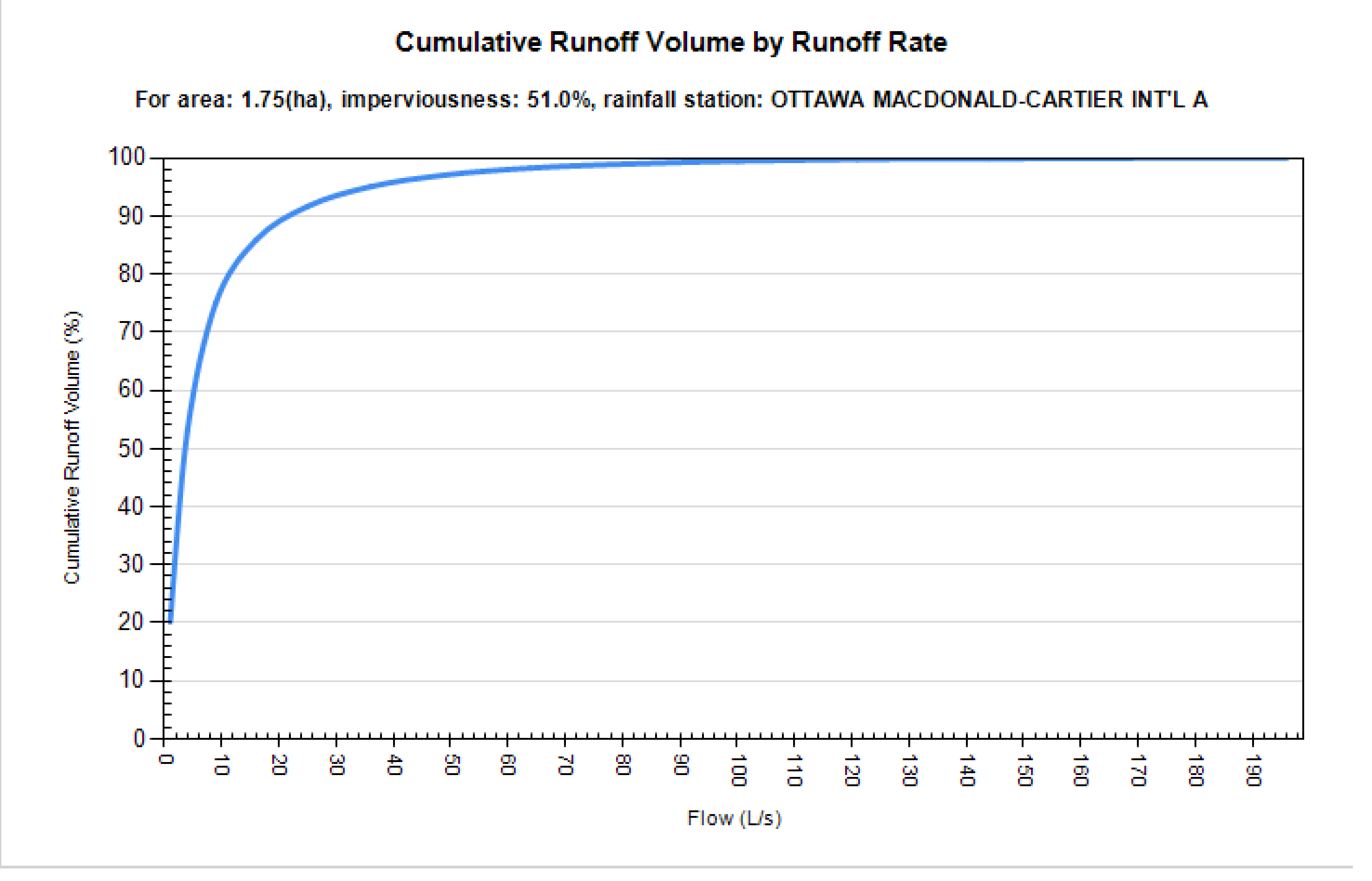
Winter Months	
Winter Infiltration	0

TSS Loading Parameters	
TSS Loading Function	Build Up/ Wash-off

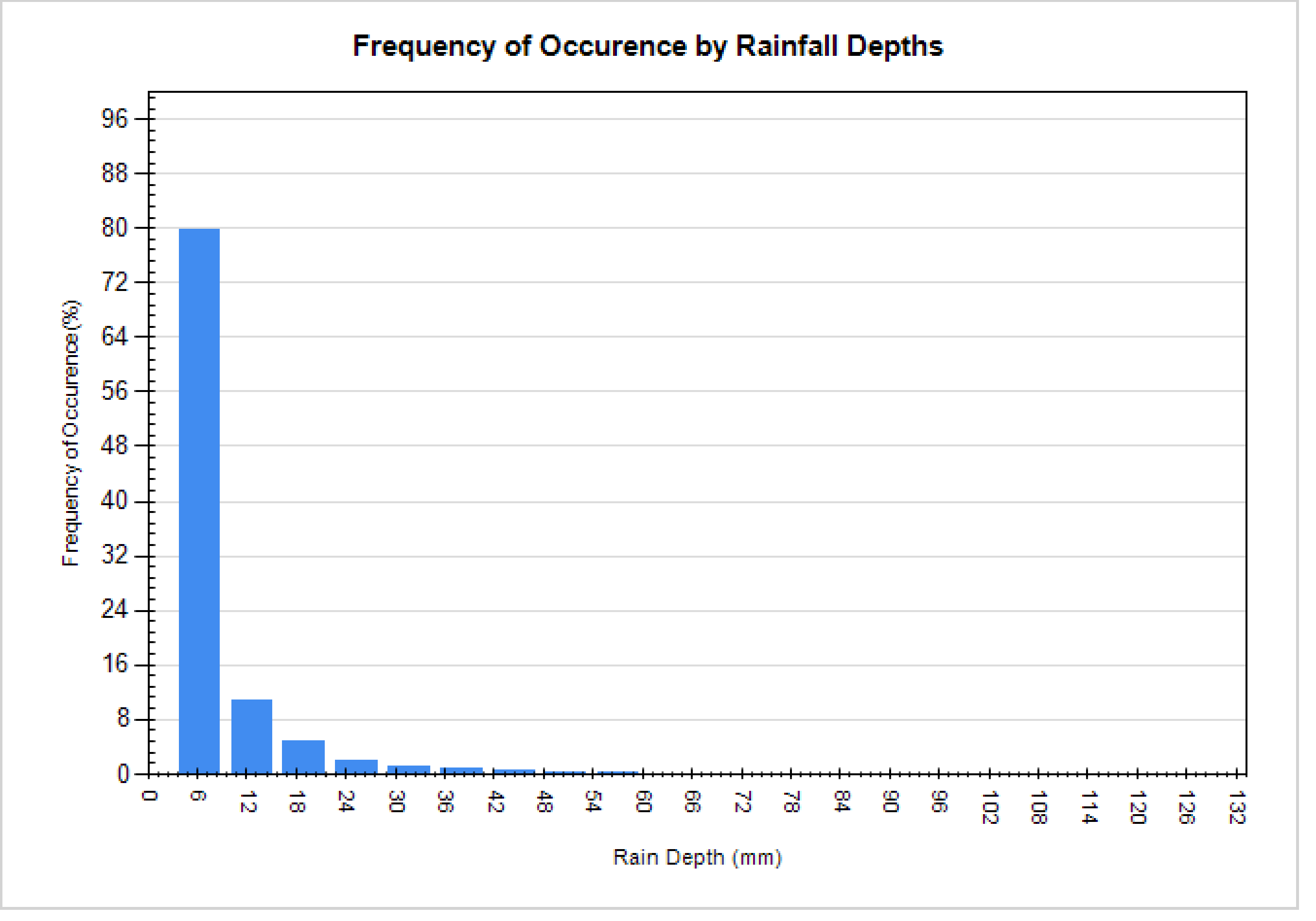
Buildup/Wash-off Parameters	
Target Event Mean Conc. (EMC) mg/L	125
Exponential Buildup Power	0.40
Exponential Washoff Exponent	0.20

TSS Availability Parameters	
Availability Constant A	0.057
Availability Factor B	0.04
Availability Exponent C	1.10
Min. Particle Size Affected by Availability (micron)	400

Cumulative Runoff Volume by Runoff Rate			
Runoff Rate (L/s)	**Runoff Volume (m^3)**	**Volume Over (m^3)**	**Cumulative Runoff Volume (%)**
1	31674	126308	20.1
4	83777	74197	53.0
9	118608	39372	75.1
16	135736	22222	85.9
25	144872	13090	91.7
36	150199	7759	95.1
49	153339	4621	97.1
64	155238	2721	98.3
81	156428	1531	99.0
100	157197	761	99.5
121	157605	353	99.8
144	157823	136	99.9
169	157917	41	100.0
196	157954	5	100.0


Cumulative Runoff Volume by Runoff Rate
For area: 1.75(ha), imperviousness: 51.0%, rainfall station: OTTAWA MACDONALD-CARTIER INT'L A
Cumulative Runoff Volume (%)
Flow (L/s)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190

Rainfall Event Analysis				
Rainfall Depth (mm)	No. of Events	Percentage of Total Events (%)	Total Volume (mm)	Percentage of Annual Volume (%)
6.35	3843	79.7	5885	28.1
12.70	520	10.8	4643	22.1
19.05	225	4.7	3470	16.5
25.40	98	2.0	2144	10.2
31.75	58	1.2	1639	7.8
38.10	32	0.7	1118	5.3
44.45	24	0.5	996	4.7
50.80	9	0.2	416	2.0
57.15	5	0.1	272	1.3
63.50	1	0.0	63	0.3
69.85	1	0.0	64	0.3
76.20	1	0.0	76	0.4
82.55	0	0.0	0	0.0
88.90	1	0.0	84	0.4
95.25	0	0.0	0	0.0
101.60	0	0.0	0	0.0
107.95	0	0.0	0	0.0
114.30	1	0.0	109	0.5
120.65	0	0.0	0	0.0
127.00	0	0.0	0	0.0



For Stormceptor Specifications and Drawings Please Visit:
http://www.imbriumsystems.com/technical-specifications

APPENDIX 2

CITY OF OTTAWA BOUNDARY SURVEY – WATER

BOUNDARY CONDITIONS

Boundary Conditions For: 3058 Jockvale Rd

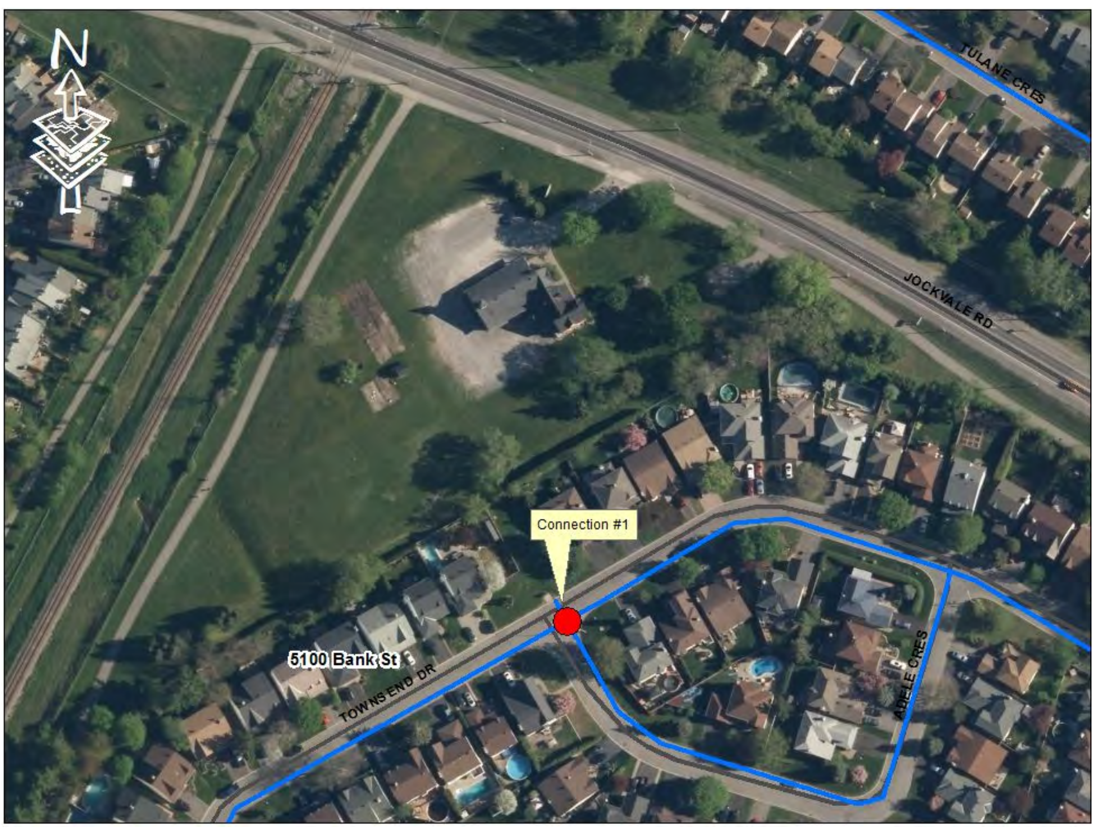
Date of Boundary Conditions: 2018-Apr-30

Provided Information:

Scenario	Demand	
	L/min	L/s
Average Daily Demand	45	0.8
Maximum Daily Demand	72	1.2
Peak Hour	112.5	1.9
Fire Flow #1 Demand	11,000	183.3

Number Of Connections: 1

Location:



BOUNDARY CONDITIONS

Results:

Connection #: 1 pre-configuration

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	155.7	82.1
Peak Hour	144.7	66.3
Max Day Plus Fire (11,000) L/min	121.0	32.7

[1]Elevation: **98.01 m**

Notes:

1) As per the Ontario Building Code in areas that may be occupied, the static pressure at any fixture shall not exceed 552 kPa (80 psi.) Pressure control measures to be considered are as follows, in order of preference:

a) If possible, systems to be designed to residual pressures of 345 to 552 kPa (50 to 80 psi) in all occupied areas outside of the public right-of-way without special pressure control equipment.
b) Pressure reducing valves to be installed immediately downstream of the isolation valve in the home/ building, located downstream of the meter so it is owner maintained.

2) Click or tap here to enter text.

3) Click or tap here to enter text.

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.

APPENDIX 3

FIRE UNDERWRITERS SURVEY REPORT

FIRE FLOW ESTIMATION per FIRE UNDERWRITERS SURVEY
Water Supply For Public Fire Protection - 1999

FIRE FLOW REQUIRED

1. BASE REQUIREMENT

F = 220 C √ A L/min *Where F is the fire flow, C is the type of construction and A is the total flow area*

TYPE OF CONSTRUCTION - WOOD CONSTRUCTION

C	1.5	*Type of Construction Coefficient per FUS Part II, Section 1*
A	1367 m2	*Total Floor Area based on FUS Part II, Section 1*
FIRE FLOW	12,201 L/min	
	12,000 L/min	*ROUNDED TO THE NEAREST 1000 L/min*

ADJUSTMENTS

2. REDUCTION FOR OCCUPANCY TYPE

Limited Combustible	-15%
FIRE FLOW	**10,200** L/min

3. REDUCTION FOR SPRINKLER PROTECTION

Not Sprinklered	0%
REDUCTION	**0** L/min

4. INCREASE FOR SEPARATION DISTANCE

N	> 45 m	0%	
S	30.1 to 45 m	5%	
E	> 45 m	0%	
W	> 45 m	0%	
% INCREASE		**5%**	*Value not to exceed 75% per FUS Part II, Seciton 4*
INCREASE		**510 L/Min**	

TOTAL FIRE FLOW

FIRE FLOW	**10,710** L/min	*Fire flow not to exceed 45,000 L/min nor be less than 2,000 L/min per FUS*
	11,000 L/min	*Section 4 rounded to the nearest 1,000 L/min*

NOTES:

TYPE OF CONSTRUCTION AND OCCUPANCY TYPE INFORMATION PROVIDED BY ARCHITECT
CALCULATIONS BASED ON FIRE UNDERWRITERS SURVEY - PART II

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