TOWN OF Paradise

Response to Applicant - Partial Disclosure
Form 4A

November 15, 2021

[redacted]

Re: Your request for access to information under Part II of the **Access to Information and Protection of Privacy Act, 2015** [Our File # 2021-23]

On October 15, 2021, the Town of Paradise received your request for access to the following records/information:

> *The policy for stormwater detention and/or retention within the Town, in addition to any committee notes, memorandums, or council minutes where the Policy, or a draft thereof, was discussed and/or accepted.*

I am pleased to inform you that a decision has been made by the ATIPP Coordinator for the Town of Paradise to provide access to the requested information. Access to some of the information within the requested record has been refused in accordance with the following mandatory exception to disclosure, as specified in the Access to Information and Protection of Privacy Act, 2015 (the Act):

> *28. (1) The head of a local public body may refuse to disclose to an applicant information that would reveal: (a) a draft of a resolution, by-law, or other legal instrument by which the local public body acts.*

> *28. (1) The head of a local public body may refuse to disclose to an applicant information that would reveal: (c) the substance of deliberations of a meeting of its elected officials or governing body or a committee of its elected officials or governing body, where an Act authorizes the holding of a meeting in the absence of the public.*

Please be advised that you may ask the Information and Privacy Commissioner to review the processing of your access request, as set out in section 42 of the Access to Information and Protection of Privacy Act, 2015 (the Act). A request to the Commissioner must be made in writing within 15 business days of the date of this letter or within a longer period that may be allowed by the Commissioner.

The address and contact information of the Information and Privacy Commissioner is as follows:

Office of the Information and Privacy Commissioner
2 Canada Drive
P. O. Box 13004, Stn. A
St. John's, NL. A1B 3V8
Telephone: (709) 729-6309
Toll-Free: 1-877-729-6309
Facsimile: (709) 729-6500

You may also appeal directly to the Supreme Court within 15 business days after you receive the decision of the public body, pursuant to section 52 of the Act.

If you have any further questions, please feel free to contact me by telephone or email.

Sincerely,

Corey Smith
for

Terrilynn Smith
ATIPP Coordinator

ATIPP Record 1

ATIPP Section 28 (1) a

[redacted]

ATIPP Section 28 (1) a

[redacted]

[redacted]

ATIPP Section 28 (1) a

[redacted]

ATIPP Section 28 (1) a

[redacted]

ATIPP Section 28 (1) a

[redacted]

ATIPP S 288 (1) a

[redacted]

ATIPP Section 28 (1) a

[redacted]

ATIPP Section 28 (1) a

ATIPP Record 3

Town of Paradise

Infrastructure and Public Works Committee Meeting

June 24, 2020, 5:00 p.m.

Virtual Meeting

ATIPP Section 28 (1) c

[redacted]

[redacted]

ATIPP Section 28 (1) c

[redacted]

[redacted]

[redacted]

ATIPP Section 28 (1) c

[redacted]

INFRASTRUCTURE AND PUBLIC WORKS COMMITTEE:

Paradise Stormwater Management Plan
M20-047
Moved by Councillor Quilty, seconded by Councillor Martin

“BE IT RESOLVED, that the Town of Paradise accept the Stormwater Management Plan prepared by CBCL Limited dated December 2019, and proceed with recommendations to make capital works improvements and prepare and adopt a net zero runoff policy, as identified in the Plan.”

Motion carried unanimously

ATIPP
Record 5

STORMWATER DETENTION POLICY (Revised November 23, 2015)

1. POLICY STATEMENT

The purpose of this document is to provide policy direction for the provision of stormwater detention systems for new developments .

2. POLICY OBJECTIVE

The objective of this Policy is to ensure that developments, where required, provide stormwater detention that temporarily stores the difference in volume between the City's pre-development and post-development design storms.The design storms to be analyzed are the 25-year, 50-year and 100-year return periods. Each design storm must be analyzed for the 1 hour, 2 hour, 6 hour, 12 hour and 24 hour durations.The proposed detention system must limit the post-development runoff rate from the development for each return period/duration to the respective pre-development runoff rate for the same return period/duration. All computer modeling of stormwater detention must be done with the latest version of XPSWMM. All detention designs should ensure the preservation of the environment and fish habitat.

3. POLICY APPLICATION

The Policy applies to all new developments within the City of St. John's with the exception of:

- Downtown St. John's - subject to City infrastructure having sufficient capacity as determined by the Department of Planning, Engineering and Regulatory Services;
- Infill development can be exempted from stormwater detention provided that the Developer demonstrate that there will be no downstream issues associated with capacity, flooding, erosion control, and velocities;
- Cemeteries, grassed playing fields, and vegetated areas of public sports and recreational facilities;
- Where there is a written agreement between the Developer and the City to provide stormwater infrastructure improvements that remedy the downstream flooding problems in lieu of constructing a stormwater detention system. The Developer would be required to provide the City with a certified cheque or an acceptable Irrevocable Letter of Credit for the value, as determined by the City, of the downstream flood remediation work; and
- Other areas where the Department of Planning, Engineering and Regulatory Services determines, based on hydrologic/hydraulic analysis, that stormwater detention is not necessary, or may be permissible at a reduced level.

4. EFFECTIVE DATE OF POLICY

The Policy came into effect January 1, 2013, and this revision is effective immediately.

5. DEVELOPER'S RESPONSIBILITY

It is the responsibility of the Developer(s) to submit for City approval a stormwater management plan which meets the requirements of this Policy. The City reserves the right to accept or reject the stormwater management plan, or propose amendments to the plan. Where requested by the Developer, the City may provide guidance as to the type of stormwater detention which might be acceptable for a particular development. The latest version of the City's Subdivision Design Manual provides the design methodology that the Developer must use to design and construct the stormwater detention system.

6. REGIONAL DETENTION

The City may, where it is considered more effective, direct Developers to cooperate in, and fund the cost of, a regional detention system as a condition to a development(s) proceeding. A regional detention system would establish large scale stormwater detention structure(s) to meet this Policy's requirements for several developments within a geographic region. Similarly, a Developer(s) may also propose a regional stormwater detention system to the City.

7. DETENTION INFRASTRUCTURE COSTS

Developers will fund all costs of stormwater infrastructure constructed within the borders of their property. In the case of a regional stormwater detention system, where the detention infrastructure serves more than one development, the regional detention infrastructure costs will be shared among developers in proportion to the amount of stormwater volume each development is expected to detain. Where the City must upgrade its infrastructure outside the borders of the development, the City may recover its costs, including interest and financial charges, through assessment charges/fees against developable properties served by, or to be served by, the stormwater detention system.

8. OWNERSHIP

Stormwater detention systems in residential developments may be accepted for ownership and maintenance by the City. Detention systems in Commerical, Industrial, or Institutional developments will not be accepted for ownership by the City.

9. ACCEPTANCE

Stormwater detention systems whose ownership is to be conveyed to the City are subject to the following requirements:

a) The Developer must construct a stormwater detention system in accordance with the approved engineering plans and must convey the system, and associated lands, at no cost to the City as a condition of Final Approval.

b) The system must be 100% complete (in accordance with the approved plans), operational, and commissioned in the presence of the Water & Wastewater Division (or their designate).The Developer must continue to own and maintain the detention system until accepted by the City.

c) The City, at its discretion, may require a field test to demonstrate the maximum discharge rate from the detention facility for a designated water level/head.

Policy: 08-04-19
Stormwater Detention Policy

Passed By Council on:01/07/2013

Purpose
The purpose of this document is to provide policy direction when stormwater detention systems are required for development where an increase in stormwater runoff may: a) contribute to risk of flooding, and/or b) exceed the capacity of City storm sewers, bridges/culverts, river channels, or ditches.

Policy Statement

2. POLICY OBJECTIVE
The objectives of the Stormwater Detention Policy are to:

a) Temporarily store the difference in volume between the 100-year 24-hour post-development runoff and the 100-year 24-hour pre-development runoff while limiting the post-development runoff rate from a development to the pre-development runoff rate.

b) Prevent increases in downstream flooding and drainage problems that could increase flood losses, damage public assets, reduce property values, and require additional capital works expenditures for flood mitigation.

c) Encourage integration of the detention system into a sustainable overall stormwater management plan for the development, and

d) Promote the incorporation of detention systems into the engineering design and layout of the development so that adequate storage areas are included in the initial stages.

3. POLICY APPLICATION
The Policy applies to all developments within the City of St. John's which present an immediate or foreseeable risk of flooding, with the exception of:

a) Developments in areas, such as Downtown, where the storm sewer system discharges directly into the Atlantic Ocean - subject to City storm sewer infrastructure having sufficient capacity as determined by the Director of Engineering,

b) Developments comprising a land area of less than 0.5 hectares and where the increase in stormwater runoff is less than or equal to 25 liters per second,

c) New developments in subdivisions where a stormwater detention system has already been provided for the entire subdivision,

d) The grassed playing field and vegetated area of public sports and recreational facilities that are not part of a development,

e) Locations where such a system would, due to timing of outflows, have an adverse effect on downstream properties by increasing peak rates of runoff – as determined by the Director of Engineering,

f) Where there is a written agreement between the Developer and the City to provide stormwater infrastructure improvements that remedy the downstream flooding problems in lieu of constructing a stormwater detention system. The Developer would be required to provide the City with a certified cheque or an acceptable Irrevocable Letter of Credit for the value, as determined by the City, of the downstream flood remediation work,

g) Small size developments where it can be demonstrated to the satisfaction of the Director of Engineering that the stormwater detention system would have no beneficial effect to downstream properties, and

h) Other areas where the Director of Engineering determines, based on hydrologic/hydraulic analysis, that stormwater detention is not necessary, or may be permissible at a reduced level.

4. AREA OF THE DEVELOPMENT TO WHICH STORMWATER DETENTION APPLIES
Generally, stormwater detention applies to the entire development with the following exceptions:

a) On already-developed property, the stormwater detention system requirements only apply to the area of the new development – provided runoff from previously developed areas can be excluded from the detention storage,

b) In residential subdivisions where new public roads will be created, the stormwater detention requirements will apply to the entire development area – including streets and lots. However, any areas of a lot that remain in a natural undeveloped state may be excluded from the area to be controlled by the stormwater detention system provided that flows from
these areas can be diverted around the detention system. Approval from the Director of Engineering must be obtained before excluding any area from the detention requirements.

c) Where the proposed development is on previously developed vacant site or is a complete redevelopment of an already-developed property, the stormwater detention system requirement will be applicable to the entire property.

5. EFFECTIVE DATE OF POLICY
This Policy will come into effect on January 1, 2013. Development applications which have been received by the City prior to January 1, 2013, and where construction is substantially underway by September 1, 2013, as determined by the Director of Engineering, will be exempt from this Policy – unless the City has already advised that stormwater detention is required or there is a capacity issue in the receiving storm sewer system.

6. DEVELOPER'S RESPONSIBILITY
It is the responsibility of the Developer(s) to submit for City approval a stormwater management plan which meets the requirements of this Policy. The City reserves the right to accept or reject the stormwater management plan, or propose amendments to the plan. Where requested by the Developer, the City may provide guidance as to the type of stormwater detention which might be acceptable for a particular development. The City's Stormwater Detention Design Manual provides the design standards that the Developer must use to design and construct the stormwater detention system.

7. REGIONAL DETENTION
The City may, where it is considered more effective, direct Developers to cooperate in, and fund the cost of, a regional detention system as a condition to a development(s) proceeding. A regional detention system would establish large scale stormwater detention structure(s) to meet this Policy's requirements for several developments within a geographic region. Similarly, a Developer(s) may also propose a regional stormwater detention system to the City.

8. DETENTION INFRASTRUCTURE COSTS
Developers will fund all costs of stormwater infrastructure constructed within the borders of their property. In the case of a regional stormwater detention system, where the detention infrastructure serves more than one development, the regional detention infrastructure costs will be shared among developers in proportion to the amount of stormwater volume each development is expected to detain. Where the City must upgrade its infrastructure outside the borders of the development, the City may recover its costs, including interest and financial charges, through assessment charges/fees against developable properties served by, or to be served by, the regional stormwater detention system.

9. OWNERSHIP
Stormwater detention systems in residential developments may be accepted for ownership and maintenance by the City. Detention systems in Commerical, Industrial, or Institutional developments will not be accepted for ownership by the City.
The City of St. John's provides no maintenance of stormwater detention systems located on private property. Maintenance must be provided by the owner of the property upon which the detention system resides – unless there is an agreement between the owner and the City which supercedes the preceding.

10.ACCEPTANCE
Acceptance of stormwater detention systems is subject to the following requirements:

a) A Developer owning property with an area greater than 0.5 hectares must construct a stormwater detention system in accordance with the approved engineering plans and must convey the system, and associated lands, at no cost to the City as a condition of Final Approval subject to the requirements of Section 9 of this Policy.

b) The City will not accept the detention system until (a) the system has been fully completed in accordance with the approved plans, (b) 80% of the proposed lots have been fully developed, and (c) adequate erosion control measures, as approved by the Director of Engineering, have been installed on the remaining 20% of the lots. The Developer must continue to own and maintain the detention system until accepted by the City.

Application

Responsibilities

All Departments

Definition

References/Appendix

Monitoring and Contravention

Approvals

Public Works& Environment Standing Committee Report - December 11, 2012; Regular Meeting of Council January 7, 2013

Review Period

Note: research / reference record

8.0 STORMWATER DETENTION

Stormwater detention facilities must be designed and constructed to ensure there is a net-zero-runoff from the proposed development for the rainfall design events contained within this document. The hydrology and hydraulics of the facility, and any connecting storm sewer infrastructure, must be modeled using the latest version of XPSWMM. Based on the proposed construction drawings, it is incumbent on the Developer to provide a functioning electronic XPSWMM model for review that demonstrates that the planned stormwater detention facility can meet the City's net-zero-runoff requirement.

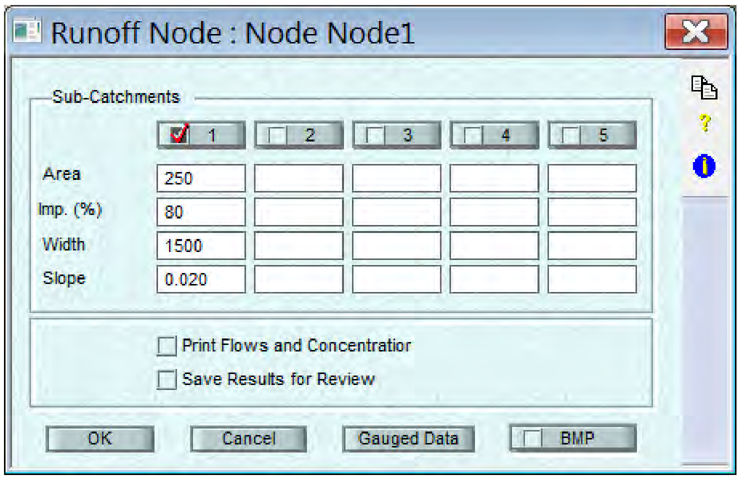
The design philosophy is to (a) determine the required volumes of storage for the 24-hour events for each of the return periods listed in Tables 1 through 6; (b) select an outlet control device or group of devices that controls the post-development flows to their respective pre-development flow condition (while ensuring that the emergency overflow device is not engaged); and (c) route the post-development runoff from the remaining rainfall design events for each return period through the detention facility and adjust the volume / outlet control accordingly until net-zero-runoff is achieved.

8.1 HYDROLOGY

The RUNOFF routing method along with the Green-Ampt infiltration method must be used to determine the pre-development and post-development hydrographs, required to estimate the stormwater detention volume. Subcatchments must be defined in the model for each structure that will direct runoff into the system (i.e catchbasins, ditch inlets, headwalls, aboveground storage ponds, culverts, diversions, etc.).

8.1.1 NODE DATA (Runoff Mode)

Hydrological data can be entered for each node through the Runoff Node window. Up to five subcatchments can be assigned to each node. The City requires input data for the Area, Percent Impervious, Width, and Slope parameters. Gauged data can also be added at each node as well as BMP treatment processes.



8.1.1.1 AREA

The area of the subcatchment(s), in hectares, shall be delineated based on the City's latest contour mapping. Care should be taken to include/exclude any areas that are artificially directed into or diverted away from a subcatchment through underground conduits or changes to topographic features from existing or proposed development.

8.1.1.2 PERCENT IMPERVIOUS

Percent of subcatchment which is impervious; defined by the sum of the areas of all existing/proposed asphalt and/or concrete surfaces, and the areas of all rooftops which are directly connected to these surfaces (or directly connected to an existing/proposed storm sewer) divided by the subcatchment total area. Multi-phased developments where the aforementioned cannot be readily quantified may assume 80%

imperviousness for future phases.

8.1.1.3 WIDTH

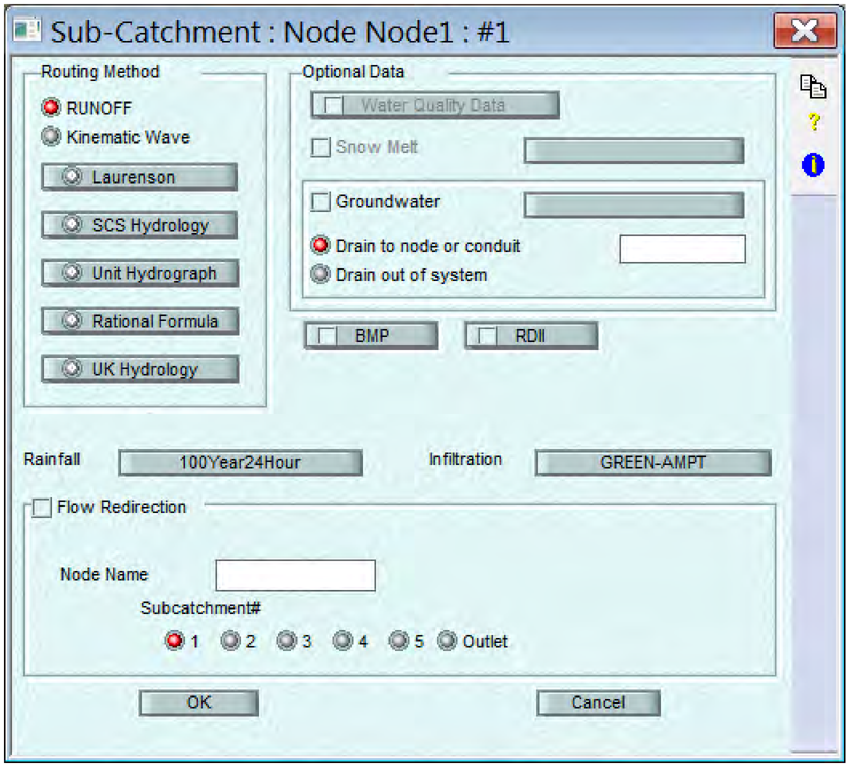
The subcatchment width is a key calibration parameter that significantly affects the hydrograph shape. A good estimate for the width is the subcatchment AREA divided by the overland flow path length. The overland flow path length is defined as the maximum length from the furthest drainage point of the subcatchment to the outlet. The City may require the use of a different WIDTH formula depending on the location of the subcatchment within the City and previous calibration work performed in that area.

8.1.1.4 SLOPE

Average slope along the overland flow path to the outlet of the subcatchment in m/m.

8.1.2 NODE SUBCATCHMENT DATA (Runoff Mode)

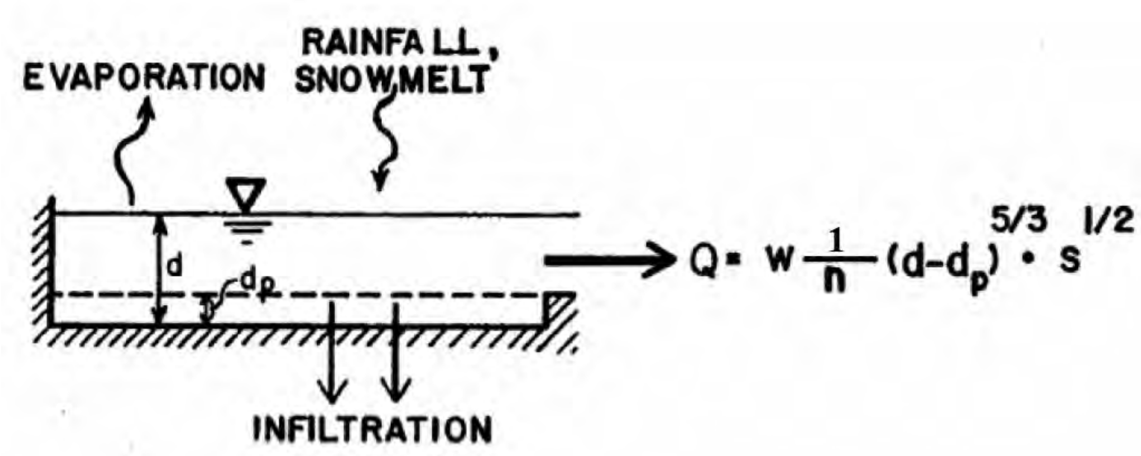
The user can select for each subcatchment the Routing Method, Rainfall event, Infiltration method as well as other optional data such as Water Quality, Snowmelt, Groundwater, BMP, RDII, and Flow Redirection.



For stormwater detention modeling the City requires that all hydrographs be generated with the SWMM Runoff Routing Method using the Green-Ampt infiltration method. The rainfall events to be modeled are discussed in Section 1.5.

8.1.3 SWMM RUNOFF ROUTING METHOD

XPSWMM models each catchment as a non-linear reservoir where input originates from rainfall, snowmelt and any upstream subcatchment.



Infiltration, evaporation, and surface runoff (Q) are output from the model. The non-linear reservoir has a capacity equal to the maximum depression storage (d_p). Surface runoff will only be generated when the depth of water (d) in the reservoir exceeds the maximum depression storage. Subcatchment runoff rates are determined by combining the Continuity equation, below, and the above version of Manning's formula (where w = width [m], s = slope [m/m], n = friction coefficient).

$$\frac{dV}{dd} = A\frac{dd}{dt} = Ai - Q$$

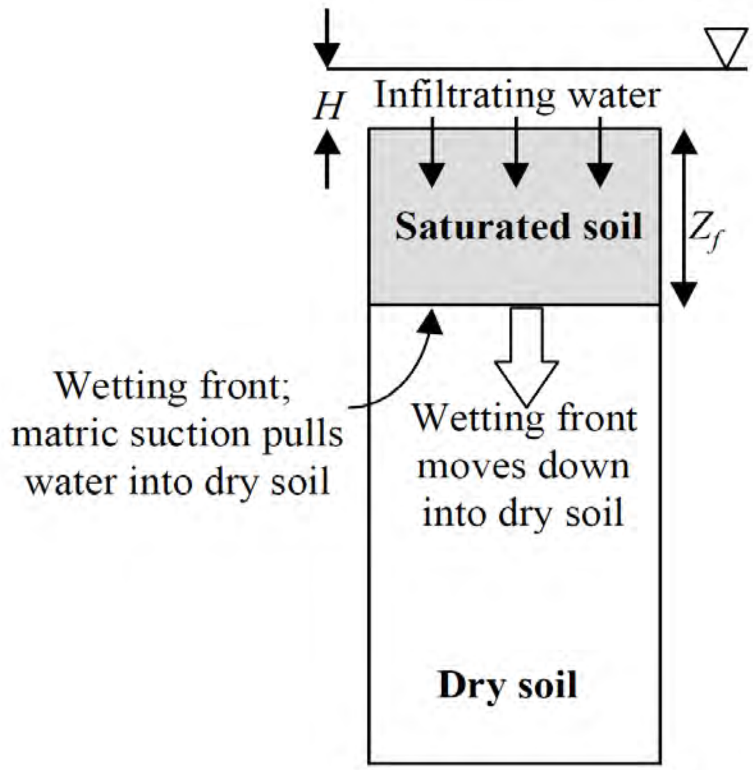
Where,

V	=	volume of water on the subcatchment [m^2]
d	=	water depth [m]
t	=	time [seconds]
A	=	surface area of the subcatchment [m^2]

I	=	rainfall excess (rainfall intensity minus evaporation and infiltration) [m/s]
Q	=	runoff rate [m^3/s]

8.1.4 GREEN-AMPT INFILTRATION METHOD

XPSWMM uses the Green-Ampt method to estimate losses due to infiltration. This method is preferred over the Horton, Uniform Loss, and SCS methods because of its precise physical basis.



The Green-Ampt equation (1911) is a representation of infiltration under a ponded surface. It assumes a homogenous soil profile, a uniform distribution of antecedent soil moisture, and that the movement of water through the soil takes place through an advancing wetting front. It was modified by Mein and Larson (1973) and Chu(1978) to allow the modeling of unsteady rainfall. XPSWMM adopts a two-stage model for infiltration whereby the volume of water which will infiltrate before the surface saturates is determined and from that point onward, infiltration capacity is estimated by the Green-Ampt equation. The Green-Ampt infiltration equation has three parameters to be specified: (1) average capillary suction at the wetting front, (2) saturated hydraulic conductivity of the soil, and (3) initial moisture deficit for the rainfall event.

8.1.4.1 AVERAGE CAPILLARY SUCTION

In soil, a wetting front separates the fully saturated soil from the dry soil and it is assumed that the wetting front moves down as a wet body parallel to the surface. Through this movement a suction head is created due to the capillary attraction in the soil voids. Unless it can be satisfactorily demonstrated otherwise, based on appropriate geotechnical field tests, then the average capillary suction at the wetting front shall be taken as 200mm.

8.1.4.2 SATURATED HYDRAULIC CONDUCTIVITY

The saturated hydraulic conductivity is a quantitative measure of a saturated soil's ability to transmit water when subjected to a hydraulic gradient. Unless it can be satisfactorily demonstrated otherwise, based on appropriate geotechnical field tests, then the saturated hydraulic conductivity of the soil shall be taken as 0.001 mm/hr.

8.1.4.3 INITIAL MOISTURE DEFICIT

The initial moisture deficit is the fractional difference between soil porosity and actual moisture content. The recommended default value is 0.30.

8.1.4.4 IMPERVIOUS DEPRESSION STORAGE

Water stored as depression storage on impervious areas which is depleted by evaporation. The recommended default value is 1mm.

8.1.4.5 PERVIOUS DEPRESSION STORAGE

Water stored as depression storage on pervious areas which is subject to infiltration and evaporation. The recommended default value is 2.5 mm.

8.1.4.6 MANNING’S n IMPERVIOUS

The Manning’s roughness for the subcatchment impervious areas such asphalt roads, driveways, and parking lots; concrete surfaces; and rooftops. The recommended default value is 0.020.

8.1.4.7 MANNING’S n PERVIOUS

The Manning’s roughness for the subcatchment pervious areas including urban grassed, treed and landscaped areas. The recommended default value is 0.50.

8.1.4.8 ZERO DETENTION

The zero detention parameter is an estimate of the percentage of the subcatchment impervious area with zero depression storage (i.e. immediate runoff). The recommended default value is 25%.

8.1.5 RAINFALL DESIGN EVENTS

The cumulative rainfall hyetographs to be used in stormwater detention design are listed in the below tables.

TABLE 1: 100-Year Design Cumulative Hyetographs

Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	3.0	10	4.3	0.5	6.0	1	7.3	2	8.9
10	7.4	20	10.8	1.0	15.0	2	18.3	4	22.4
15	13.2	30	19.1	1.5	26.5	3	32.5	6	39.7
20	20.7	40	30.0	2.0	41.7	4	51.1	8	62.4
25	29.6	50	42.9	2.5	59.7	5	73.1	10	89.3
30	36.5	60	52.9	3.0	73.6	6	90.1	12	110.1
35	38.9	70	56.3	3.5	78.3	7	95.8	14	117.1
40	40.6	80	58.8	4.0	81.8	8	100.2	16	122.5
45	42.1	90	60.9	4.5	84.7	9	103.7	18	126.8
50	43.1	100	62.3	5.0	86.7	10	106.2	20	129.8
55	43.5	110	63.0	5.5	87.7	11	107.4	22	131.2
60	43.8	120	63.4	6.0	88.2	12	108.0	24	132.0

TABLE 2: 50-Year Design Cumulative Hyetographs

Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	2.7	10	3.8	0.5	5.5	1	6.7	2	8.3
10	6.7	20	9.7	1.0	13.7	2	16.9	4	20.8
15	11.9	30	17.1	1.5	24.3	3	30.0	6	36.8
20	18.7	40	27.0	2.0	38.3	4	47.2	8	57.9
25	26.8	50	38.6	2.5	54.8	5	67.6	10	82.8
30	33.0	60	47.5	3.0	67.6	6	83.3	12	102.1
35	35.1	70	50.6	3.5	71.9	7	88.6	14	108.6
40	36.7	80	52.9	4.0	75.2	8	92.7	16	113.6
45	38.0	90	54.7	4.5	77.8	9	96.0	18	117.6
50	38.9	100	56.0	5.0	79.6	10	98.2	20	120.3
55	39.4	110	56.7	5.5	80.5	11	99.3	22	121.7
60	39.6	120	57.0	6.0	81.0	12	99.9	24	122.4

TABLE 3: 25-Year Design Cumulative Hyetographs									
Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	2.4	10	3.4	0.5	5.0	1	6.2	2	7.5
10	6.0	20	8.6	1.0	12.5	2	15.5	4	18.7
15	10.7	30	15.2	1.5	22.2	3	27.5	6	33.2
20	16.8	40	23.9	2.0	34.9	4	43.2	8	52.2
25	24.0	50	34.2	2.5	49.9	5	61.8	10	74.7
30	29.6	60	42.2	3.0	61.5	6	76.2	12	92.1
35	31.5	70	44.9	3.5	65.5	7	81.1	14	98.0
40	32.9	80	47.0	4.0	68.5	8	84.8	16	102.5
45	34.1	90	48.6	4.5	70.9	9	87.8	18	106.0
50	34.9	100	49.7	5.0	72.5	10	89.8	20	108.5
55	35.3	110	50.3	5.5	73.4	11	90.9	22	109.8
60	35.5	120	50.6	6.0	73.8	12	91.4	24	110.4

TABLE 4: 10-Year Design Cumulative Hyetographs									
Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	2.0	10	2.8	0.5	4.3	1	5.4	2	6.5
10	5.1	20	7.2	1.0	10.9	2	13.6	4	16.3
15	9.0	30	12.7	1.5	19.3	3	24.0	6	28.8
20	14.1	40	20.0	2.0	30.4	4	37.8	8	45.4
25	20.2	50	28.5	2.5	43.4	5	54.1	10	64.9
30	24.9	60	35.2	3.0	53.5	6	66.6	12	80.1
35	26.5	70	37.4	3.5	57.0	7	70.9	14	85.2
40	27.7	80	39.2	4.0	59.6	8	74.1	16	89.1
45	28.7	90	40.5	4.5	61.7	9	76.7	18	92.2
50	29.4	100	41.5	5.0	63.1	10	78.5	20	94.4
55	29.7	110	42.0	5.5	63.8	11	79.4	22	95.4
60	29.9	120	42.2	6.0	64.2	12	79.9	24	96.0

TABLE 5: 5-Year Design Cumulative Hyetographs									
Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	1.7	10	2.4	0.5	3.8	1	4.8	2	5.7
10	4.3	20	6.1	1.0	9.6	2	12.0	4	14.3
15	7.7	30	10.8	1.5	16.9	3	21.2	6	25.2
20	12.1	40	16.9	2.0	26.7	4	33.4	8	39.7
25	17.3	50	24.2	2.5	38.2	5	47.8	10	56.8
30	21.4	60	29.9	3.0	47.0	6	58.9	12	70.1
35	22.7	70	31.8	3.5	50.0	7	62.6	14	74.5
40	23.8	80	33.2	4.0	52.3	8	65.5	16	78.0
45	24.6	90	34.4	4.5	54.2	9	67.8	18	80.7
50	25.2	100	35.2	5.0	55.4	10	69.4	20	82.6
55	25.5	110	35.6	5.5	56.1	11	70.2	22	83.5
60	25.6	120	35.8	6.0	56.4	12	70.6	24	84.0

TABLE 6: 2-Year Design Cumulative Hyetographs									
Time (min)	1hr Rain (mm)	Time (min)	2hr Rain (mm)	Time (hr)	6hr Rain (mm)	Time (hr)	12hr Rain (mm)	Time (min)	24hr Rain (mm)
0	0.0	0	0.0	0.0	0.0	0	0.0	0	0.0
5	1.3	10	1.8	0.5	3.0	1	3.8	2	4.5
10	3.3	20	4.5	1.0	7.5	2	9.6	4	11.4
15	5.8	30	8.1	1.5	13.3	3	17.0	6	20.2
20	9.2	40	12.7	2.0	21.0	4	26.8	8	31.8
25	13.1	50	18.1	2.5	30.0	5	38.3	10	45.5
30	16.2	60	22.4	3.0	37.0	6	47.2	12	56.0
35	17.2	70	23.8	3.5	39.4	7	50.2	14	59.6
40	18.0	80	24.9	4.0	41.2	8	52.5	16	62.4
45	18.6	90	25.7	4.5	42.6	9	54.4	18	64.5
50	19.1	100	26.3	5.0	43.6	10	55.6	20	66.1
55	19.3	110	26.6	5.5	44.1	11	56.3	22	66.8
60	19.4	120	26.8	6.0	44.4	12	56.6	24	67.2

The job control for evaporation must be the default setting of 3mm per day. Simulation times must be long enough to produce the complete hydrograph for each model.

8.2 HYDRAULICS

All manholes, catchbasins, ditch inlets, inlet headwalls, and outfalls must be represented as nodes in XPSWMM corresponding to the design information on the engineering drawings. Likewise, all storm sewer mains, catchbasin leads, open channels, culverts and bridges must be represented as links in XPSWMM. Orifice, weirs, riser pipes, and in general any outlet control device, must be modeled within XPSWMM as a multi-link. Detention facilities can be modeled as nodes using the storage option or as links if an underground oversized pipe facility is being designed. Emergency overflows must be included in the model and they may be modeled as links or multi-links, depending on the application. All nodes must be georeferenced in the x-y-z direction to the City's NAD83 coordinate and geodetic systems.

The routing of all hydrographs must be done using the Dynamic Wave Hydrograph Method using the default values for the performance/stability factors.

The junction surface area default is 1.2 square meters. This is a global parameter so care must be taken that nodes that have larger diameters/areas than the default are accounted for in the Node Data window for the hydraulics mode.

It is preferred that the hydraulic grade line, for each proposed pipe in the development, not extend above the crown of the pipe. If this cannot be avoided then the hydraulic grade line and the lowest floor elevation of all basements must be plotted on the Plan & Profile drawings demonstrating there will be no impact on residences. This is not a requirement for developments with slab-on-grade structures but under no circumstances can the hydraulic grade line be higher than the ground elevation of any node.

8.2.1 CATCHBASINS

For all areas requiring stormwater detention and where an approved 2D catchbasin analysis has not been provided then the following will be required.

In residential areas manholes spacing must be no greater than 60m with two double catchbasins, located on opposite sides of the street, connecting to each manhole on a continuous grade. Manholes in sags shall have four double catchbasins – two located on each side of the street. T-intersections shall require three double catchbasins and four-way intersections will require four double catchbasins.

In parking lots on continuous grades a minimum of one double catchbasin will be required for every 1,000 square meters. For parking lots designed as sags, two double catchbasins for every 3,000 square meters will be required. As well, two double catchbasins will be required on the turnouts for each access/egress. For commercial developments that have steep access/egress then a slotted drain across the full width of the access/egress may also be required.

8.3 WET PONDS

Stormwater Detention Facilities receive stormwater runoff primarily from conveyance systems (eg. ditches, roads, storm sewers) and discharge at or less than pre-development flow rates to receiving waters or to existing storm sewer systems. **All stormwater detention facilities require the appropriate approvals from the City of St. John’s, the Provincial Department of Environment and Conservation, and the Federal Department of Fisheries and Oceans.**

8.3.1 Definitions

<u>**Active storage:**</u> The temporary storage volume provided in a stormwater pond. In a wet pond this is the storage between **PWL** and **HWL**.

<u>**Aquatic Bench**</u>: Those shallow areas (0.5m deep) around the edge of a permanent pool of a stormwater detention facility that support aquatic vegetation – both submerged and emergent.

<u>**Detention storage**</u>: The temporary storage and gradual release of stormwater in a storage element.

<u>**HWL**</u>: The high water elevation in the wet pond for the 100-year event.

<u>**Inactive Storage**</u>: Often referred to as “dead storage”, it is the volume of water between the pond bottom and the PWL.

<u>**Permanent pool**</u>: The portion of a stormwater pond which retains a permanent volume and depth of water.

<u>**PWL**</u>: The permanent water elevation created by the inactive storage of a permanent pool.

<u>**Sediment Forebay**</u>: A permanent pool that is designed to facilitate maintenance and improve sediment removal by trapping larger particles near the inlet of the pond. The forebay is designed to be the deepest area of the pond to minimize the potential for particle re-suspension and prevent conveyance of the suspended material to the outlet.

8.3.2 Level of Service

Stormwater detention facilities must be designed to provide adequate flood protection (storage volume for quantity control). **All stormwater facilities must be designed to provide active storage for the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year return period 24-hour design storms.**

8.3.3 Overland Drainage Routes

.1 Overland and underground drainage routes that direct flows from the design storm events to the storage detention facility must be provided.

.2 An emergency overland escape route from all storage detention facilities must be provided. In general, the escape route must provide a minimum capacity between 1 and 2 cubic meters per second. Appropriate capacity should be determined at the time of design. Optionally and with the approval of the Department of Engineering, additional freeboard may be considered in cases where it is difficult to establish an escape route. The additional freeboard would provide a higher level of service overall.

.3 Sanitary sewer manholes must be located outside of impoundment (pond) areas. Whenever possible, sanitary sewer manholes should not be located within the overland drainage route.

.4 Sanitary sewer manholes located within overland drainage routes must be sealed. Bolting is at the discretion of the Environmental Services Division.

8.3.4 Wet Pond Storage Detention Facilities

Wet ponds are impoundment areas used to temporarily store stormwater runoff in order to: promote settlement of runoff pollutants, restrict downstream discharge to or below predevelopment levels, to minimize downstream flooding and reduce erosion potential. Wet ponds are similar to lakes and ponds in the St. John's area in that there is always a permanent body of water. During runoff events, additional temporary storage is provided above the permanent water level. After the runoff event, the water level gradually recedes back to its original pond level. Wet ponds may be constructed by an embankment or through excavation of a depression. Design of the facility usually includes the upper stage (above PWL), where the volume from runoff events is stored, and the lower stage (below PWL), where sedimentation is promoted. It is the lower stage that provides the pond's primary source of water quality enhancement. Sediment forebays are required on all wet ponds to help confine settlement for larger pollutant particles.

8.3.5 Volumetric Sizing

Wet ponds must provide an active storage volume for the 100 year return period rainfall event.

- As a minimum, the permanent pool (pond bottom to PWL) must be sized for a volume equal to the 2-year 24-hour runoff generated by the entire contributing drainage area with full development of the proposed site. The aquatic bench must have a maximum 7:1 side-slope while areas of the pond deeper than 0.5m must have a maximum 4:1 slope.

- As a minimum, active storage (PWL to HWL) must be sized for a volume equal to the 100-year 24-hour runoff generated by the entire contributing drainage area with full development of the proposed site. Slopes above the PWL must be 4:1 maximum. The 100 year volume must be contained before spillover is permitted.

Release rates from the ponds must be restricted to the pre-development flow condition or downstream limiting capacity, whichever is less.

8.3.6 Land Dedication

.1 Wet ponds that are to be ultimately operated by The City of St. John's are to be located on land which is owned by the City or will be conveyed to the City as a condition of approval for the development.

.2 Wet ponds should not be located in river valleys unless there are no other viable locations.

.3 The maximum level of inundation, the high water level (HWL), must not encroach onto private property. Lots bordering the wet pond are required to have abutting property elevations a minimum of 0.3 m above the spillover elevation of the pond and basement elevations must be 0.3m above the HWL and HGL.

8.3.7 Drainage Area

There is no minimum drainage area requirement as the bottom and sides of as the wet pond must be lined with a geosynthetic clay liner.

8.3.8 Winter Operation

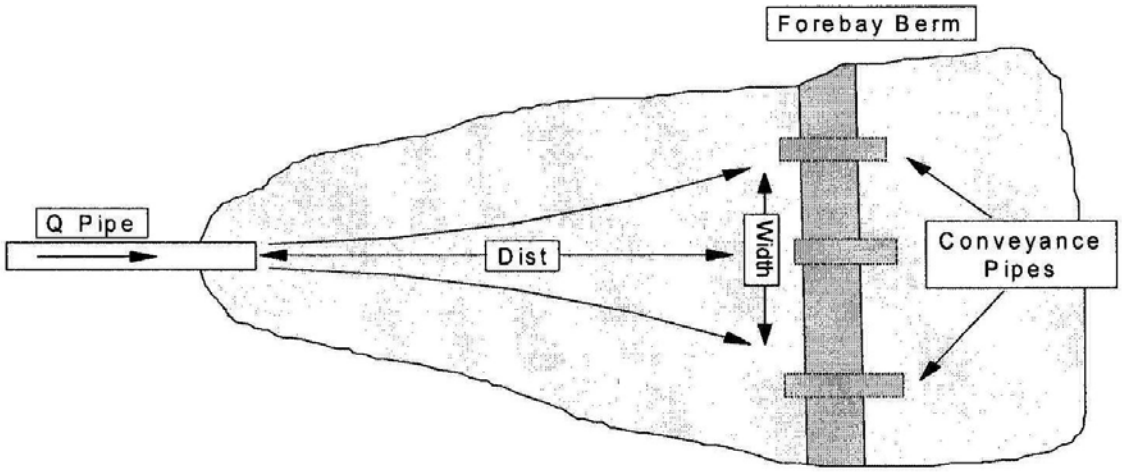
During the winter, ice cover will reduce the design volume of the permanent pool. To compensate for the loss of volume due to ice cover a 1 meter freeboard must be constructed between the HWL and the emergency spillway. Precautions should be taken to minimize the effects of freezing of pipes and orifices.

8.3.9 Circulation

Narrow and/or dead bay areas are not permitted. Inlets and outlets should be located to maximize detention time and circulation, and to reduce short-circuiting through the pond.

8.3.10 Sediment Forebay

A sediment forebay facilitates maintenance and improves sediment removal of larger particles near the inlet of the pond. The forebay should be one of the deeper areas of the pond to minimize the potential for particle re-suspension. **Sediment forebays are required on all wet ponds.** The forebay can be included within the wet pond area or as a separate facility. As well, each inlet location must have a forebay.



Sizing

Sizing of the forebay depends on the inlet configuration. There are several calculations that need to be made to ensure that it is adequately sized. **In all cases, the forebay length should be greater than, or equal to, the larger of the two forebay lengths determined below by Equations 8.0 and 8.1.**

8.3.10.1 Forebay Length Using Settling Calculations

The primary method to calculate forebay volume and length must be based on settling calculations. The calculations determine the distance required to settle out a certain size of sediment. It is assumed that the flow out of the pond dictates the velocity through the forebay and the rest of the pond. Although this is not entirely correct, it is reasonable for the determination of an appropriate forebay length. Equation 8.0 defines the appropriate forebay length for a given settling velocity and hence particle size to be trapped in the forebay.

$$L = \sqrt{\frac{rQ_p}{V_s}} \qquad \textit{Equation } 8.0$$

Where	L	=	forebay length (m)
	r		length-to-width ratio of forebay
	Q_p	=	peak flow rate from pond (cms)
	V_s	=	Settling velocity = 0.0003 m/s

In all instances the forebay should not exceed one-third of the pond surface area. As well, the length-to-width ratio in the forebay should be 2:1.

8.3.10.2 Forebay Length Using Dispersion Calculations

The dispersion refers to the length of fluid required to slow a jet discharge, such as pipe flow. Equation 8.1 provides a simple guideline for the length of the forebay required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity of 0.5 m/s (discharge jet) at the forebay berm.

$$L = \frac{8Q}{dV_f} \qquad \textit{Equation } 8.1$$

Where:

L = forebay length (m)

Q = inlet flow rate (cms)

d = depth of permanent pool in forebay (m)

V_f = desired forebay velocity = 0.5 m/s

8.3.10.3 Forebay Width

The minimum bottom width of the deep zone in the forebay is given by Equation 8.2.

$$Width_{bottom} = \frac{L}{8} \text{..........} Equation\ 8.2$$

The bottom forebay width is calculated using the largest length derived from Equations 8.0 and 8.1.

8.3.10.4 Forebay Depth

The minimum depth of the sediment forebay should be 1.5 m. The recommended minimum depth is 2.0 m.

8.3.10.5 Forebay Length-to-Width Ratio

The total length of the forebay should provide a length to width ratio 2:1 for each inlet. A length to width ratio < 2:1 is undesirable since the storage will not be utilized effectively. In this case, the addition of flow baffles, or other means of lengthening the flow path in the forebay, may be used, subject to approval by the Department of Engineering. When lengthening methods are used, effective length is measured along the flow path.

8.3.10.6 Forebay Berm

An earthen berm should be used to separate the forebay from the rest of the pond. The top of the berm should be submerged slightly, 0.15 m to 0.3 m below the normal water level (NWL). A submerged berm provides a safety benefit to the public (provides a barrier to the public walking along the berm) and allows vegetation to be planted around and along the berm. The berm should be constructed with a solid substrate to facilitate removal of accumulated sediment and debris.

Emergent vegetation should be planted along the berm to promote filtration of water as it passes over (see Section 8.3.10 and **Appendix A** for guidelines and species). The plants should be established on the top and sides of the berm at a maximum planting depth of 30 cm.

Although not required, pipes may be installed in the berm to serve as the primary conveyance system from the forebay to the pond, or as a secondary conveyance system to supplement flows over the submerged berm. Flow calculations should be made to ensure the berm does not create a flow restriction, causing the forebay to overflow under design conditions.

The invert of any conveyance pipes installed in the berm should be set at least 0.6 m above the bottom of the forebay to prevent siphoning of settled material into the rest of the pond. A maintenance pipe/valve should also be installed in the berm to allow drawdown of the forebay during maintenance. If only the forebay is to be pumped out or drawn down during maintenance, the forebay berm must be designed as a small dam since the rest of the pond will not be drained. Care must be taken not to compromise the structural integrity of the berm or liner during drawdown conditions.

8.3.11 Wet Pond Length-to-Width Ratio

The overall performance of the pond is influenced by the flow path through the pond. Problems encountered with other municipality's pond designs include construction of the outlet too close to the inlet, and having multiple inlets at opposing ends of the pond based on servicing convenience. In both cases, short-circuiting reduces the effective volume of the facility. Where possible, all stormwater servicing should be conveyed to one inlet location. To provide the longest flow path though the pond, the inlet should be located as far away from the outlet as possible. A pond with a minimum length to width ratio of 3:1 will generally have an acceptable flow path. The preferred length to width ratio ranges from 4:1 to 5:1. A ratio outside of this range requires the approval of the Department of Engineering. Effective length excludes forebay length.

The provision of additional berms or flow baffles in the pond to redirect flows and lengthen the flow path is also acceptable to ensure short- circuiting will not occur.

8.3.12 Wet Pond Depth

The depths of the permanent and active storage areas are based on a variety of criteria, including potential stratification, the tolerance of plants to fluctuating water levels, and safety.

<u>Permanent Storage Area</u>

The **minimum** depth from the pond bottom to PWL shall be 1.5 m, with the recommended depth being 2.0 m. A **maximum** depth of 3.0 m should not be exceeded. Depths in excess of 3.0 m require approval from the Department of Engineering. Although ponds deeper than 3.0 m may have benefits in terms of temperature, stratification is more likely, resulting in anoxic conditions which release metals and organics from the pond sediments.

<u>Active Storage Area</u>

The **maximum** active storage depth shall be 1.5 m. Depths in excess of 1.5 m require approval from the Department of Engineering. The active storage depth is defined as the depth between PWL and HWL. In addition, a minimum freeboard of 1.0 m is required above HWL.

8.3.13 Hydraulics

The 100 year elevation will be established taking into consideration the adjacent building's footing elevations. When the wet pond is at the 100 year elevation, water should not back up through the storm sewer and weeping tile connections to create hydraulic pressure on foundations. Areas affected by the HWL and resulting hydraulic grade line should be kept to a minimum. Free flow conditions are preferable; this is achieved when the crown of the closest incoming storm sewer(s) is at or above the HWL. All hydraulic conditions must be approved by the Department of Engineering.

When free flow conditions are not achieved based on the HWL, hydraulic grade line (HGL) elevations in the storm sewers must be determined based on the pond at HWL and the appropriate losses taken into account (ie. junction losses, pipe losses, etc.). **Surrounding footing (or slab) elevations must be a minimum of 0.3 m above the HGL.** Other options to protecting weeping tile connections include a separate weeping tile system connected downstream of the pond. Weeping tile connected to sanitary is not permitted in any circumstances.

Surcharging to ground surface will **not** be permitted. Backflow prevention devices are required on all weeping tile connections as per the National Building Code. All upstream storm piping below the HWL and HGL must be rubber gasketed.

8.3.14 Landscaping & Vegetation

Landscaping and vegetation plans must be submitted with the construction drawings. The drawings must be reviewed and approved by the Department of Engineering, Environmental Services Division and the Parks Division. All landscaping must be prepared by a qualified consultant and must conform to the approved plant species noted in **Appendix A**. A holistic planting strategy is required to provide aesthetics, safety, enhanced pollutant removal, waterfowl (unwanted) control, potential recreation amenity, and enhanced biological activity within the pond (the purpose of the planting is to provide a sustainable community with a **naturalization treatment)**. In this light, the ‘wet pond’ is not merely a man-made water storage device, it is a hydrodynamic (vertically fluctuating) biological system that functions most efficiently when all systems work in concert with one another. Obligatory for the creation of a ‘biological system’, is the need for appropriate plant material. For the City of St. John’s, plant species native to the Avalon Peninsula should be used. Planting density is to be determined based on individual site characteristics and to promote natural succession. As well, the overall planting should be designed to minimize maintenance following the establishment period. Manicured and mown areas should be kept to a minimum, as these areas can attract unwanted waterfowl and become a problem.

8.3.14.1 Vegetation Gradients

Vegetation gradients refer to vegetation transitions found along the vertical axis of any sized drainage basin; whether it is regional or site scale. When referring to a wet pond system with characteristic fluctuating water levels, vegetation gradients can be in flux during a prolonged establishment period due in part to a cycle of sediment deposition, re-suspension, and subsequent deposition. The significance of this is the need for consideration when designing vegetation gradients. With most natural wet pond systems, there is constant vegetation gradient fluctuation, typically with ample horizontal space. This is mind, the difference between a true wet pond, and the wet pond systems proposed is that the horizontal space is fixed. Design implications of this include the need for wet pond wall slopes to accommodate not only safety aspects, but as well fixed spatial sediment deposition cycles, and the need to facilitate changing vegetation gradients. The design of variable vegetation gradients necessitates the need to provide adequate overlap of vegetation types.

The City guidelines define five hydrologic/vegetation gradients present within a wet pond: **i)**. Deep Water areas, **ii)**. Shallow Water areas, **iii)**. Shoreline Fringe Areas, **iv)**. Flood Fringe areas, and **v)**. Upland areas. The following describes the conditions present in each and introduces vegetation types applicable for each.

8.3.14.2 Deep Water Areas

The majority of an area in a wet pond is comprised of deep water areas. Plantings in deep water areas are restricted to *aquatic* and *submergent vegetation*. Refer to **Appendix A** for appropriate species used within this gradient. The transition between shallow and deep water plantings will eventually establish itself according to water level fluctuations, sediment deposition cycles, and light availability.

8.3.14.3 Shallow Water Areas

Shallow water areas, the aquatic bench, are considered to be the areas of the permanent pool where the depth is 0.5 meters or less. This is typically defined as the perimeter of the pond. Plantings in shallow water areas include both *submergent* and *emergent vegetation*. *Submergent* plant species should be planted at water depths between 0.3 meters and 0.5 meters. *Emergent* plant species should be planted at water depths at 0.3 meters. The wet pond wall side slopes will determine the amount of vegetation that can be established. The selection of plant species should consider nutrient uptake (for absorbing excess nitrates conveyed with first-flush stormwater), stormwater filtration, safety, and aesthetics. Other benefits of *emergent* vegetation include the prevention of re-suspension of bottom sediments, and the reduction of flow velocities to aid in sedimentation. Refer to **Appendix A** for appropriate species used within this gradient.

8.3.14.4 Shoreline Fringe Areas

Shoreline fringe areas are the areas subject to frequent wetting from storm events. In general, this is the gradient delineated between the PWL and HWL for erosion/water quality control. This area will typically have higher soil moisture conditions during the frequent storm events. The area close to the NWL (normal water level) elevation is subject to more frequent flooding and wave action from the pond. This area must be adequately protected from erosion. Plantings in shoreline fringe areas include both *emergent* and *hydric vegetation*. Due to the frequency of inundation, plant stocks should be used instead of seed. Refer to **Appendix A** for appropriate species used within this gradient.

8.3.14.5 Flood Fringe Areas

When the wet pond is used to control peak flow rates, a zone of infrequent inundation is created. This gradient is referred to as the flood fringe area and is generally the area slightly below or above the HWL (high water level). Plantings in the flood fringe area include transitional *hydric* species and a mix of grass, perennial, and shrub species. In addition, deterrent vegetation (species which discourage access either by species characteristics and/or density) may be planted to provide safety measures as an alternative to fencing. Together with upland plantings, an effective barrier to public entry can be obtained. Refer to **Appendix A** for appropriate species used within this gradient.

8.3.14.6 Upland Areas

Upland areas are landscaped areas above the HWL (high water level) that provide aesthetic and passive recreation amenities around the pond. Plant species should be chosen to restrict access to steep areas or inlet/outlet locations. Refer to **Appendix A** for appropriate species used within this gradient.

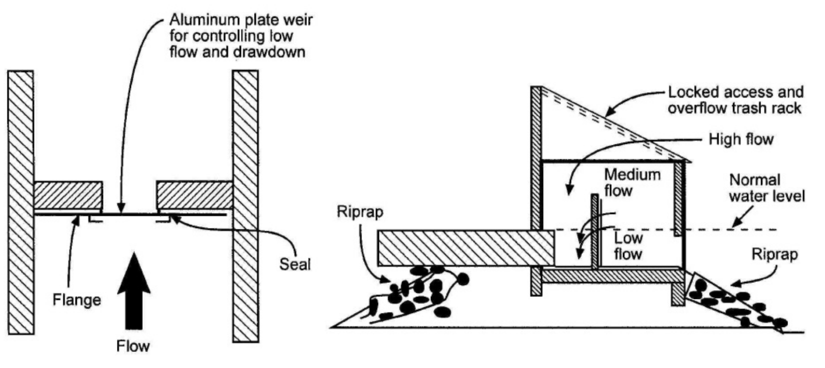
A minimum horizontal buffer strip of 5 meters should be provided between the HWL and the property line, or a 15 meter horizontal buffer strip between NWL and the property line, whichever is greater. Any formal pathways to be incorporated must be constructed above the 100 year elevation (HWL). Pathway locations and design should also consider the protection of any native habitats created or protected. Any deviations require approval of the Department of Engineering and the Parks Division.

8.3.15 Inlet Structure

Inlet areas and inlet structures must be designed to control design velocities. Because of concerns for winter operation, the minimum diameter of inlet pipes must be 450mm with a pipe slope equal to or greater than 1%. The stormwater conveyance system from the development must have one discharge location into the detention pond. Rock lined channels which convey stormwater from the pipe outlet to the pond will not be approved as they promote water temperature increases. A hard-bottomed surface (eg. Interlocking stone) near the outfall for the inlet pipe is required to ensure that erosion and scour of the pond bottom do not occur. A non-submerged inlet pipe is generally preferred over submerged inlets. Where a submerged inlet is required, its obert must be located 150mm below the expected maximum ice depth. The headwalls and wingwalls at inlets should be constructed of natural stone and plant material to blend in better with the natural landscape. Access to the inlet area must be provided to facilitate maintenance and repairs.

8.3.16 Outlet Structure

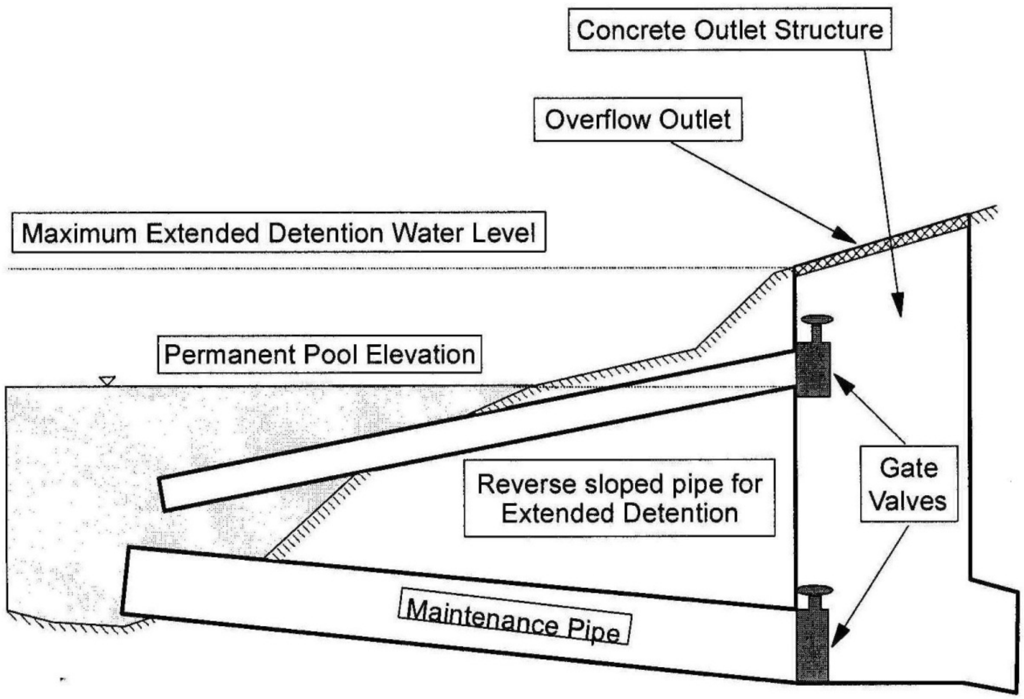
Outlet control structures vary greatly in design and to a certain degree can be site specific. In general, they can be modeled as orifices and/or weirs and the optimum design must meet the flood-control objectives. One possible configuration is indicated below where a manhole structure is built into a dike or berm with low to medium flows controlled by an orifice and weir, and high flows handled "ditch inlet type" opening. A trash rack type screen is mounted over the orifice to minimize the size of objects/obstructions entering the structure and the rack is designed to be raised and lowered for easy maintenance.



Cross section and plan of an outlet

A second alternative for an outlet structure is indicated below using a reverse sloped pipe to control the permanent pool elevation where a maintenance pipe is provided for draining the pond during maintenance. The maintenance pipe should be sized to drain the pond in 6 hours subject to this release rate not impacting on the downstream system.

Reverse Sloped Pipe Outlet Configuration



Submerged outlets must be set 150mm below the expected maximum ice depth. Reversed sloped pipes must have a minimum 300mm diameter. NOTE: perforated riser outlet control pipes are not permitted due to problems with ice and access issues for maintenance. The emergency spillway must be sized to accommodate the largest flow generated from the 100-year design criteria and flows must be conveyed through an appropriately sized conduit to the nearest watercourse.

8.4 DRY PONDS

Dry ponds are impoundment areas used to temporarily store stormwater runoff in order to restrict downstream discharge to predetermined rates, and to reduce downstream flooding and erosion potential. They may be constructed by an embankment or through excavation of a depression. Most dry ponds have no permanent pool of water. As a result, they can be effectively used for quantity control. Generally, dry ponds are not intended as water improvement facilities. Water quality enhancement is required through the use of sediment forebays that include a permanent pool. Dry ponds may be constructed where topographic or planning constraints exist that limit the implementation of wet ponds or wetlands.

8.4.1 Volumetric Sizing

Dry ponds must provide an active storage volume for the 100 year return period rainfall event. As a minimum, the active storage must be sized for a volume equal to the 100-year 24-hour runoff generated by the entire contributing drainage area with full development of the proposed site. All side-slopes must be 4:1 maximum. The 100 year volume must be contained before spillover is permitted. Release rates from the ponds must be restricted to the pre-development flow condition or downstream limiting capacity, whichever is less.

8.4.2 Land Dedication

.1 Dry ponds that are to be ultimately operated by The City of St. John’s are to be located on land which is owned by the City or will be conveyed to the City as a condition of approval for the development.

.2 The maximum level of inundation, the high water level (HWL), must not encroach onto private property. Lots bordering the dry pond are required to have abutting property elevations a minimum of 0.3 m above the spillover elevation of the pond and basement elevations must be 0.3m above the HWL and HGL.

8.4.3 Drainage Area

There is no minimum drainage area requirement.

8.4.4 Low Flow Channel

A low flow channel must be constructed through the pond, suitable vegetation along the channel must be used in accordance with Appendix A.

8.4.5 Winter Operation

Dry ponds are normally the least affected by winter/spring conditions, heowever, precautions should be taken to minimize the effects of freezing of pipes and orifices.

8.4.6 Sediment Forebay

A sediment forebay must be used to facilitate maintenance and allow for the removal of larger sediment particles. Sizing of the forebay is dependent on the inlet configuration. There are several calculations that need to be made to ensure that it is adequately sized. **In all cases, the forebay length should be greater than, or equal to, the larger of the two forebay lengths determined below by Equations 8.0 and 8.1.** Refer to ***<u>Sections 8.3.6.1 through to 8.3.6.5</u>*** for sizing criteria.

8.4.6.1 Forebay Berms

An earthen berm must be used to separate the forebay from the rest of the pond. Since the downstream side of the berm will be dry, the berm should be designed as a small dam. A weir should be designed at the top of the berm to convey flows to the downstream section of the pond during storm events. The forebay should be incorporated as a permanent pool set below the bottom elevation of the dry pond.

To facilitate cleaning of the forebay, a maintenance pipe should be installed in the berm. A valve, to open and close the pipe, should be installed on the upstream side of the pipe. Under normal operating conditions, the valve should be closed. During maintenance periods, the valve should be open to allow draining of the forebay.

Vegetation should be planted on the top of the berm to promote filtration of water as is passes over the berm. Suitable vegetation is listed in Appendix A. The vegetation should be planted on the forebay side of the berm at a depth no greater than 30 cm. As a secondary benefit, the vegetation will also act as a barrier to public access.

8.4.7 Dry Pond Length-to-Width Ratio

For dry ponds with a continuous flow path, all stormwater should be conveyed to one inlet location, if possible. To provide the longest flow path through the pond, the inlet should be located as far away from the outlet as possible. A pond with a length to width ratio greater than, or equal to, 3:1 will have an acceptable flow path. The preferred length to width ratio ranges from 4:1 to 5:1. Effective length excludes forebay length.

8.4.8 Dry Pond Depth

The **maximum** active depth for a dry pond is 1.50 m, measured from the elevation of the pond bottom to the 1:100 year elevation or HWL. In addition, a minimum freeboard of 0.30 m is required above the water level in the pond that corresponds to the design overland emergency discharge rate. The maximum active depth from the PWL in the forebay to the 1:100 year elevation (or HWL) must be 1.50 m. The primary factor in establishing these depth restrictions is concern for the safety of children.

8.4.9 Hydraulics

The 100 year elevation will be established taking into consideration the adjacent building's footing elevations. When the dry pond is at the 100 year elevation, water should not back up through the storm sewer and weeping tile connections to create hydraulic pressure on foundations. Areas affected by the HWL and resulting hydraulic grade line should be kept to a minimum. Free flow conditions are preferable; this is achieved when the crown of the closest incoming storm sewer(s) is at or above the HWL. All hydraulic conditions must be approved by the Department of Engineering.

When free flow conditions are not achieved based on the HWL, hydraulic grade line (HGL) elevations in the storm sewers must be determined based on the pond at HWL and the appropriate losses taken into account (ie. junction losses, pipe losses, etc.). **Surrounding footing (or slab) elevations must be a minimum of 0.3 m above the HGL.** Other options to protecting weeping tile connections include a separate weeping tile system connected downstream of the pond. Weeping tile connected to sanitary is not permitted in any circumstances.

Surcharging to ground surface will **not** be permitted. Backflow prevention devices are required on all weeping tile connections as per the National Building Code. All upstream storm piping below the HWL and HGL must be rubber gasketed.

8.4.10 Landscaping & Vegetation

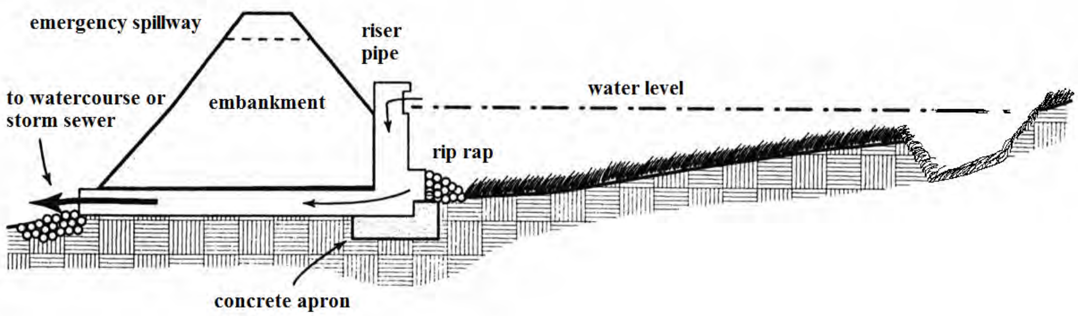
Refer to Section 8.3.14 for landscaping requirements.

8.4.11 Inlet Structure

Refer to Section 8.3.15 for inlet structure criteria.

8.4.12 Outlet Structure

Outlet control structures vary greatly in design and to a certain degree can be site specific. In general, they can be modeled as orifices and/or weirs and the optimum design must meet the flood-control objectives. One possible configuration is indicated below where a riser pipe structure is built into an embankment with low flows controlled by riprap-covered intake, and medium to high flows handled perforated riser pipe opening. Other configuration may be considered and would require the approval of both the Engineering Department and the Public Works Department.



Profile view of a dry pond design.

8.5 DELIVERABLES

The following items must be submitted to the City for review when stormwater detention is required for a particular development (all information must be georeferenced to the City's NAD83 coordinate system):

(a) PDFs of all pre-development and post-development drainage areas used in modeling which denote all proposed infrastructure, existing infrastructure in the immediate area, overall drainage area, subcatchments, watercourses, and contours. Note: infrastructure is defined as streets and driveways, buildings, manholes, catchbasins, ditch inlets, headwalls, bridges, culverts, open channels, etc.

(b) ArcGIS polygon shape files containing the pre-development and post-development drainage areas with subcatchments. There must two fields included in the shape files attribute table containing (i) the area, in hectares, and (ii) the percent impervious for each subcatchment.

(c) An ArcGIS polygon shape file for the proposed buildings.

(d) An ArcGIS polygon shape file for the proposed streets, parking areas and driveways. The area of the street/parking areas must include the curb, gutter, and sidewalk.

(e) An ArcGIS point shapefile containing a 1m elevation grid of the proposed development containing, but not limited to: lot grading elevations, street elevations (centerline, gutter, top of curb, back of sidewalk), side sloping, etc. Note: elevations should not be provided within the footprint of any proposed buildings.

(f) PDFs of all construction drawings including: Plan & Profile drawings for the proposed infrastructure; Detail drawings of the proposed detention facility, outlet control devices, and emergency overflow; and an overall plan indicating the proposed development in its entirety indicating adjacent infrastructure/structures in close proximity.

(g) A fully functioning electronic XPSWMM model, or models, with all associated model files and supporting computations for all scenarios used to size and design the storm sewer infrastructure and detention facility

(h) A summary report in PDF providing: (i) a tabular summary for each post-development scenario of the post-development peak flows into and out of the detention facility, the corresponding pre-development flow, the maximum elevation of the water surface within the facility, and the flow through the emergency overflow; (ii) if nodal storage is used then a table must be included containing the design Elevation(m)-Storage Area(ha) curve for the detention facility; and (iii) the recommended maximum storage detention volume and elevation required for the facility.

APPENDIX A - Recommended Plant Species

The list provided is intended as a guide only. It is not an overly-extensive list of native plants available for use in restoration projects within the St. John's Metro region. The list covers species in all vegetation gradients. These species should be available commercially either locally and/or shipped from propagators on the mainland. If plant species are unable to be sourced on the island during time of construction, pre-emptive plant shipments received approximately one growing season prior to installation may be necessary to enable plant acclimatization. Project developers should consider this so as to avoid any delays for realizing the project desired completion date.

Native species used in any project must be native to the Avalon region and appropriate for the specified vegetation gradient.

Qualified consultants such as a landscape architect, environmental designer, horticulturalist, wetland ecologist, or botanist should be retained to make the determination of suitable plant species for a given project. It is not enough to simply select species from the list provided within this document. A detailed understanding of the site conditions, community context, the ecology of the selected restoration species, as well as a detailed project plan and set of end goal objectives is required. An environmental specialist such as those mentioned above must be retained to select species and assist in the planning and design of the project.

As mentioned, the species listed should be commercially available. Given that the intent of this list is for stormwater wet pond restoration/naturalization, they have been classified in terms of moisture regimes/vegetation gradients. This will serve as a general guide for selecting plant species. However, other considerations mentioned in earlier sections and as well soil type, nutrient regime, aspect, and slope position may be of equal or greater importance.

APPENDIX A

Latin Name	**Common Name**	**Moisture Regime**				**Notes**
		Aquatic	**Emergent**	**Hydric**	**Upland**	
Sedge, Rush, & Broad-leaved Aquatic Species						
Acorus americanus	Sweet Flag		•	•		
Carex aquitilis	Water Sedge	•	•			good filter
Carex arctata	Drooping Wood Sedge		•	•		good filter
Carex crinita	Fringed Sedge	•	•			good filter
Carex hystericina	Porcupine Sedge		•	•		good filter
Carex lacustris	Lake Sedge	•	•			good filter
Carex pseudocyperus	Cyprus-like Sedge	•	•			good filter
Eleocharis palustris	Spiked Rush	•	•			good filter
Juncus articulatus	Jointed Rush	•	•			good filter
Junduc Canadensis	Canada Rush	•	•			good filter
Juncus effusus	Soft Rush	•	•			good filter
Juncus tenuis	Path Rush		•	•		good filter
Nuphar lutea	Yellow Pond Lily	•				
Potamogeton natans	Floating-leaved Pondweed	•				good filter
Potamogeton pectinatus	Sago Pondweed	•				good filter
Saggitaria latifolia	Arrowhead	•	•			
Schoenoplectus acutus	Hard-stemmed Bulrush	•	•			good filter
Schoenoplectus tabernaemontani	Soft-stemmed Bulrush	•	•			good filter
Scirpus sp.	Bulrush	•	•			good filter
Sparganium chlorocarpum	Green-fruited Bur-reed	•				good filter
Sparganium erectum	Common Bur-reed	•				good filter
Sparganium eurycarpum	Giant Bur-reed	•				good filter
Uticularia vulgaris	Bladderwort	•	•			good filter
Latin Name	**Common Name**	**Moisture Regime**				**Notes**

		Aquatic	Emergent	Hydric	Upland	
Forb & Grass Species						
Achillea millefolium	Yarrow				•	
Asclepias syriaca	Common Milkweed			•	•	
Aster nemoralis	Bog Aster		•	•		
Aster puniceus	Purple-stemmed Aster		•	•		
Calmagrostis canadensis	Bluejoint			•		
Caltha palustris	Marsh Marigold		•	•		
Chelone glabra	Swamp Turtlehead		•	•		
Deschamspia caespitosa	Tufted-Hair Grass			•	•	
Epilobium angustifolium	Fireweed				•	
Eriophorum virginicum	Cottongrass		•	•		
Eupatorium maculatum	Joe-Pye Weed			•		
Geum rivale	Avens			•		
Glyceria Canadensis	Rattlesnake Mannagrass			•	•	
Iris versicolor	Blue-flag Iris		•	•		
Lupinus polyphyllus	Common Lupine				•	
Matteuchia struthiopteris	Ostrich Fern			•	•	
Osmunda cinnamonea	Cinnamon Fern			•	•	
Osmunda clintoniana	Interrupted Fern			•	•	
Osmunda regalis	Royal Fern		•	•		
Rudbeckia hirta	Black-Eyed Susan				•	
Sanguisorba Canadensis	Bottlebrush			•		
Schizachyrium scoparium	Little Blue Stem				•	
Solidago canadensis	Canada Goldenrod				•	
Solidago macrophylla	Large-leaved Goldenrod			•	•	
Solidago rugosa	Rough-stemmed Goldenrod			•		

Thalictrum aquilegifolium	Meadow Rue		•	•		
Viola sp.	Violet		•	•		

Latin Name	Common Name	Moisture Regime				Notes
		Aquatic	Emergent	Hydric	Upland	
Tree & Shrub Species						
Abies balsamea	Balsam Fir			•	•	
Acer rubrum	Red Maple			•	•	
Acer spicatum	Mountain Maple			•	•	
Alnus crispa	Green Alder			•	•	
Alnus rugosa	Speckled Alder			•	•	
Amelanchier alnifolia	Saskatoon Serviceberry			•	•	
Amelanchier canadensis	Shadblow Serviceberry			•	•	
Andromeda glaucophylla	Bog Rosemary			•		
Aronia melonocarpa	Black Chokeberry			•	•	
Betula alleghaniensis	Yellow Birch			•	•	
Betula papyrifera	Paper Birch				•	
Betula pumilla	Bog Birch		•	•		
Chamaedaphne calyculata	Leatherleaf			•		
Cornus alternifolia	Pagoda Dogwood			•	•	
Cornus racemosa	Gray Dogwood			•	•	
Cornus sericea	Red-Twigged Dogwood			•	•	
Cornus stolonifera	Red-osier Dogwood			•	•	
Ilex verticillata	Winterberry					Requires male for berries

Juniperus communis	Common Juniper				•	
Juniperus horizontalis	Horizontal Juniper				•	
Kalmia angustifolia	Shepards Laurel			•	•	
Kalmia polifolia	Bog Laurel			•		
Larix larcina	Larch			•	•	
Ledum groendlandicum	Labrador Tea			•		
Myrica gale	Sweetgale		•	•		
Nemopanthus mucronatus	Mountain Holly					
Picea glauca	White Spruce				•	
Picea mariana	Black Spruce		•	•		
Potentilla fruticosa	Shrubby cinquefoil				•	
Prunus pensylvanica	Pin Cherry			•	•	
Prunus virginiana	Choke Cherry			•	•	
Rhododendron canadense	Rhodora			•	•	
Ribes sp.	Currant			•	•	
Salix bebbiana	Beaked Willow		•	•		
Salix discolor	Pussy Willow		•	•	•	
Salix glauca	Smooth Willow		•	•		
Salix lucida	Shining Willow		•	•		
Sambucus racemosa	Red Elderberry				•	
Sorbus americana	American Mountain Ash			•	•	
Spirea latifolia	Meadowsweet		•	•		
Viburnum cassinoides	Northern Wild Raisin				•	
Viburnum trilobum	American Cranberrybush					

Note: research / reference record

ATIPP 8

Paradise Storm Water Management Plan

Final Report

163063.00 ● Final Report ● December 2019

Prepared for:

Town of Paradise

Prepared by:

CBCL LIMITED
Consulting Engineers

Final	G. Sheppard	December 9, 2019	J. Bursey
Draft Final	G. Sheppard	August 4, 2017	C. Flanagan
Draft	G. Sheppard	May 16, 2017	C. Flanagan
Issue or Revision	***Reviewed By:***	***Date***	***Issued By:***

C B C L

CBCL LIMITED

Consulting Engineers

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REGISTERED PROFESSIONAL ENGINEER

PEG Newfoundland and Labrador

JENNIFER BURSEY

Jennifer Bursey

SIGNATURE

Dec 9, 2019

DATE

NEWFOUNDLAND & LABRADOR

Report: 163063.00

CBCL LIMITED
Consulting Engineers

December 9, 2019

Ms. Tracy-Lynn Goosney, P. Eng.
Manager of Engineering Services
Town of Paradise
28 McNamara Drive
Paradise, NL
A1L 0A6

Dear Ms. Goosney:

RE: Storm Water Management Plan Final Report

We are pleased to submit the final report for the above-noted project.

We have enjoyed working on this interesting project and look forward to assisting the Town with the implementation of the capital works improvements.

Yours very truly,

CBCL Limited

Greg Sheppard
Project Manager
Direct: 709-364-8623 Ext. 288
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Project No: 163063.00

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CHAPTER 1 INTRODUCTION

1.1 Background

Located on the northeast Avalon, the Town of Paradise (Town) is one of the fastest growing municipalities in Newfoundland and Labrador. Incorporated in 1992, it comprises the former towns of Paradise and St. Thomas, as well as the developed areas of Three Island Pond, Topsail Pond, Evergreen Village and Elizabeth Park.

Rapid development within the Town has resulted in increased impervious surfaces, and hence greater runoff during precipitation events. In order to assess the capacity of existing bridges and culverts located on rivers within the Town's boundary, Paradise needs well-documented planning tools. The storm water management plan (SWMP) will address this need with respect to bridges and culverts.

Paradise has three primary drainage basins which are illustrated on Map 1 in Appendix A. The major watercourses are Topsail River (Basin A, 1,720 ha), Waterford River (Basin B, 780 ha), and Horse Cove Brook (Basin C, 674 ha). An additional area of 6,876 ha partially contributes to Basin A under hydraulically-controlled conditions (i.e. there is a dam on Thomas Pond which is operated by Newfoundland Power). All three basins contain a mixture of underground storm water infrastructure, and aboveground ditches, culverts and bridges.

In 2013, BAE-Newplan Group Limited (BAE-Newplan) was contracted to complete a Storm Water study for the Town of Paradise. For this study, BAE-Newplan developed HEC-HMS and HEC-RAS models for the rivers, ponds and major control points of the Town. Drainage characteristics for the current (2013) extent of development and for hypothetical future development scenario where the Town would be 100% developed were examined. BAE-Newplan identified a number of undersized culverts and made recommendations to replace these culverts.

As a continuation of the 2013 study, Town staff began to prepare XPSWMM models of Basins A, B, and C. In 2016, the Town contracted CBCL Limited (CBCL) to update the HEC-HMS and XPSWMM models, and to provide upgrading and policy recommendations.

1.2 Scope of Work

The objectives of this study, as outlined in the RFP, are to:

- Review the 2013 Storm Water Study report and models by BAE-Newplan;
- Identify flood prone areas with input from Paradise staff;
- Update the existing HEC-HMS model to reflect changes in the basins since their development in 2013;
- Complete the XPSWMM storm water models started by Paradise staff;
- Identify stream crossings that are currently undersized and determine appropriate size to accommodate expected future flows;
- Compare these recommendations to the structures recommended in the previous Storm Water Study and update as required;
- Provide discussion on the various systems for storm water detention/retention, and identify potential locations in the Town that may benefit from such systems; and
- Prepare a Storm Water Management Plan for the Town which identifies areas of deficiencies, presents solutions to correct the deficiencies, provides cost opinions, an implementation schedule for each solution, and policy recommendations.

A Wetland Functional Assessment along Horse Cove Brook was completed by CBCL in March, 2017 and is provided under a separate cover.

CHAPTER 2 INFORMATION COLLECTION AND REVIEW

CBCL assembled and reviewed all available information in completing this project. Background information was obtained from previous reports, namely the 2013 Storm Water Management Plan. Local data (ie. updated precipitation data) was reviewed for use as model input. Zoning maps were examined to characterize projected development.

In addition, potential areas/locations of concern, information gaps, and issues requiring clarification were identified and discussed during a field visit with the Town in December 2016. During this field visit, CBCL compiled information on potentially blocked outlets, low areas with flooding concerns, possible constraints to storm water flow (ie., a pedestrian bridge), and locations where the existing watershed boundaries required verification. Most of the Town's concerns were in Basin B, although Basins A and C also contain areas of concern. CBCL returned to these areas of concern several times to obtain additional measurements, and to characterize the condition of hydraulic structures.

Furthermore, details were extracted from as-built drawings, were available. CBCL also retained Legge Surveys Ltd. to survey missing information. The survey included locations, elevations, and geometric properties of all underground infrastructure, as well as above-ground flow paths and associated hydraulic structures. The survey was limited to areas within the Town's drainage basins. Subsequent field visits were arranged to address any remaining issues. For instance, additional site investigations were necessary to determine flow paths in St. Anne's Industrial Park, as this is an area of uncertainty due to a lack of as-built drawings.

It is noted that water level and/or flow measurements for calibration were not be obtained. The implications of an uncalibrated model are discussed further in Chapter 5.

CHAPTER 3 2013 STORM WATER MASTER PLAN REVIEW – HEC-HMS AND HEC-RAS

CBCL reviewed and updated the HEC-HMS model established for the 2013 Paradise Storm Water Master Plan by BAE-Newplan.

3.1 Field Information

Two types of improved field information were used to update, and refine, the 2013 Paradise Storm Water Master Plan model:

1. **LiDAR**: Basin areas, slopes and longest flow paths were estimated using LiDAR, rather than 1:2,500 scale mapping contours. CBCL used GIS and the geospatial extension HEC-geoHMS to extract this data from the LiDAR.
2. **Topographic Survey and Survey of Hydraulic Structures**: Surveys of underground and aboveground storm water infrastructure were obtained to be used as input into the models.

3.2 Hydrologic Modelling

The hydrologic (HEC-HMS) components of the 2013 Paradise Storm Water Master Plan which were updated include:

- Curve numbers (CN): Edited such that existing conditions reflect developments as of 2016, and future development conditions reflect changes to zoning map, updated sub-basin delineations and the use of soil maps;
- Basin lag times: Edited to reflect any changes to the slope, curve number, and length of maximum flow path, resulting from the use of LiDAR and updated CN;
- Elevation-area curves for natural storage in lakes: Edited using contours created from the LiDAR, rather than the 1:2,500 scale mapping contours;
- Design storms: Edited to incorporate all historical data; See Section 4.2.1; and
- Average slope: Edited based on LiDAR, and weighted according to flow accumulation.

3.2.1 Curve Numbers

From the review of the BAE-Newplan HEC-HMS model inputs, it appears the CN values used corresponded to antecedent runoff conditions (ARC) type II, and soil type B. ARC II generally correspond to average moisture conditions of the soil.

CBCL updated the CN values by following the Department of Municipal Affairs and Environment's *Technical Document For Flood Risk Mapping Studies*, which suggests an ARC type III be used. ARC type III corresponds to soil with a high moisture content, generally due to heavy rainfall over the preceding days.

In addition, the updated model CN values are based on the particular hydrologic soil group (A, B, C, D, or a combination) as determined from the soil map obtained from the National Soil Data Base (available through Agriculture and Agri-Foods Canada).

3.2.2 Design Storms

The 2013 HEC-HMS model used the City of St. John's Subdivision Design Manual rainfall hyetographs for the 1:100 AEP for 0.5, 1, 2, 6, 12 and 24 hour durations. The effects of climate change were not considered.

In 2015, the Provincial Office of Climate Change and Energy Efficiency issued a report titled *Intensity-Duration-Frequency Curve Update for Newfoundland and Labrador*. This report contains updated intensity-duration-frequency (IDF) curves for various locations throughout the province. Updated IDF curves for two stations on the northeast Avalon were created: St. John's A and Ruby Line stations. In addition to producing up-to-date IDF curves, climate change projections for these stations were also generated. The IDF curve for the Ruby Line station was used for this study, as it is closest to the drainage basins. Hyetographs representing the 1:100 AEP current climate and climate change precipitation were created using the Alternating Block Method, and are presented in Section 4.2.1. These hyetographs were simulated in the updated HEC-HMS models, and the XPSWMM models.

For comparison purposes, the updated HEC-HMS models were also simulated with the City of St. John's Subdivision Design Manual hyetographs, from the 2013 study.

3.3 Results

Updated results were calculated for 1:100 AEP historical rainfall data and are presented in Table 3-1. Climate change data was not included in this analysis so as to better compare with the 2013 HEC-HMS models, which did not include any future rainfall projections.

A detailed summary of the HEC-HMS models and maps can be found in Appendix B. Appendix B also contains figures from the 2013 report showing the locations of results for each basin.

Table 3-1 – HEC-HMS Peak Outflow Comparison (Historical Data)

Drainage Basin Outlet	BAE-Newplan 2013 City of St. John's Design Storms (m^3/s)	CBCL Update City of St. John's Design Storms (m^3/s)	CBCL Update Ruby Line IDF Hyetograph (m^3/s)
A	18.6 (2-hr storm)	46.5 (2-hr storm)	51.9
B	14.0 (12-hr storm)	43.2 (2-hr storm)	51.7
C	11.4 (6-hr storm)	42.0 (6-hr storm)	46.3

The updated model results were found to be significantly larger than the 2013 results (Table 3-1). This can be attributed to the different curve numbers used. It has been found in past flood risk mapping studies conducted by CBCL that the hydrologic model is most sensitive to CN values. For example, for the Waterford River Flood Risk Mapping Study (WRFRMS) completed by CBCL in 2018, a sensitivity analysis was performed, which involved altering the CN by ±10%, ±20% and ±30%. For an increase in CN of 10% the 1:100 AEP peak flow at the location of interest increased by approximately 46%. A decrease in CN of 10% reduced the 1:100 AEP flow by approximately 36%. This sensitivity analysis illustrates the importance of calibrating the hydrologic model. For the WRFRMS calibration data in the form of precipitation data recorded at the Ruby Line station by the City of St. John's during Hurricane Gabrielle (September 2001), and corresponding flow data at Environment Canada's hydrometric gauge Waterford River at Kilbride (02ZM008) were available. The calibration of the WRFRMS hydrologic model was accomplished by decreasing the CN (originally modelled as ARC Type III) values by approximately 29%.

As a check, a reduction in CN of 29% was applied to the updated HEC-HMS model for Basin A. Correspondingly, the time of concentration for each subbasin was also adjusted to reflect the change to CN. By reducing the CN estimates (based on ARC Type III) by 29% the resulting peak flow at the outlet of Basin A was reduced to 21.8 m^3/s compared to 51.9 m^3/s with ARC Type III CN values. It is also noteworthy that if the updated Basin A HEC-HMS model was created using CN ARC Type II values, then the peak flow at the outlet of Basin A is 33.3 m^3/s. This comparison is summarized in the table below.

Table 3-2 - Comparison of 1:100 AEP Flow at Basin A Outlet for Different CN Values

Drainage Basin Outlet	1:100 AEP (m^3/s)*		
	CN = ARC III	CN = 0.71 * ARC III	CN = ARC II
A	51.9	21.8	33.3

* Using the 1:100 AEP Alternating Block Hyetograph created from the Ruby Line IDF

CHAPTER 4 HYDROLOGIC MODELLING

4.1 Overview

To further develop the Town's existing XPSWMM model, each basin required the input of catchments, stream crossings, rivers and underground infrastructure. Drainage areas were delineated to the extents of each watershed, including some areas outside of the Town boundary (Map 1, Appendix A). The modelling in XPSWMM is comprised of a hydrologic component (described in Chapter 4) and a hydraulic component (described in Chapter 5).

The hydrologic modelling component calculates runoff hydrographs by routing design storms through each drainage basin as overland flow (EPA Runoff Method in XPSWMM). The 1:10 AEP design storm was used for the analysis of the local storm sewers, and the 1:100 AEP design storm was used for trunk sewers, bridges and culverts on major tributaries. The Green-Ampt method was use to model infiltration. Existing land use was obtained from aerial imagery and future changes in development were determined from zoning maps.

4.2 Model Development

The following sections describe the design storms and basin characteristics used in the hydrologic XPSWMM model.

4.2.1 Design Storms

The hydrologic model requires design storms in the form of rainfall hyetographs (time-series precipitation data) created from intensity-duration-frequency (IDF) curves. The City of St. John's Subdivision Design Manual IDF curves were last updated in 2002, and these were the design storms used in the 2013 Paradise Storm Water Master Plan.

For this project, CBCL used the most up-to-date rainfall data available. Synthetic rainfall hyetographs were created using the Ruby Line IDF curve and the alternating block method, which involves combining storms of various durations into a single rainfall event. In accordance with the Town's Engineering Guidelines for Subdivisions, local storm sewers and ditches were analyzed using the 1:10 AEP rain event, while trunk storm sewers, and river crossing structures were analyzed using the 1:100 AEP rain event. The 1:100 AEP hyetograph based on historical data is shown in Figure 4-1.

The effect of climate change was also considered in the hydrologic analysis. The *Intensity-Duration-Frequency Curve Update for Newfoundland and Labrador* report includes projected IDF data for a 2011-2040 timeframe. Design hyetographs representing climate change conditions were created and simulated in the XPSWMM model. The climate change hyetograph is depicted in Figure 4-2.

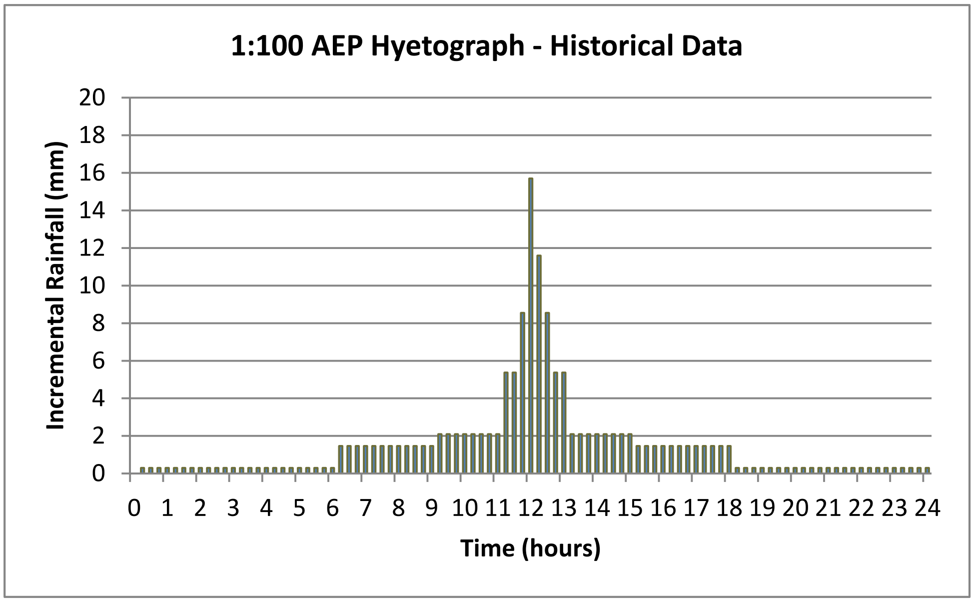


Figure 4-1 – Historical Design Storm

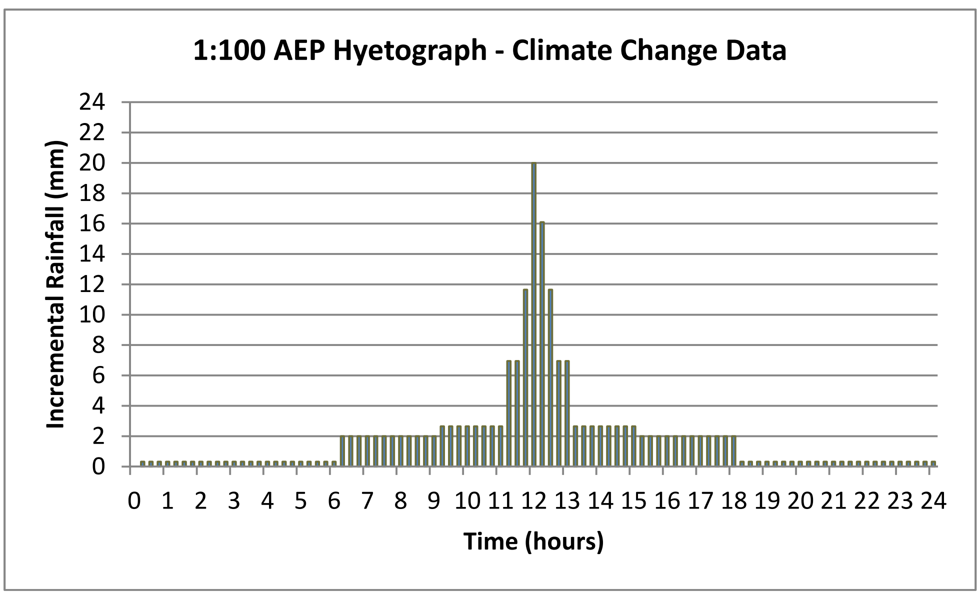


Figure 4-2 – Climate Change Design Storm

Throughout this report, the acronyms CC and CLC are used for the Current Climate and Climate Change scenarios, respectively.

4.2.2 Basin Characteristics

CBCL Limited delineated drainage basins at a higher resolution than the previous study, through the use of LiDAR, the topographic survey, and field verification of flow paths. Updated subcatchment delineation maps are included in Appendix A.

The hydrologic behaviour of each basin is related to soil infiltration and land use characteristics. Infiltration losses are calculated in XPSWMM using the Green-Ampt infiltration method, applying soil parameters described in the City of St. John's Subdivision Design Manual. Values of hydraulic conductivity were based on a soil survey report titled *Soils of the Avalon Peninsula, Newfoundland* from Agriculture and Agri-Food Canada. The existing land use varies between the three basins as follows:

- Basin A (8,596 ha) includes Topsail River and its tributaries between Neil's Pond and Topsail Beach. This basin contains several developed areas, including the neighbourhoods around Octagon Pond and Adams Pond, Grand Meadows subdivision, and Valley Ridge subdivision. There are low-density residential areas near Three Island Pond and Topsail Pond. It is noted that approximately 6,876 ha South of the Town boundary contributes to flow in Basin A as regulated flow. This area includes Three Arm Pond, Paddy's Pond, Cochrane Pond, and Thomas Pond.

- Basin B (780 ha) drains to the Waterford River, which ultimately discharges into St. John's Harbour. This is the most developed of the three basins, and includes three major residential subdivisions: Elizabeth Park, Trails End Drive, and Karwood Estates. This basin also includes several industrial areas, including St. Anne's Industrial Park, part of Donovan's Industrial park (located in the City of Mount Pearl), and Kenmount Road Extension Industrial Park. Maps 4 and 5 illustrate Basin B.

- Basin C (673 ha) is the least developed of the three basins. It includes much of the area surrounding St. Thomas Line from Paradise Road to the ocean at Horse Cove. Only a small portion of Basin C has underground storm sewers: the Atlantica Drive development and Seascape Drive subdivision. The remaining area is either low-density residential development with road side ditches, or undeveloped. Most of Basin C consists of the Horse Cove Brook watershed, while a smaller portion (178 ha) discharges directly into the ocean. Maps 9 and 10 of Appendix A illustrate the Basin C and the hydraulic structure locations.

Area-weighted averages of watershed parameters are shown in Table 4.1. Parameters for each sub-basin are presented in Appendix C.

Table 4-1 – Average Watershed Characteristics

	Total Area (ha)	Average Percent Impervious (%)	Average Width (m)	Average Slope (%)
Basin A	8,596	27	1,444	2.6
Basin B	780	45	179	4.3
Basin C	673	14	139	8.3

Throughout this report, the abbreviations CD and FD are used for the Current Development and Future Development conditions, respectively. For FD conditions, it was assumed that a zero net increase in runoff policy will be implemented by the Town.

4.2.3 Calibration

Generally, a hydrologic model is calibrated using observed detailed precipitation and flow data for a large rain event for the study area. Simulating a design storm in a calibrated hydrologic model gives the modeller greater confidence in the resulting flow. To calibrate a hydrologic model detailed rainfall data is simulated in the developed model, and the resulting hydrograph is compared to a hydrograph observed during the rainfall event. Then, edits are made to the model parameters (within reasonable ranges) to attempt to produce a modelled hydrograph that is similar to the observed hydrograph. For this study, detailed precipitation data is available from the Ruby Line rain gauge. However, there are no existing active hydrometric gauges located within the Town to obtain a measured hydrograph, therefore traditional model calibration is not possible.

4.3 Results

The hydrologic mode in the XPSWMM model is used to calculate the total storm water runoff accumulation at various collection points throughout the Town. The 1:100 AEP rain events for both current climate and climate change conditions were simulated in the hydrologic model, the peak flows at the outlet of each basin, as determined through the uncalibrated XPSWMM model, are presented in Table 4-2. Each Basin was analysed for three distinct scenarios:

1. Current Development and Current Climate (CD + CC);
2. Future Development and Climate Change (FD + CLC);
3. Future Development with Proposed Improvements and Climate Change (FD + PI + CLC);

For Scenarios 2 and 3 future development conditions were modeled assuming the zero net increase in runoff policy is in place. Therefore, for the purposes of this study future development was modeled the same as current development.

A zero net increase in runoff policy requires that runoff from undeveloped land remains at pre-development runoff rates. To achieve this, some means of maintaining pre-development runoff rates, such as a detention pond or an underground storage system, is required for new developments.

The decision to implement a zero net increase in runoff policy should be based on whether or not such a policy benefits the majority of residents. As most of the developable land in Paradise is located in the upper areas of watersheds, a zero net policy would benefit properties located downstream from these areas as well as residents using major roads such as Topsail Road and St. Thomas Line.

It would be informative to demonstrate that a zero net policy is appropriate for Paradise by comparing the required flood protection upgrades under two scenarios: one with zero net runoff and one without. However, given the uncertainty associated with climate change, cost estimates of flood protection infrastructure would likely be inaccurate and not useful in supporting a decision to move forward with such a policy. In other words, it is challenging, if not impossible, to accurately quantify the effects of climate change.

To this end, we have assumed that Paradise will implement a zero net policy because it is the most equitable way to ensure that current and future residents and property owners share the responsibility of dealing with climate change. Land development will cost more; however, these costs will be passed on to future residents (that is, those who buy developed land) while existing residents are protected to the best extent possible.

It is also noteworthy that Basin B, which discharges to the Waterford River, flows through the cities of Mount Pearl and St. John's, both of which have zero net increase in runoff policies.

Without a zero net increase in runoff policy, peak flows for Scenarios 2 and 3 will increase, this may result in larger structure sizes required to pass the design flows for Scenario 3. Unfortunately, this study did not include floodplain mapping. Therefore, a comparison of flooding extents with and without a detention policy cannot be made.

Table 4-2 - XPSWMM Model Results at Basin Outlets

Outlet Location	Peak Flow (m^3/s)		
	Scenario 1: CD + CC	**Scenario 2: FD + CLC**	**Scenario 3: FD + PI + CLC**
Basin A – Topsail River	26.0	35.9	37.4
Basin B – Waterford River at Kenmount Rd	27.1	35.0	35.0
Basin C – Horse Cove Brook	21.9	32.1	33.8

As a check, the peak flows simulated at the outlet of Basin B for Scenarios 1 and 2 were compared to the flows simulated at that location in the Waterford River Flood Risk Mapping Study. This comparison is presented in Table 4-3. As illustrated, the flows from the uncalibrated XPSWMM model compare well to WRFRMS flows for similar conditions.

Table 4-3 - Basin B Flow Comparison to Waterford River Flood Risk Mapping Study

Flow Scenario	Peak Flow (m^3/s)	
	Current XPSWMM Model (Uncalibrated)	**Waterford River Flood Risk Mapping Study (Calibrated)**
Scenario 1: CD + CC	27.1	24.2
Scenario 2: FD + CLC	35.0	34.3*

* Flow corresponding to CD + CLC conditions in WRFRMS

CHAPTER 5 HYDRAULIC MODELLING

5.1 Overview

The hydraulic modelling component simulates the storage and transport of storm water through the Town's drainage system. Hydraulic modelling was conducted using a 1D XPSWMM model with the runoff routing method.

The hydraulic model includes the main tributaries, the structures on those main tributaries, and underground infrastructure. Most driveway culverts and roadside ditches are not included. The model geometry was developed using surveyed river cross sections, as-built drawings, surveyed infrastructure, and LiDAR data provided by the Town.

5.2 Model Development

The following sections detail the hydraulic model inputs as well as a discussion on model calibration.

5.2.1 Streams and Underground Storm Sewer Pipes

Basin A contains several developed areas with underground storm sewers, as well as low-density residential areas and undeveloped areas which do not have an underground system.

The three major residential subdivisions in Basin B (Elizabeth Park, Trails End Drive, and Karwood Estates) have underground storm sewers.

In Basin C, storm water is mostly conveyed through the streams and wetlands of Horse Cove Brook; only a small portion of Basin C has underground storm sewers. The remaining area is either low-density residential development, with roadside ditches, or undeveloped.

In the XPSWMM model, Paradise's drainage system (both open channels and pipes) is simulated using links, with parameters such as roughness, slope, and length used control the flow through the system. Open channels are modeled with extended cross-sections, cut from the LiDAR data, to include overbanks and floodplains.

5.2.2 Pond Storage

Ponds are modeled in XPSWMM as nodes with defined depth-area curves. Pond outlet rating curves are modeled either as natural channels, control structures in the form of weirs or conduits and/or both. For ponds that are part of the Topsail Pond generation development, Newfoundland Power was contacted for operating rules and structure drawings.

Several ponds outside Town boundaries contribute to the drainage system, including Three Arm Pond, Paddy's Pond, Cochrane Pond, and Thomas Pond. Each of these is accounted for in the hydraulic model for Basin A.

Required node characteristics include spill crest and invert elevations, ponding allowance, and outfall modes.

The potential for increasing storage volume in existing ponds was examined. However, this option was deemed unfeasible due to various restrictions within 1 m of each pond's normal water level. A summary of each pond in the Town and its nearest restriction is shown below in Table 5.1. Only the nearest restriction is included in the table and there are several other limitations present.

Table 5-1 – Pond Storage Limitations

Location	Normal Surface Elevation (m)	Scenario 2 Surface Elevation (m)	Nearest Restriction	Restriction Elevation (m)	Scenario 2 Difference (m)	Permissible Increase
Topsail Pond	109.4	110.1	Three Island Pond Rd Centerline	110.1	0.0	0
Three Island Pond	117.2	117.9	Private Building Outside Ground	118.0	0.1	0
Topsail Round Pond	117.4	118.5	Private Building Outside Ground	118.2	-0.3	0
Pond West of Three Island Pond	121.9	122.2	New Road Centerline	122.8	0.6	0
Octagon Pond	149.0	149.5	Private Building Outside Ground	150.0	0.5	0
Rocky Pond	152.0	152.5	Powerline Pole Base	152.8	0.3	0
Adams Pond	133.2	134.9	Trail at Outlet	134.5	-0.4	0
Neil's Pond	164.8	165.3	Commercial Property Line	165.5	0.2	0
Neville's Pond	153.1	153.5	Shelby Rd Centerline	153.9	0.4	0
Bremigen's Pond	168.4	168.9	Top of Dam	169.3	0.4	0

5.2.3 Calibration

Calibration of the hydraulic model gives the modeller greater confidence in the resulting water levels. Generally, a hydraulic model is calibrated by simulating a known flow hydrograph and comparing the modeled water levels to measured water levels. The hydraulic parameters can then be altered (within reasonable ranges) such that the modeled water levels match the observed water levels as closely as possible. For this study, model calibration was not possible because there are no existing water level gauges within the Town.

As a check of the XPSWMM model, the precipitation recorded at the Ruby Line rain gauge during Hurricane Igor was simulated in the XPSWMM model. Hurricane Igor made landfall in September 2010 and resulted in significant flooding across Newfoundland. The Town provided CBCL with some photographs of flow conditions and flooding observed during Hurricane Igor, and the simulated water levels were compared to the photographs. One of the photos sent by the Town was of Topsail River (Basin A), near 1960 Topsail Road, and is presented in Figure 5-1. As can be seen in this photo, although there is a high flow, it does not appear that the flow breached the river banks and overtopped Topsail Road. However, the uncalibrated XPSWMM model indicates that Topsail Road would overtop during Hurricane Igor. This comparison indicates that the XPSWMM model is likely overestimating the flooding extents, at least in some places, for the design storms and illustrates the importance of calibrating the model.

Figure 5-1 - Topsail River near 1960 Topsail Road During Hurricane Igor

5.3 Results

As described in Section 4.3, each basin was analysed for three distinct Scenarios:

1. Current Development and Current Climate (CD + CC);
2. Future Development and Climate Change (FD + CLC);
3. Future Development with Proposed Improvements and Climate Change (FD + PI + CLC);

Peak flows and maximum storm water depths were examined throughout the Town. The capacities of all hydraulic structures on open channels were checked.

Scenarios 1 and 2 were used to identify undersized structures and areas prone to flooding. Scenario 3 was used to develop capital works improvements, such as increasing the sizes of culverts. The proposed improvements are described in more detail in Chapter 8. Appendix D contains a summary of results for all hydraulic structures for each of the three scenarios.

CHAPTER 6 FLOOD RISK MAPPING

Flood maps were created for the Horse Cove Brook watershed. This watershed has an area of 705 ha, and comprises 75% of Basin C. Floodlines were delineated for Scenario 2 (FD + CLC) and Scenario 3 (FD + PI + CLC). Maps 11 and 12 of Appendix A show the flooding extents for these two scenarios.

To produce the flood maps, two digital surfaces were created for each scenario; one representing the depth of storm water accumulation throughout the watershed, and one for the existing ground surface. Flood extent lines were created by intersecting these two surfaces and calculating the elevation difference at each point.

Map 12, Appendix A shows the reduced extents of flooding after the implementation of Basin C's proposed improvements. Regardless of removing hydraulic restrictions from the system, several houses are still located within the Horse Cove Brook flood zone. These locations are considered to be within the natural floodplain of the brook, and will remain at risk of flooding unless the surrounding land is raised, or other flood mitigation measures (such as berms) are implemented.

It is noted that the presence of wetlands in Basin C provides integrated wetland storage. In 2017, CBCL conducted a functional assessment of Horse Cove Brook. The field study revealed that the two delineated wetlands, labelled in the *Horse Cove Brook Wetland Functional Assessment* (CBCL, 2017) report as WL-1 and WL-2, have experienced significant sedimentation, most likely resulting from a lack of sediment and erosion control measures. This disturbance is affecting the natural storage capabilities of the wetlands. It may be worth taking measures to restore or protect the wetlands to maximize their natural water storage potential.

CHAPTER 7 REVIEW OF STORM WATER MANAGEMENT POLICIES AND BEST PRACTICES

One the most efficient ways to deal with flood risks is to manage high runoff at its source, through Low Impact Development (LID) and Best Management Practices (BMPs). This chapter first describes the planning approach for implementing LIDs and BMPs, and then presents several examples of LIDs and BMPs that can be used to achieve this.

7.1 Land Use Planning Policy

Thoughtful site planning begins with the identification of critical site features such as wetlands, habitat areas, and/or drinking water protection areas that should be set aside as protected open space. Natural features, such as vegetated buffers and view sheds, will also play an integral role in any LID planning exercise. After the critical open space areas are identified and set aside, sustainable development areas are then identified as "building envelopes". General goals include the following:

- **Concentrate Development and Mix Uses**: The LID site planning process sets aside key natural features and focuses development into clustered patterns on the remaining land. The LID planning process results in housing that makes more efficient use of land and conserves critical natural features such as wetlands, vegetated buffers, and drinking water protection areas; and
- **Protect Land and Ecosystems**: The reduction of impervious surfaces reduces the amount of surface runoff, and through the infiltration of storm water, recharges the groundwater system, thereby restoring the natural hydrologic cycle. This preserves groundwater supplies and base flow to streams and wetlands.

The Credit Valley Conservation, a community conservation area in Ontario, has published a comprehensive guide to implementing sustainable development through Low Impact Development and Storm Water Best Management Practices (“Low Impact Development Storm Water Management Planning and Design Guide”, Version 1.0 – 2011). This document could serve as an example for creating a similar document for the Town of Paradise. An extract is presented below, which tabulates the summary of storm water management and land use planning steps in order to achieve low impact development.

Table 7-1 – Summary of Storm Water Management at Key Scales and Land Use Planning Stages

Scale	Planning Stage	Description
Watershed plans	Master Plans Growth Plan Official Plan	Major themes and objectives for the municipality's future growth are established, and challenges and opportunities for growth are identified, such as municipal policy direction for innovative SWM approaches and other climate change initiatives.
Community/ Subwatershed	Secondary Plan	Major elements of the natural heritage system are identified including terrestrial, aquatic and water resources (hydrology, hydrogeology, fluvial geomorphology, etc.). Stormwater management objectives for surface and groundwater resources. Future drainage boundaries, locations of stormwater management facilities and watercourse realignments are established.
	Block Plan	The location of lots, roads, parks and open space blocks, natural heritage features and buffers, and stormwater management facilities are defined. A full range of opportunities to achieve stormwater management objectives are identified, establishing a template for the more detailed resolution of the design of stormwater management facilities at subsequent stages in the planning and design process.
Neighbourhood	Draft Plan of Subdivision/ Functional Servicing Plan	Conceptual design is carried out for stormwater management facilities. Consideration must be given to how stormwater management objectives can be achieved and how these objectives influence the location and configuration of each of the components listed above
	Registered Plan	Detailed design is carried out for stormwater management facilities.
Site	Site Plan	Site-specific opportunities are identified to integrate stormwater management facilities into all of the components of a development including landscaped areas, parking lots, roof tops and subsurface infrastructure. Solutions must be considered in the context of the overall stormwater management strategy for the block or secondary plan area to ensure that functional requirements are achieved
Site	CA Permits and other approvals	Detailed design of SWM for the site

At the implementation (regulatory) level, specific BMPs could be incorporated into language in the municipal regulations (ie. requirements for a certain width of stream buffer or how the buffer would be calculated). However, since every site and every development is different, it is recommended that the policies and by-laws produced be generic enough to allow flexibility of method employed, while focusing on the ultimate objective of controlling runoff volumes and runoff water quality.

7.2 Best Practices for Storm Water Management

This section will focus on typical BMPs at the site level that are most suited to urban areas. The following descriptions reference the United States Environmental Protection Agency (USEPA) *National Menu of Storm Water Best Management Practices*, which is very comprehensive and constantly updated (it contains close to 150 fact sheets on individual approaches, from education to implementation).

7.2.1 Best Practices Based on Density of Development and New Versus Retrofit Application

Table 7-2 below shows the various approaches that are recommended depending on whether a site is located within a high density or low density development, as well as whether the project involves a new development or a retrofit application.

Table 7-2 – Recommendations Based on Density of Development and New Vs Retrofit Application

Recommendations for Storm Water Management in Low-Density Urban Areas	
New Development	**Retrofit Applications**
• Grassed swales; • Infiltration trenches; • Permeable pavement; • Riparian buffers; • Sand and organic filters; • Soil amendments; and • Vegetated filter strips.	• Curb and gutter elimination; • Permeable pavement; • Sand and organic filters; • Soil amendments; • Vegetated filter strips; and • Rain barrels and cisterns.
Recommendations for Storm Water Management in High-Density Urban Areas	
New Developments	**Retrofit Applications**
• Bioretention cells; • Green parking design; • Infiltration trenches; • Inlet protection devices; • Permeable pavement; • Permeable pavers; • Rain barrels and cisterns; • Sand and organic filters; • Soil amendments; • Storm water planters; • Tree box filters; • Vegetated filter strips; and • Vegetated roofs.	• Inlet protection devices; • Permeable pavement; • Permeable pavers; • Rain barrels and cisterns; • Sand and organic filters; • Soil amendments; • Storm water planters; and • Tree box filter.

Each of these measures is described in detail below.

7.2.2 Description of Each Recommended Storm Water BMP Measure

Grassed Swales

Grassed swales are shallow grass-covered hydraulic conveyance channels that help to slow runoff and facilitate infiltration. The suitability of grassed swales depends on land use, soil type, imperviousness of the contributing watershed, and dimensions and slope of the grassed swale system. In general, grassed swales can be used to manage runoff from drainage areas that are less than 4 hectares (10 acres) in size, with slopes no greater than 5 percent.

Use of natural, low-lying areas is encouraged and natural drainage courses should be preserved and utilized.

Infiltration Trenches

Infiltration trenches are rock-filled ditches with no outlets. These trenches collect runoff during a storm event and release it into the soil by infiltration (the process through which storm water runoff penetrates into soil from the ground surface). Infiltration trenches may be used in conjunction with another storm water management device, such as a grassed swale, to provide both water quality control and peak flow attenuation. Runoff that contains high levels of sediments or hydrocarbons (for example, oil and grease) that may clog the trench are often pretreated with other techniques such as water quality inlets (a series of chambers that promote sedimentation of coarse materials and separation of free oil from storm water), inlet protection devices, grassed swales, and vegetated filter strips.

Permeable Pavement

As an alternative to asphalt or concrete surfaces, permeable pavement allows storm water to drain through the porous surface to a stone reservoir underneath. The reservoir temporarily stores surface runoff and allows it to infiltrate into the subsoil. The appearance of the alternative surface is often similar to asphalt or concrete, but it is manufactured without fine materials and instead incorporates void spaces that allow for storage and infiltration. Underdrains may also be used below the stone reservoir if soil conditions are not conducive to complete infiltration of runoff.

Riparian Buffers

A riparian, or forested, buffer is an area along a shoreline, wetland, or stream where development is restricted or prohibited. The primary function of aquatic buffers is to physically protect and separate a stream, lake, or wetland from future disturbance or encroachment. If properly designed, a buffer can provide storm water management and can act as a right-of-way or floodplain during floods, sustaining the integrity of stream ecosystems and habitats.

Sand and Organic Filters

Sand and organic filters direct storm water runoff through a sand bed to remove floatables, particulate metals, and pollutants. Sand and organic filters provide water quality treatment, reducing sediment, biochemical oxygen demand (BOD), and fecal coliform bacteria, although dissolved metal and nutrient removal through sand filters is often low. Sand and organic filters are typically used as a component of a treatment train to remove pollution from storm water before discharge to receiving waters, to groundwater, or for collection and reuse. Variations on the traditional surface sand filter (such as the underground sand filter, perimeter sand filter, organic media filter, and multi-chamber treatment train) can be made to fit sand filters into more challenging design sites or to improve pollutant removal.

Soil Amendments

Soil amendments increase the soil's infiltration capacity and help reduce runoff from the site. They have the added benefit of changing physical, chemical, and biological characteristics so that the soils become more effective at maintaining water quality. Soil amendments, which include both soil conditioners and fertilizers, make the soil more suitable for the growth of plants and increase water retention capabilities. The use of soil amendments is conditional on their compatibility with existing vegetation, particularly native plants.

Vegetated Filter Strips

Filter strips are bands of dense vegetation planted downstream of a runoff source. The use of natural or engineered filter strips is limited to gently sloping areas where vegetative cover can be established and channelized flow is not likely to develop. Filter strips are well suited for treating runoff from roads and highways, roof downspouts, very small parking lots, and other small or linear impervious surfaces. They are also ideal components for the fringe of a stream buffer, or as pretreatment for a structural practice.

Curb and Gutter Elimination

Curbs and gutters transport flow as quickly as possible to a storm water drain without allowing for infiltration or pollutant removal. Eliminating curbs and gutters can increase sheet flow and reduce runoff volumes. Sheet flow, the form runoff takes when it is uniformly dispersed across a surface, can be established and maintained in an area that does not naturally concentrate flow, such as parking lots. Maintaining sheet flow by eliminating curbs and gutters and directing runoff into vegetated swales or bioretention basins helps to prevent erosion and more closely replicate predevelopment hydraulic conditions. A level spreader, which is an outlet designed to convert concentrated runoff to sheet flow and disperse it uniformly across a slope, may also be incorporated to prevent erosion.

Bioretention Cells / Rain Gardens

A bioretention cell or rain garden is a depressed area with porous backfill under a vegetated surface. These areas often have an underdrain to encourage filtration and infiltration, especially in clayey soils. Bioretention cells provide groundwater recharge, pollutant removal, and runoff detention. Bioretention cells are an effective solution in parking lots or urban areas where green space is limited.

Green Parking Design

Green parking refers to several techniques that, applied together, reduce the contribution of parking lots to total impervious cover. Green parking lot techniques include: setting maximums for the number of parking lots created; minimizing the dimensions of parking lot spaces; utilizing alternative / porous pavers in overflow parking areas; using bioretention areas to treat storm water; encouraging shared parking; and providing economic incentives for structured parking.

Rain Barrels and Cisterns

Rain barrels and cisterns harvest rainwater for reuse. Rain barrels are placed outside a building at roof downspouts to store rooftop runoff for later reuse in lawn and garden watering. Cisterns store rainwater in significantly larger volumes in manufactured tanks or underground storage areas. Rainwater collected in cisterns may also be used in non-potable water applications such as toilet flushing. Both cisterns and rain barrels can be implemented without the use of pumping devices by relying on gravity flow instead. Rain barrels and cisterns are low-cost water conservation devices that reduce runoff volume and, for very small storm events, delay and reduce the peak runoff flow rates. Both rain barrels and cisterns can provide a source of chemically untreated "soft water" for gardens and compost, free of most sediment and dissolved salts.

Storm Water Planters

Storm water planters are small landscaped storm water treatment devices that can be placed above or below ground and can be designed as infiltration or filtering practices. Storm water planters use soil infiltration and biogeochemical processes to decrease storm water quantity and improve water quality, similar to rain gardens and green roofs but smaller in size. Storm water planters are typically a few square feet of surface area compared to hundreds or thousands of square feet for rain gardens and green roofs. Types of storm water planters include contained planters, infiltration planters, and flow-through planters.

Tree Box Filters

Tree box filters are in-ground containers used to control runoff water quality and provide some detention capacity. Often premanufactured, tree box filters contain street trees, vegetation, and soil that help filter runoff before it enters a catch basin or is released from the site. Tree box filters can help meet a variety of storm water management goals, satisfy regulatory requirements for new development, protect and restore streams, control combined sewer overflows (CSOs), retrofit existing urban areas, and protect reservoir watersheds. The compact size of tree box filters allows volume and water quality control to be tailored to specific site characteristics. Tree box filters provide the added value of aesthetics while making efficient use of available land for storm water management. Typical landscape plants (for example, shrubs, ornamental grasses, trees and flowers) are an integral part of the bioretention system. Ideally, plants should be selected that can withstand alternating inundation and drought conditions and that do not have invasive root systems, which may reduce the soil's filtering capacity.

Vegetated Roofs

Green roofs consist of an impermeable roof membrane overlaid with a lightweight planting mix with a high infiltration rate and vegetated with plants tolerant of heat, drought, and periodic inundations. In addition to reducing runoff volume and frequency and improving runoff water quality, a green roof can reduce the effects of atmospheric pollution, reduce energy costs, and create an attractive environment. They have reduced replacement and maintenance costs and longer life cycles compared to traditional roofs.

7.3 Land Use Planning Policy

In summary, there are range of BMPs that can be implemented, with different options appropriate for planned new developments or for existing developments. These are best implemented within a policy framework that spans several planning levels. In order not to be too prescriptive, the documents referenced above focus more on the objectives rather than on the detailed approach to reaching them.

CHAPTER 8 CAPITAL WORKS IMPROVEMENTS

Most proposed improvements involve replacing hydraulic structures with a bridge or culvert of higher capacity. However, alternative solutions were examined in some locations. These include:

- Removal of culverts on Neil's Pond Brook (Basin A);
- Flood prevention measures at Basin B outlet;
- Construction of a grassed bioswale along T'Railway near St. Anne's Industrial Park (Basin B); and
- Addition of detention storage near Bremigen's Pond (Basin B).

Each of these options is explained in more detail below.

Class D cost estimates for the proposed improvements are contained in Appendix E. The estimates for structure replacements have been prepared assuming they are issued as a package of 8-10 structures per tender.

8.1 Removal of Culverts on Neil's Pond Brook

Neil's Pond Brook flows from Neil's Pond to Octagon Pond, crossing Burnaby Street and McNamara Driver near the Paradise Town Hall. A series of four culverts behind the Town Hall creates a significant flow restriction in the system. These culverts are labelled A-58, A-59, A-60, and A-61 in Appendices A and D.

The capacity of these culverts is approximately 70% of the Scenario 2 peak flows produced at this location. The excess volume of storm water that cannot be conveyed by culverts A-58 to A-61 will cause flooding in the vicinity of McNamara Drive and the Town Hall.

In order to minimize flooding risk, these culverts should be removed and the existing stream should be expanded into an open channel. Additional flow decreases may be achieved by employing some of the best management practices described in Chapter 7 of this report.

8.2 Flood Prevention Measures at Basin B Outlet

The outlet of the Basin B model is a large open-bottom arch culvert under Kenmount Road. The low area between St. Anne's industrial park and Kenmount Road is prone to flooding. Immediately upstream of the outlet culvert is a footbridge and three small circular culverts. The Basin B drainage network was analyzed in XPSWMM with and without the bridge and culverts, and this location was identified as a restriction to storm water flow. It was found that replacing the bridge with a structure that does not restrict flow results in a decrease in flooding depth of approximately 0.5 m.

Some residential properties on the north side of Kinsdale Road are within the expected floodplain for Scenario 2 (see Map 6, Appendix A). The construction of a berm at an elevation higher than the expected water depth will prevent flooding of Kinsdale Road. The berm height required to mitigate this flooding is approximately 95 m long at an elevation of 138.0 m (average height above existing ground of 0.5 m). The berm location is shown in Map 8, Appendix A.

8.3 Construction of Grassed Swale along T'Railway

Much of Basin B's area north of Topsail Road drains into a short stretch of storm sewer that discharges to St. Anne's Industrial Park. This industrial park is flat and experiences significant flooding during large storm events. To alleviate the flooding, redirecting the storm sewer into a new grassed swale was examined. The swale would bypass the industrial park by following T'Railway, as shown in Map 8, Appendix A. This location follows an existing easement and avoids disrupting traffic in Topsail Road.

A 400 m long swale at an average slope of 1.5% was modeled in XPSWMM and proved effective as a flow bypass. An existing ditch is present for approximately 100 m of this length. A 1 m deep cross-section with 2:1 side slopes is required to convey the flow. Two concrete box culverts are required along its length: one under the parking lot access road to 1345 Topsail Road, and one under St. Anne's Crescent. There is an existing culvert under the parking lot access road which requires upsizing, while the St. Anne's Crescent culvert will be a new addition.

The primary purpose of this swale is to alleviate flooding in St. Anne's Industrial Park. In Scenario 2, the industrial park experiences 7.5 m^3/s of overtopping flow. By constructing a swale and redirecting flow, the overtopping decreases to 2.9 m^3/s. The grassed, pervious nature of the swale provides a natural means of reducing downstream flows, as discussed in Section 7.2.2.

8.4 Detention Storage near Bremigen's Pond

A possible location for storm water detention is along the stream between Bremigen's Pond and Kenmount Road. The potential of adding berms to restrict flow was examined for this location. The berms are small structures which cause upstream water to back up until the depth is sufficient to allow overtopping. Three berms at a height of 0.6 m, which does not raise water levels enough to interfere with the adjacent industrial areas, were analyzed. The implementation of these berms causes a reduction in downstream flows of 0.5 m^3/s. Map 7 in Appendix A shows the location of each berm and the resulting floodlines for Scenario 2 conditions.

8.5 Additional Storage Considerations

CBCL considered the possibility of increased storm water storage in Elizabeth Park Subdivision and at each major pond in the Town. It was ultimately determined that these locations were unfeasible for additional storage.

8.5.1 Elizabeth Park Subdivision Storage

The Elizabeth Park subdivision is located north of Topsail Road, between the Outer Ring Road and Kenmount Road. Two existing detention ponds are present: one at the upstream end near Stephanie Avenue and one downstream, at Topsail Road. The possibility of using some of the open green spaces along the stream that flows from the upper limits of the subdivision down to Topsail Road for storm water detention was examined.

To this end, CBCL Limited visited the site on May 1, 2017 to examine potential storm water detention locations. Two potential locations were identified: upstream of the Ellesmere Avenue culvert crossing and upstream of Canterbury Drive culvert crossing in Elgin Park. Both locations were investigated in XPSWMM to determine the feasibility of creating storage areas. This option would involve excavating a wider channel and adding a pervious base layer to each. The larger channels promote infiltration and decrease flows, while causing water levels to increase.

The first potential storage location is at the upstream end of the Ellesmere Avenue culvert. In order to maintain a 0.6 m freeboard between channel flow and edge of asphalt, channel depth is limited to a maximum of 0.9 m. This depth corresponds to an available surface area of 1,050 m^2 for storage. The second potential storage location would be at the bend in the stream upstream of Canterbury Drive. A 0.3 m freeboard between channel depth and the adjacent trail was selected to determine the new channel size. This constraint limits the allowable depth to 0.6 m. Therefore, the approximate available surface area for channel increase is 3,528 m^2.

Excavating material to increase channel size at both of these locations would results in a downstream flow decrease of only 0.1 m^3/s. The stream through Elizabeth Park subdivision is one of three watercourses in Basin B that converge just upstream of the outlet. This branch is the smallest of the three and contributes approximately 16% of the total Basin B flow. For this reason, the described detention options in Elizabeth Park do not have significant impact on flows at the outlet.

8.5.2 Additional Pond Storage

As shown in Table 5-1, the water surface level of each major pond is within 1 m of existing urban development. The table lists the nearest restriction, while there are several more present at each location. Due to these restrictions it was determined that allowing additional storm water storage in this ponds should not be permitted.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

The HEC-HMS models created for Basins A, B and C for the 2013 Storm Water Master Plan were reviewed and updated with new LiDAR and SCS curve numbers that are representative of current development conditions and hydrologic soil groups for ARC type III. The models were simulated for the 1:100 AEP event using updated IDF curve data from the Ruby Line rain gauge. The peak flow simulated at the outlet of each basin was compared to the peak flows from the 2013 study and it was found that the updated models produced significantly larger flows. A sensitivity analysis of curve numbers was examined for Basin A, the results illustrate that the models are very sensitive to curve numbers. The large range of flows produced through the sensitivity analysis also illustrates the importance of calibrating the hydrologic models.

The Town had started assembling XPSWMM models of the three basins. CBCL completed building these models which simulate both hydrologic and hydraulic conditions simultaneously. Three scenarios were examined in the models:

1. Current Development and Current Climate (CD + CC);
2. Future Development and Climate Change (FD + CLC); and
3. Future Development with Proposed Improvements and Climate Change (FD + PI + CLC).

Hyetographs representing the 1:100 AEP event for current climate and climate change conditions using the Ruby Line IDF were created and simulated in the models. The absence of measured flow and/or water level data means the XPSWMM models are uncalibrated. Consequently, the proposed improvements are sized to pass flows determined from the uncalibrated models. A check of the XPSWMM model for Basin A was conducted using a photograph of Topsail River, captured during Hurricane Igor. The photo shows that Topsail Road was not overtopped during Hurricane Igor. However, with the precipitation data recorded during Hurricane Igor simulated in the XPSWMM model for Basin A, the model indicates that Topsail Road would overtop. This model check indicates that the XPSWMM models may overestimate flows and/or water levels, at least at some locations throughout the system, and also illustrates the importance of calibrating the XPSWMM models.

Many hydraulic structures in Basin A are undersized. These undersized structures significantly restrict storm water flows and create flood risks.

For Basin B, the low-lying area near the outlet at Kenmount Road was identified as flooded for Scenario 2. The footbridge just upstream of the outlet chokes the flow, causing an increase in flooding depth. St. Anne's Industrial Park was identified as a significant area of flooding. The volume of storm water flowing through the relatively flat industrial park exceeds the existing culverts capacities, causing overtopping and flooding.

Similar to Basin A, Basin C also contains many undersized structures. Flood maps created for Basin C identified several houses located in the natural flood plain of Horse Cove Brook. These areas remain at risk of flooding even if all restrictions are removed from the drainage system.

Several improvement options were examined to mitigate these risks, as discussed in Chapter 8 and Appendix D.

9.2 Recommendations

9.2.1 Recommendations for Hydrometric Monitoring

It is recommended that the Town install at least one water level gauge in each of the three basins on a main tributary. To determine flows, the Town should also collect velocity measurements at the locations of each water level gauge over a period of several years and for a range of water levels. By collecting the cross sections data and velocity measurements concurrently, flow values can be determined. By obtaining velocity measurements for a range of water levels, rating curves can be created for each water level gauge. The rating curves can then be used to determine flow for all the recorded water levels.

9.2.2 Recommendations for Capital Works Improvements

In prioritizing improvements, CBCL recommends that the Town consider upgrading structures on Topsail Road and St. Thomas Line. Appendix D lists all structure locations and identifies any that lack capacity. Structures were considered undersized if they overtopped during the simulation of 1:100 AEP rain for either Scenarios 1 or 2. The improvements should be conducted starting with the most downstream structure and progressing to upstream. If an upstream structure is upgraded first, there is the potential to exacerbate the flooding at downstream structures.

The following lists summarize the priority structures in order of recommended upgrade.

- A-1: River confluence near 1973 Topsail Road
- A-30: 1973 Topsail Road
- A-31: 1960 Topsail Road
- A-32: 1956 Topsail Road
- A-34: Newdale Road River Crossing
- A-48: St. Thomas Line near Civic No. 9
- A-49: Carlingford Drive East
- A-62: McNamara Drive River Crossing
- A-71: Windmill Road
- C-3: Whelan Crescent near St. Thomas Line
- C-5: Squires Road
- C-19: 440 St. Thomas Line

- C-20: O’Brien’s Way
- C-21: 394 St. Thomas Line
- C-22: 380 St. Thomas Line
- C-23: Lawlor’s Road
- C-6: 11 Neary Road
- C-7: Neary Road near St. Thomas Line
- C-8: Father Lacey Place
- C-25: Raymond’s Lane
- C-26: 292 St. Thomas Line
- C-27: 290 St. Thomas Line
- C-28: 282 St. Thomas Line
- C-29: Johnathan Drive
- C-33: Deborah Lynn Heights near St. Thomas Line
- C-34: Quilty’s Road
- C-35: Byrne’s Road
- C-36: Hickey’s Road
- C-24: Across St. Thomas Line near Raymond’s Lane
- C-37: 205 St. Thomas Line
- B-12 to B-14, and B-19: Bioswale along T’Railway
- B-29: Canterbury Drive
- B-31: Ellesmere Avenue
- C-45: St. Thomas Line at Stapleton’s Road
- C-47: St. Thomas Line near No. 684
- C-48: St. Thomas Line near Moonlight Drive
- C-2: 494 St. Thomas Line
- A-75: Westport Drive near St. Thomas Line
- A-54: Road behind Paradise Rec Centre
- A-78: Plateau Park near Stonewall Drive
- A-4: 64 Topsail Pond Road
- A-10: 317 Buckingham Drive
- A-11: 28 Three Island Pond Road
- A-12: 59 Three Island Pond Road
- A-13: 75 Three Island Pond Road
- A-14: 74 Three Island Pond Road
- A-15: 105 Three Island Pond Road
- A-18: Three Island Pond Road near Shalloway Road
- A-24: 291 Three Island Pond Road
- A-51: Path South of Paradise Rec Centre
- A-53: Octagon Pond East Inlet
- A-58: Neil’s Pond Brook near Town Hall Parking Lot
- A-59: Neil’s Pond Brook rear of Town Hall Parking Lot
- A-60: Neil’s Pond Brook near NW corner of Town Hall Parking Lot
- A-61: Neil’s Pond Brook north of Town Hall Parking Lot
- A-65: Downstream of Neil’s Pond Outlet

- C-2: Waterford River at Kenmount Road

It should also be noted, that the proposed structures have not been optimized nor have any preliminary designs been carried out. For instance, the available/required cover and conflicts with existing road or driveway elevations have not been analyzed. It is recommended that for each structure identified as currently undersized, a detailed design be carried out.

CBCL recommends that the Provincial Department of Municipal Affairs and Environment as well as the Department of Fisheries and Oceans be consulted during the design of the proposed infrastructure improvements.

9.2.3 Recommendation for Zero Net Increase in Runoff Policy

It is recommended that the Town of Paradise implement a zero net-runoff policy. This means that the peak storm water runoff after development cannot exceed the pre-development peak runoff. Usually, an outlet control device or group of devices, are used in combination with storm water storage to control post-development peak flows to their respective pre-development flows for a series of storms of different return periods (for example the 25-year, 50-year and 100-year storms).

The City of St. John's Subdivision Design Manual suggests their policy be met using a detention facility. However, BMPs can provide detention as well as added benefits (Chapter 7).

9.2.4 Recommendations for Guidelines for Storm Water Analyses

The Paradise Subdivision Design Guidelines require the use of the rational method when calculating peak storm water runoff for catchments of less than 10 ha, or an appropriate computer model for catchments greater than 10 ha. No constraints are provided on the characteristics of an appropriate computer model, or method. However, runoff coefficients are prescribed, suggesting the use of the SCS Method. The design drainage area does not mention using LiDAR where available. Also, currently there are no requirements for the maintenance of pre-development flows or volumes.

Storm water design guidelines are necessary to ensure that development occurs in a safe and environmentally responsible manner by:

- Not placing development in an area at risk of flooding;
- Not increasing downstream flooding and erosion risks; and
- Not deteriorating the health of the local ecosystem.

Design guidelines can thus minimize flooding risks from development as well as ecological and other impacts of development.

It is therefore recommended that the Town of Paradise adopt the following from the City of St. John's Subdivision Design Manual (latest edition):

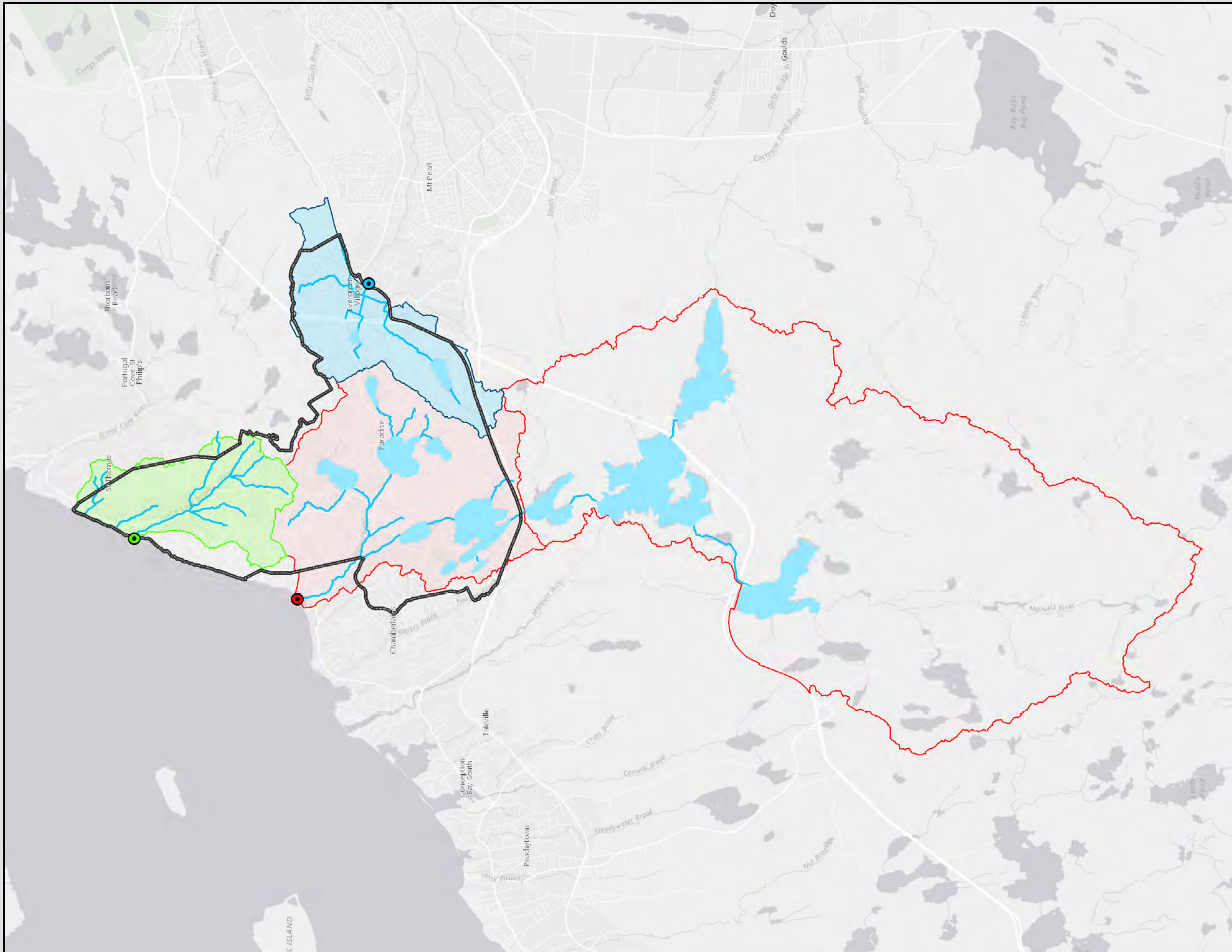
- <u>Equations</u>: infiltration (Green-Ampt), subcatchments width (area/overland flow path length), runoff routing, estimating pre-development and post-development hydrographs.
- <u>Parameters and constant</u>s: capillary suction, hydraulic conductivity, impervious depression storage, pervious depression storage, Manning's roughness coefficient.

It is recommended that the hyetograph used be based on the updated Ruby Line IDF curve for both current climate and climate change conditions, and be created using the Alternating Block Method.

The advantage of prescribing equations, parameters, constants, and rainfall design events is that the storm water design calculations for different developments are predictable, comparable, consistent and simple to review. Results are also less sensitive to the choice of parameters or equations, which could be subjective. If/when ownership, operation and maintenance of a given facility are turned over to the Town, the operation and maintenance will be simpler and more predictable.

APPENDIX A

Maps



Legend

- Basin A Outlet
- Basin B Outlet
- Basin C Outlet
- Town Boundary
- Flow Paths
- Basin A - Partial Contribution
- Basin A
- Basin B
- Basin C
- Ponds

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PARADISE STORMWATER MANAGEMENT PLAN

Town of Paradise Drainage Basins

MAP 1

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:75,000

0 950 1,900 3,800 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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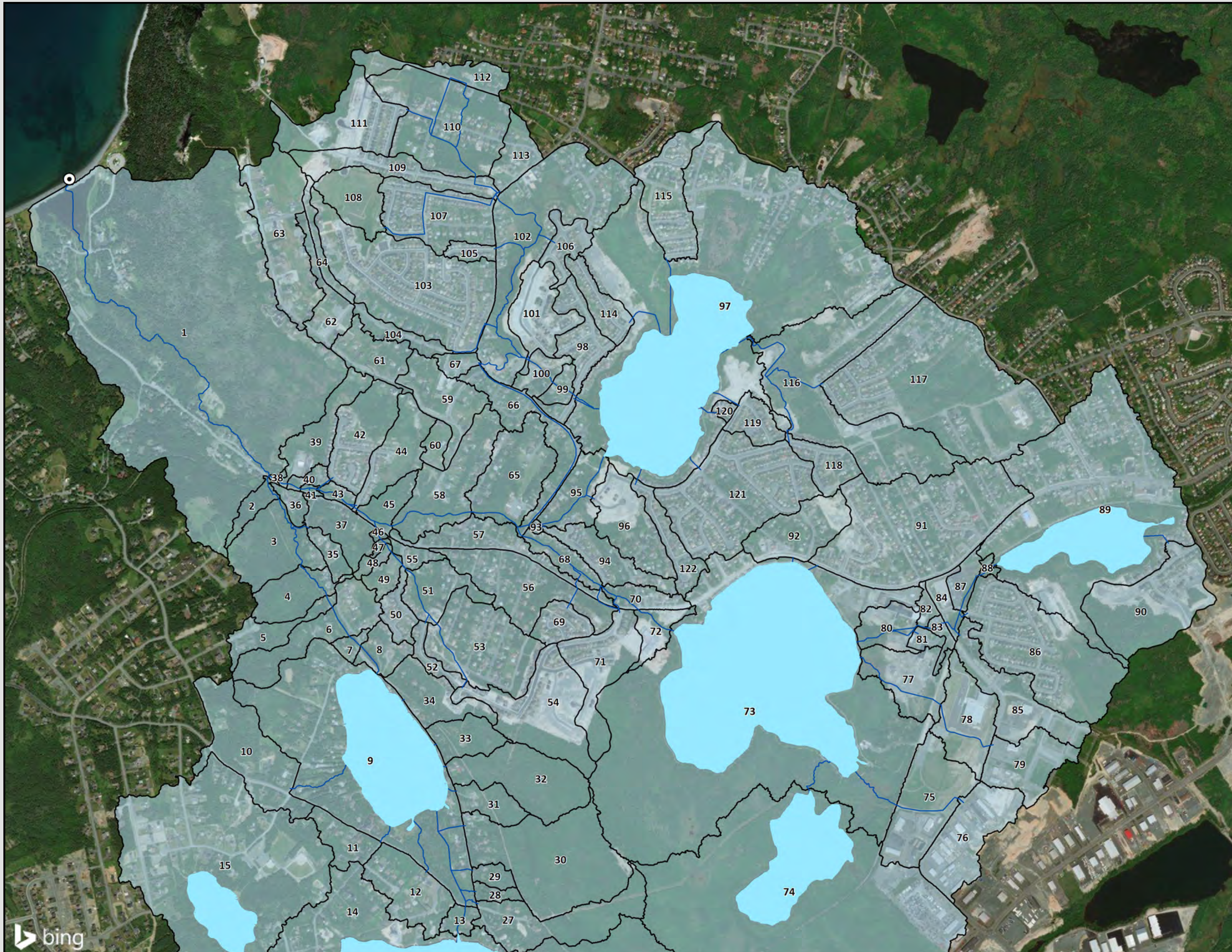

Legend

- Basin A Outlet
- Major Flow Paths
- Ponds
- Watersheds

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PARADISE STORMWATER MANAGEMENT PLAN

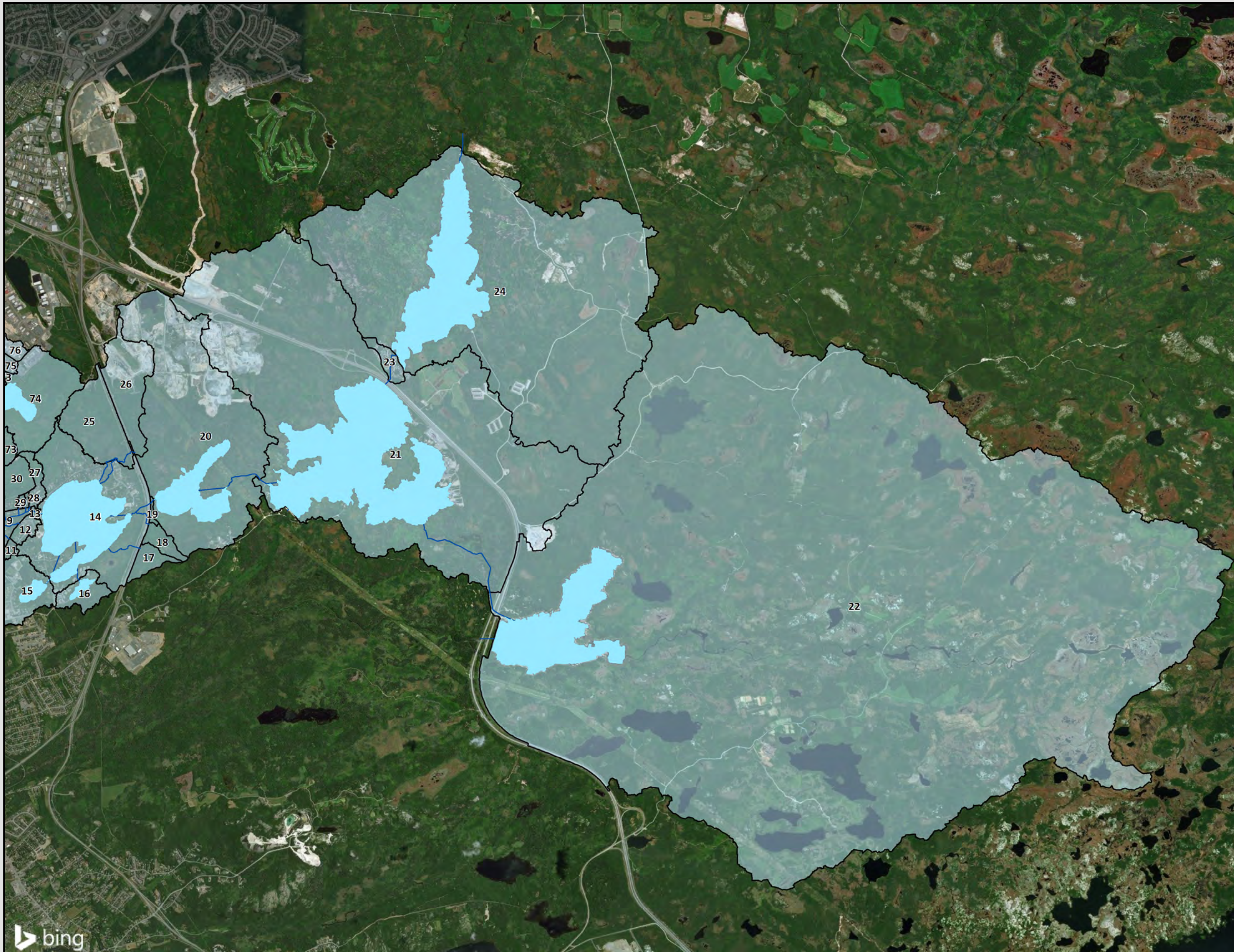
BASIN A - Drainage Areas

MAP 2-1

Drawn: MD	Date: 03/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:15,500

0 205 410 820 m

Coordinate System: NAD 1983 MTM 1
Units: Meter



Legend

- Basin A Outlet
- Major Flow Paths
- Ponds
- Watersheds

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN A - Drainage Areas

MAP 2-2

Drawn: MD	Date: 03/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:47,000

0 600 1,200 2,400 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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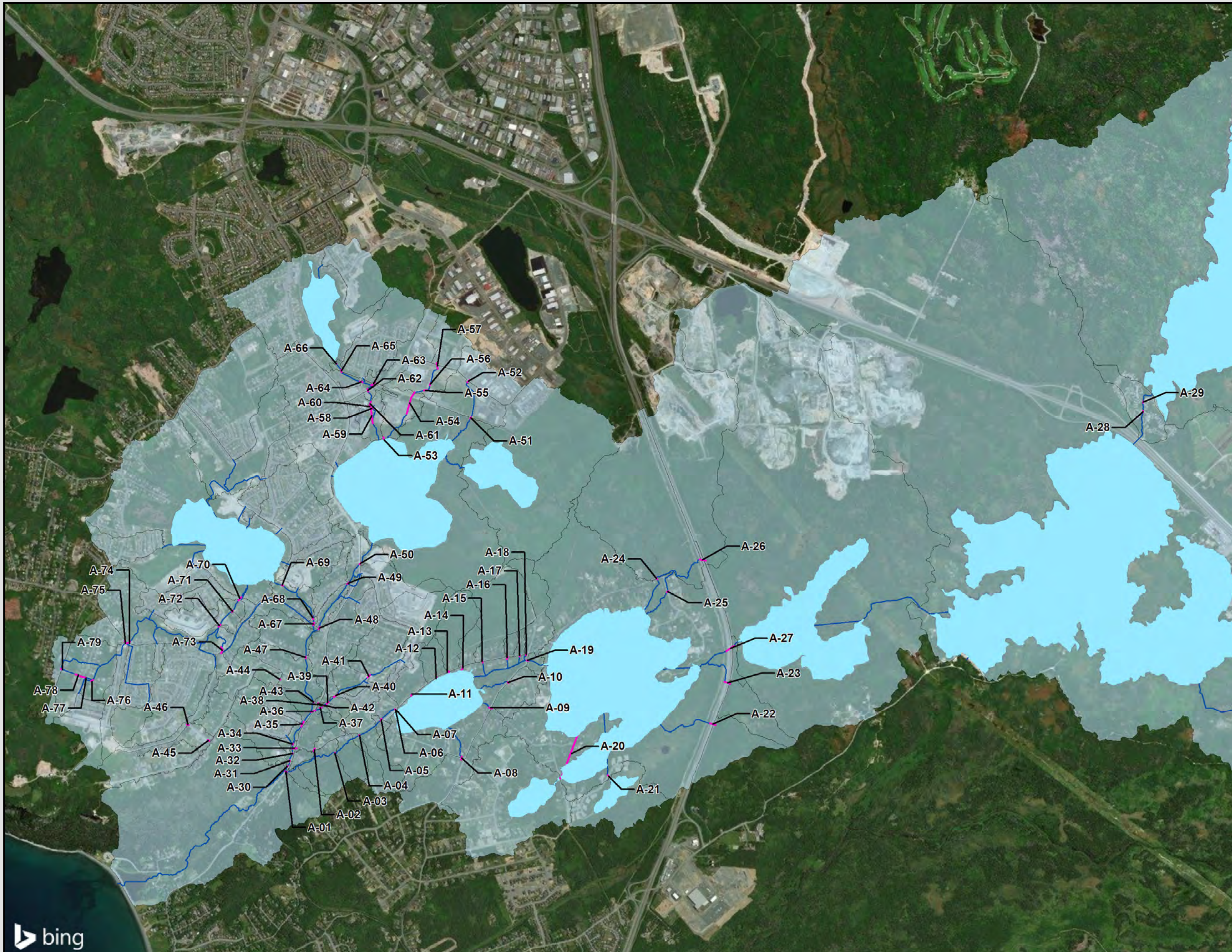

Legend

- Ponds
- Hydraulic Structures
- Major Flow Paths
- Watersheds

TOWN OF Paradise

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN A - Hydraulic Structures

MAP 3

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:27,500

0 355 710 1,420 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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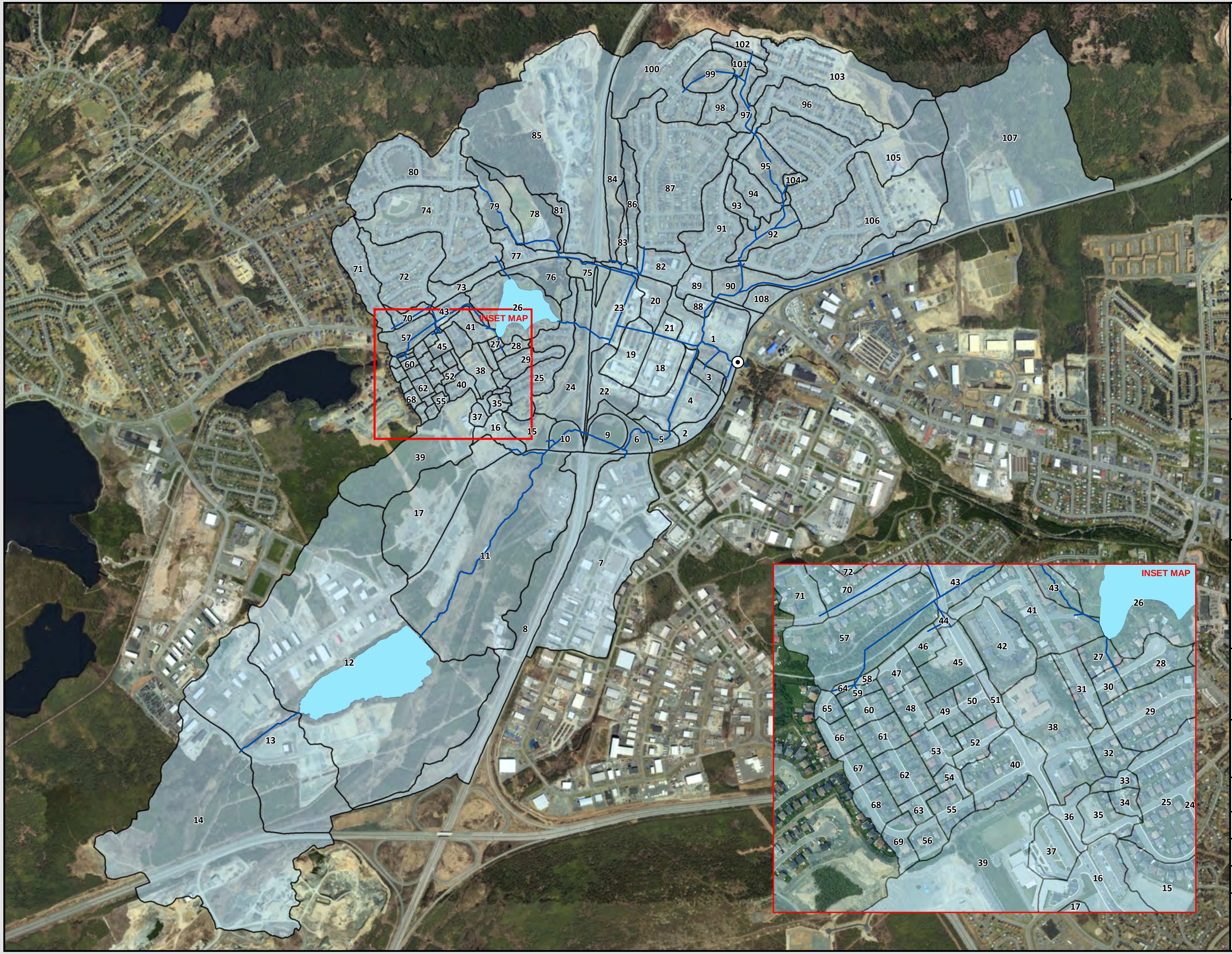

Legend

- Basin B Outlet
- Major Flow Paths
- Ponds
- Watersheds

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN B - Drainage Areas

MAP 4

Drawn: MD	Date: 03/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:17,500

0 230 460 920 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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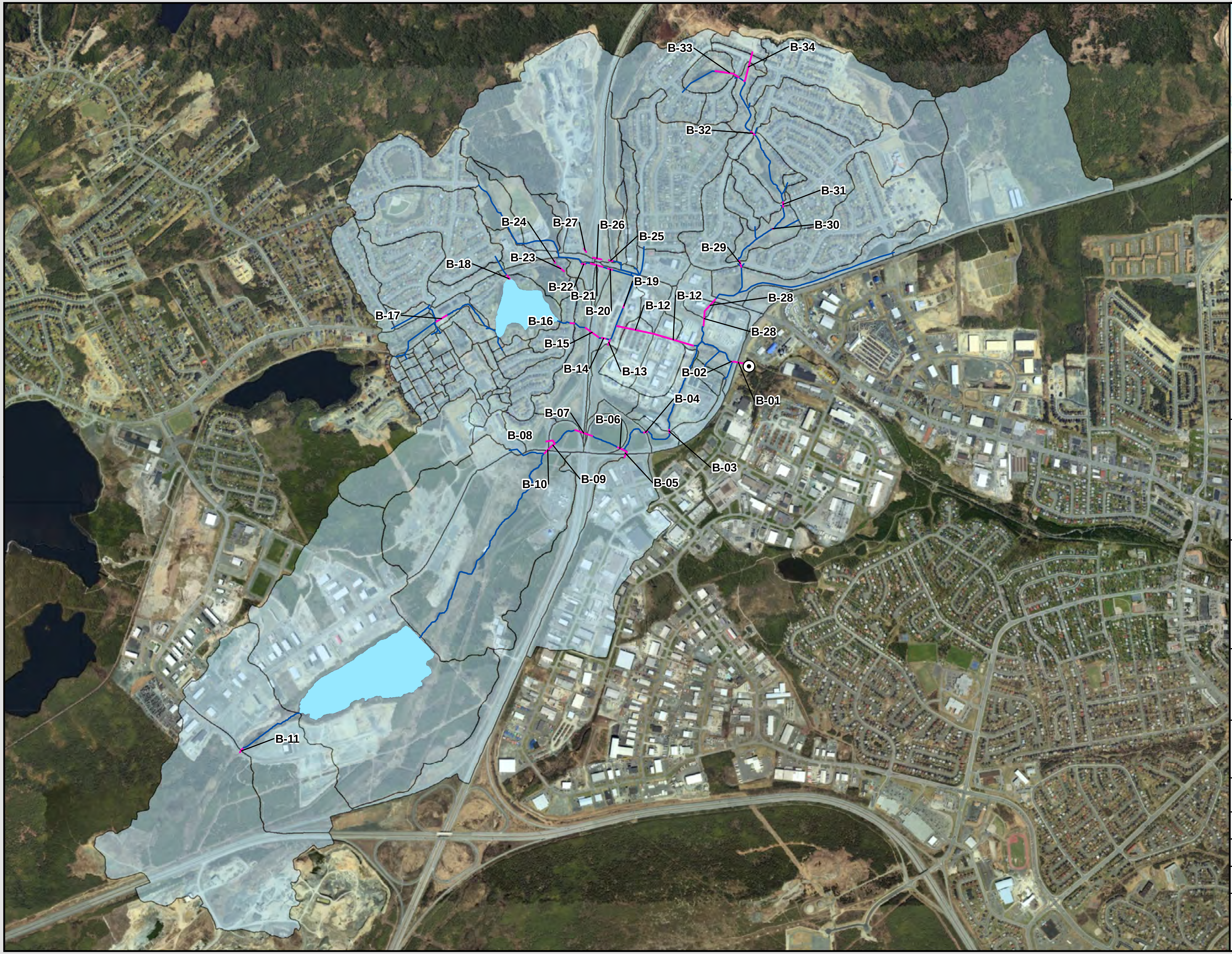

Legend

- Basin B Outlet
- Ponds
- Hydraulic Structures
- Major Flow Paths
- Watersheds

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN B - Hydraulic Structures

MAP 5

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:17,500

0 230 460 920 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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Legend

- Basin B Outlet
- Closed Conduit Flowpaths
- Open Channel Flowpaths
- Scenario 2 Floodlines
- Footbridge

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN B - Scenario 2 Floodlines

MAP 6

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:2,500

0 30 60 120 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

Legend

- Proposed Berm
- Closed Conduit Flowpaths
- Open Channel Flowpaths
- Storm Water Storage Area
- Ponds

TOWN OF Paradise

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN B - Bremigan's Stream Storage Options

MAP 7

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:4,000

0 50 100 200 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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Legend

- Basin B Outlet
- Proposed Bioswale
- Proposed Berm
- Closed Conduit Flowpaths
- Open Channel Flowpaths
- Scenario 3 Floodlines
- Footbridge

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN B - Proposed Improvements

MAP 8

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:2,500

0 30 60 120 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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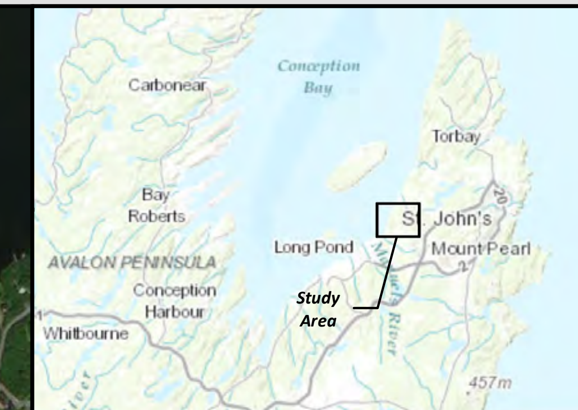

Legend

- Basin C Outlet
- Major Flow Paths
- Horse Cove Brook Watersheds
- Other Watersheds

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN C - Drainage Areas

MAP 9

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:17,000

0 225 450 900 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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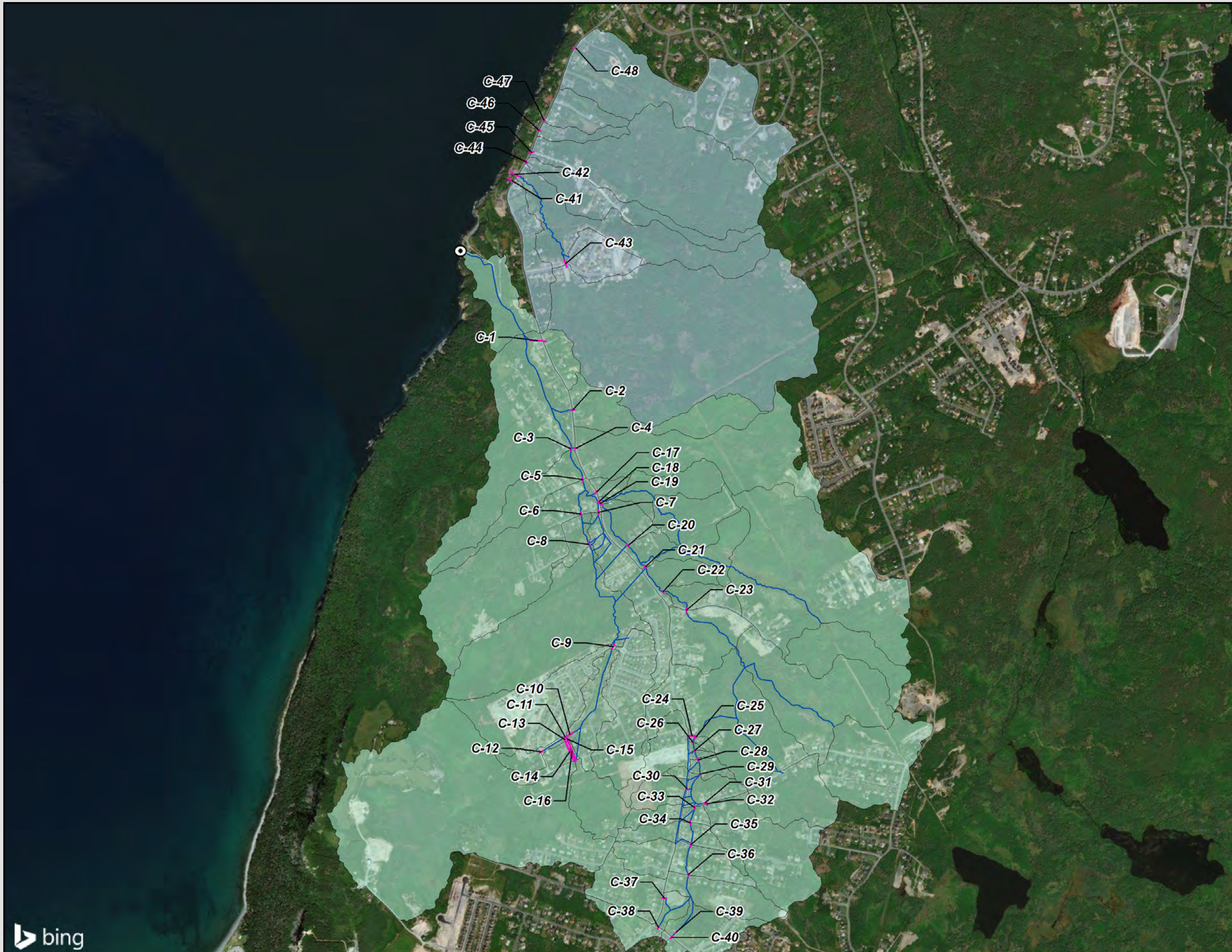

Legend

- Basin C Outlet
- Hydraulic Structures
- Major Flow Paths
- Horse Cove Brook Watersheds
- Other Watersheds

TOWN OF Paradise

CBCL LIMITED
Consulting Engineers

PARADISE STORMWATER MANAGEMENT PLAN

BASIN C - Hydraulic Structures

MAP 10

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:17,000

0 225 450 900 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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Legend

- Basin C Outlet
- Wetland Boundary
- Horse Cove Brook Watershed
- Scenario 2 Floodlines

TOWN OF Paradise

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN C - Scenario 2 Floodlines

MAP 11

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:13,500

0 180 360 720 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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Legend

- Basin C Outlet
- Wetland Boundary
- Horse Cove Brook Watershed
- Scenario 3 Floodlines

TOWN OF Paradise

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PARADISE STORMWATER MANAGEMENT PLAN

BASIN C - Scenario 3 Floodlines

MAP 12

Drawn: MD	Date: 04/08/2017
Checked:	Project No.: 163063.00
Approved:	Scale @ 11"x17" 1:13,500

0 180 360 720 m

Coordinate System: NAD 1983 MTM 1
Units: Meter

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APPENDIX B

HEC-HMS Results

Basin A: Original Model

Existing Development Conditions

Storm Event: 1 in 100 Year

Hydrologic Element	Drainage Area (km^2)	Maximum Discharge (m^3/s)	Duration at Max Discharge
Junction-1	1.231	2.729	6 Hour
Junction-10	0.682	1.269	6 Hour
Junction-11	2.7	5.023	12 Hour
Junction-12	1.021	4.268	2 Hour
Junction-13	3.339	19.233	2 Hour
Junction-14	3.904	2.211	24 Hour
Junction-15	0.576	3.171	2 Hour
Junction-16	1.404	4.08	6 Hour
Junction-17	0.575	2.859	2 Hour
Junction-18	0.405	0.783	6 Hour
Junction-19	6.299	8.947	2 Hour
Junction-2	8.926	5.256	24 Hour
Junction-20	1.389	7.493	2 Hour
Junction-21	0.6417	2.329	2 Hour
Junction-22	17.416	18.636	2 Hour
Junction-23	0.233	1.23	2 Hour
Junction-3	4.776	3.435	2 Hour
Junction-4	4.776	3.441	2 Hour
Junction-5	16.012	17.221	2 Hour
Junction-6	9.713	8.286	2 Hour
Junction-7	3.904	2.211	24 Hour
Junction-8	5.932	11.163	2 Hour
Junction-9	0.609	0.972	12 Hour
Reach-1	16.012	17.209	2 Hour
Reach-10	8.926	5.256	24 Hour
Reach-11	4.776	3.435	2 Hour
Reach-13	3.904	2.211	24 Hour
Reach-14	0.108	0.192	6 Hour
Reach-15	1.021	4.259	2 Hour
Reach-17	0.162	1.967	2 Hour
Reach-18	0.234	1.013	2 Hour
Reach-2	6.299	8.945	2 Hour

Basin A: Updated Model

Existing Development Conditions

Storm Event: 1 in 100 Year (existing) - Alternating Block Method

Hydrologic Element	Drainage Area (km^2)	Peak Discharge (m^3/s)
Junction-1	1.18	13.9
Junction-10	0.475	5.9
Junction-11	2.257	27.3
Junction-12	0.967	5.3
Junction-13	3.159	31.4
Junction-14	3.936	6.7
Junction-15	0.586	7
Junction-16	1.461	16.1
Junction-17	0.284	3.7
Junction-18	0.219	2.8
Junction-19	6.163	11.3
Junction-2	8.793	16.4
Junction-20	1.376	17.4
Junction-21	0.739	9.7
Junction-22	17.143	51.9
Junction-23	0.24	3.3
Junction-3	4.436	5.5
Junction-4	4.436	5.5
Junction-5	15.72	36.3
Junction-6	9.557	25.5
Junction-7	3.936	6.7
Junction-8	5.833	20.5
Junction-9	0.628	7
Reach-1	15.72	36.3
Reach-10	8.793	16.4
Reach-11	4.436	5.5
Reach-13	3.936	6.7
Reach-14	0.331	4.3
Reach-15	0.967	5.2
Reach-17	0.168	2.5
Reach-18	0.209	2.5
Reach-2	6.163	11.3

Reach-20	0.277	0.524	6 Hour
Reach-21	0.609	0.972	12 Hour
Reach-22	1.231	2.706	6 Hour
Reach-23	0.295	0.583	6 Hour
Reach-24	0.372	1.108	6 Hour
Reach-25	0.575	2.859	2 Hour
Reach-27	1.404	4.08	6 Hour
Reach-28	1.192	1.266	24 Hour
Reach-3	4.64	3.76	6 Hour
Reach-5	0.144	0.753	2 Hour
Reach-6	0.305	1.086	2 Hour
Reach-7	0.282	0.789	2 Hour
Reach-8	9.713	8.276	2 Hour
Reach-9	0.363	1.075	6 Hour
Reservoir-1	5.932	8.6	1/2 Hour
Reservoir-2	4.64	3.76	6 Hour
Reservoir-4	4.36	2.312	24 Hour
Reservoir-5	3.796	2.167	24 Hour
Reservoir-6	0.576	0.356	24 Hour
Reservoir-7	1.192	1.267	24 Hour
SUB A0	1.404	2.639	6 Hour
SUB A1 i	0.144	7.55	2 Hour
SUB A1 ii	0.089	1.521	2 Hour
SUB A10	1.292	2.552	2 Hour
SUB A11 i	0.246	1.076	6 Hour
SUB A11 ii	0.424	15.043	2 Hour
SUB A12	0.363	0.972	12 Hour
SUB A13	1.94	1.246	12 Hour
SUB A14 i	0.609	2.78	2 Hour
SUB A14 ii	0.858	0.192	6 Hour
SUB A15	0.416	0.656	12 Hour
SUB A16	0.108	0.513	6 Hour
SUB A17	0.395	1.017	2 Hour
SUB A18	0.18	0.754	2 Hour
SUB A19	0.234	0.514	2 Hour
SUB A2ii	0.3367	1.972	2 Hour
SUB A2 i	0.305	4.163	2 Hour
SUB A20	0.162	18.736	2 Hour

Reach-20	0.256	3.1
Reach-21	0.628	7
Reach-22	1.18	13.8
Reach-23	0.319	4.4
Reach-24	0.426	5.5
Reach-25	0.284	3.7
Reach-27	1.461	16.1
Reach-28	0.708	1.6
Reach-3	4.555	4.8
Reach-5	0.07	1
Reach-6	0.342	4.6
Reach-7	0.358	4.2
Reach-8	9.557	25.5
Reach-9	0.503	6
Reservoir-1	5.833	9.1
Reservoir-2	4.555	4.8
Reservoir-4	4.126	4.6
Reservoir-5	3.605	6
Reservoir-6	0.586	0.9
Reservoir-7	0.708	1.6
SUB A0	1.423	15.7
SUB A1 i	0.07	1
SUB A1 ii	0.17	2.3
SUB A10	1.278	16.8
SUB A11 i	0.421	5.1
SUB A11 ii	0.261	3.5
SUB A12	0.503	6
SUB A13	2.298	30
SUB A14 i	0.628	7
SUB A14 ii	0.606	7.7
SUB A15	0.31	4.1
SUB A16	0.331	4.3
SUB A17	0.269	3.6
SUB A18	0.209	2.3
SUB A19	0.209	2.5
SUB A2ii	0.397	5.1
SUB A2 i	0.342	4.6
SUB A20	0.168	2.5

SUB A21	0.445	1.108	6 Hour
SUB A22	1.752	6.932	2 Hour
SUB A23 i	0.372	2.861	2 Hour
SUB A23 ii	1.017	1.309	2 Hour
SUB A24	0.575	0.556	6 Hour
SUB A25	0.189	4.085	6 Hour
SUB A26	0.239	7.777	2 Hour
SUB A27	1.404	1.087	2 Hour
SUB A28	1.192	1.369	2 Hour
SUB A3	0.282	0.791	2 Hour
SUB A4	0.551	3.221	2 Hour
SUB A5	0.936	2.154	6 Hour
SUB A6	0.295	0.584	6 Hour
SUB A7	0.405	0.783	6 Hour
SUB A8	0.277	0.527	6 Hour
SUB A9	0.367	1.27	2 Hour

SUB A21	0.381	4.9
SUB A22	2.182	27.7
SUB A23 i	0.426	5.5
SUB A23 ii	0.95	12.2
SUB A24	0.284	3.7
SUB A25	0.183	2.5
SUB A26	0.301	4.2
SUB A27	1.461	16.2
SUB A28	0.708	9.9
SUB A3	0.358	4.2
SUB A4	0.548	7
SUB A5	0.861	9.6
SUB A6	0.319	4.4
SUB A7	0.219	2.8
SUB A8	0.256	3.1
SUB A9	0.33	3.8

Basin B: Original Model

Existing Development Conditions

Storm Event: 1 in 100 Year

Hydrologic Element	Drainage Area (km^2)	Maximum Discharge (m^3/s)	Duration at Max Discharge
Junction-1	1.09	5.975	2 Hour
Junction-2	3.379	8.58	2 Hour
Junction-3	2.524	5.45	2 Hour
Junction-4	1.611	9.009	2 Hour
Junction-5	1.611	3.928	12 Hour
Junction-6	3.379	8.585	2 Hour
Junction-7	6.177	13.975	12 Hour
Junction-8	0.694	3.552	2 Hour
Reach-2i	0.694	3.538	2 Hour
Reach-2 ii	1.09	5.956	2 Hour
Reach-3	1.187	3.203	12 Hour
Reach-4 i	2.524	5.443	2 Hour
Reach-4 ii	3.379	8.58	2 Hour
Reservoir-1	1.632	1.596	24 Hour
Reservoir-2	1.187	3.203	12 Hour
Reservoir-3	0.203	0.506	12 Hour
Reservoir-4	1.611	3.928	12 Hour
SUB B0	0.725	2.151	2 Hour
SUB B1	0.907	12.141	2 Hour
SUB B2 i	0.892	5.382	2 Hour
SUB B2 ii	0.855	3.704	6 Hour
SUB B3 i	0.203	1.478	2 Hour
SUB B3 ii	0.491	3.359	2 Hour
SUB B3 iii	0.396	2.56	2 Hour
SUB B3 iv	0.521	3.148	2 Hour
SUB B4	1.187	6.747	2 Hour

Basin B: Updated Model

Existing Development Conditions

Storm Event: 1 in 100 Year (existing) - Alternating Block Method

Hydrologic Element	Drainage Area (km^2)	Peak Discharge (m^3/s)
Junction-1	1.808	19.2
Junction-2	4.17	32.7
Junction-3	3.151	20.7
Junction-4	1.897	20.3
Junction-5	1.897	10
Junction-6	4.17	32.7
Junction-7	7.794	51.7
Junction-8	0.747	7.3
Reach-2i	0.747	7.3
Reach-2 ii	1.808	19.2
Reach-3	1.727	10.3
Reach-4 i	3.151	20.7
Reach-4 ii	4.17	32.7
Reservoir-1	1.937	7.6
Reservoir-2	1.727	10.3
Reservoir-3	0.191	1.2
Reservoir-4	1.897	10
SUB B0	0.886	12.1
SUB B1	1.051	14.5
SUB B2 i	1.214	15.6
SUB B2 ii	1.019	12.1
SUB B3 i	0.191	2.5
SUB B3 ii	0.556	6.3
SUB B3 iii	1.061	12.1
SUB B3 iv	0.089	1.2
SUB B4	1.727	21.5

Basin C: Original Model

Existing Development Conditions

Storm Event: 1 in 100 Year

Hydrologic Element	Drainage Area (km^2)	Maximum Discharge (m^3/s)	Duration at Max Discharge
Junction-1	1.556	2.745	12 Hour
Junction-10	4.2206	8.686	6 Hour
Junction-11	4.3786	9.11	6 Hour
Junction-12	0.025	0.104	2 Hour
Junction-13	0.837	1.424	12 Hour
Junction-14	1.556	2.746	12 Hour
Junction-15	0.6766	2.308	2 Hour
Junction-16	0.061	0.138	6 Hour
Junction-17	0.4305	1.499	2 Hour
Junction-18	0.298	1.155	2 Hour
Junction-19	0.286	0.496	12 Hour
Junction-2	1.2786	3.797	6 Hour
Junction-20	5.2286	11.366	6 Hour
Junction-3	0.349	0.919	6 Hour
Junction-4	0.395	1.066	6 Hour
Junction-5	0.89	2.601	6 Hour
Junction-6	0.273	0.968	2 Hour
Junction-7	1.2786	3.797	6 Hour
Junction-8	1.19	2.057	12 Hour
Junction-9	1.842	3.24	12 Hour
Reach-1	4.3786	9.098	6 Hour
Reach-10	0.725	1.343	6 Hour
Reach-11	0.972	1.683	12 Hour
Reach-12	0.89	2.6	6 Hour
Reach-13	0.395	1.065	6 Hour
Reach-16	0.025	0.104	2 Hour
Reach-2	1.556	2.744	12 Hour
Reach-3 i	0.124	0.218	6 Hour
Reach-3 ii	0.286	0.496	12 Hour
Reach-5 ii	1.19	0.137	6 Hour
Reach-5 iii	1.556	2.057	12 Hour
Reach-5 - i	0.061	2.745	12 Hour

Basin C: Updated Model

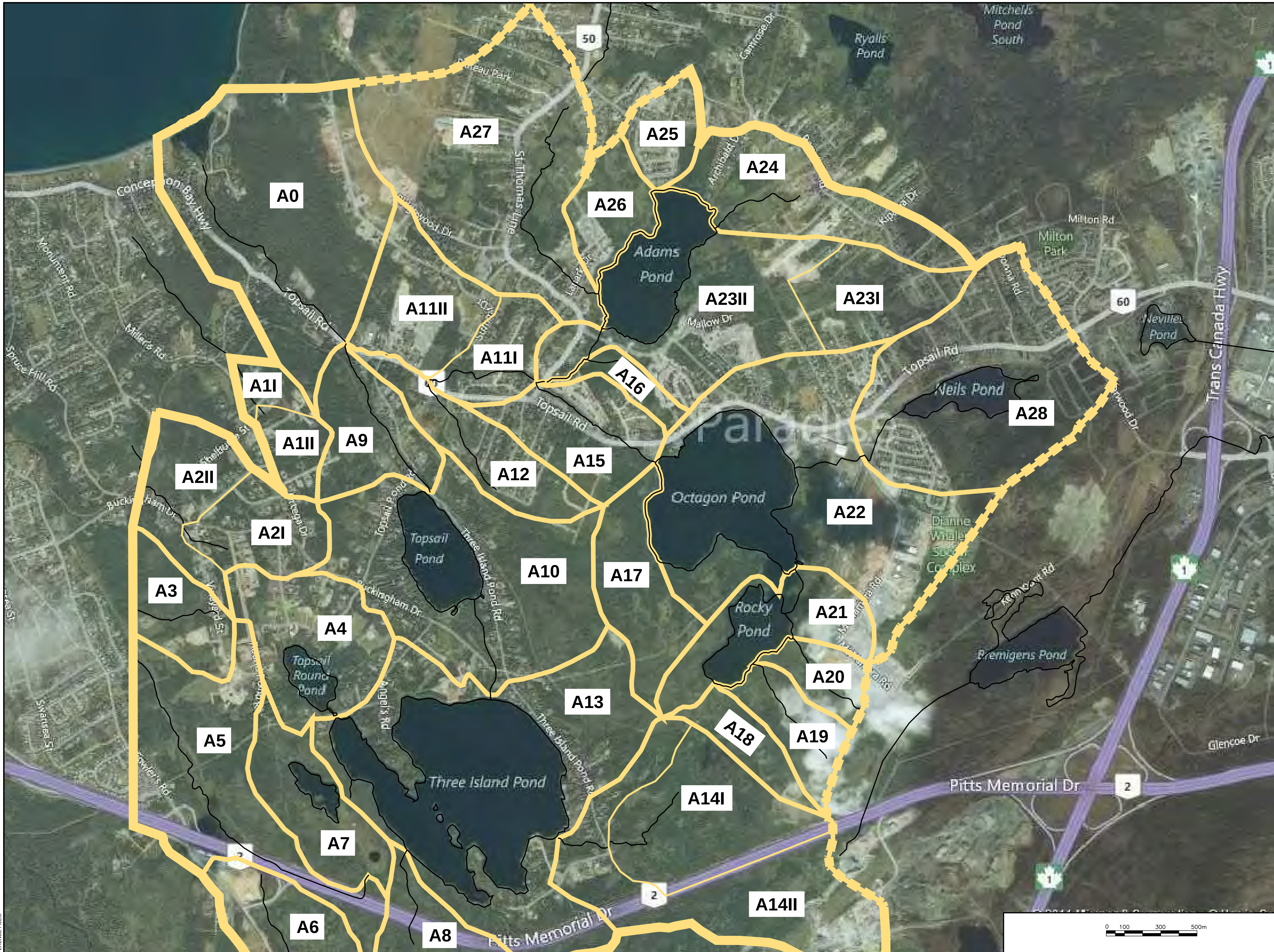
Existing Development Conditions

Storm Event: 1 in 100 Year (existing) - Alternating Block Method

Hydrologic Element	Drainage Area (km^2)	Peak Discharge (m^3/s)
Junction-1	1.363	15.2
Junction-10	4.052	44.9
Junction-11	4.189	46.3
Junction-12	0.154	2.2
Junction-13	0.728	7.9
Junction-14	1.363	15.2
Junction-15	0.667	8.5
Junction-16	0.045	0.6
Junction-17	0.299	3.9
Junction-18	0.231	3.1
Junction-19	0.361	4.4
Junction-2	1.294	15.7
Junction-20	4.66	51
Junction-3	0.246	3.4
Junction-4	0.267	3.6
Junction-5	0.853	10.4
Junction-6	0.295	3.5
Junction-7	1.294	15.7
Junction-8	1.056	11.7
Junction-9	1.724	19.1
Reach-1	4.189	46.3
Reach-10	0.662	6.6
Reach-11	1.542	15.2
Reach-12	0.853	10.4
Reach-13	0.267	3.6
Reach-16	0.154	2.2
Reach-2	1.363	15.2
Reach-3 i	0.16	1.9
Reach-3 ii	0.361	4.4
Reach-5 ii	1.056	11.7
Reach-5 iii	1.363	15.2
Reach-5 - i	0.045	0.6

Reach-6	1.6536	4.6	6 Hour
Reach-7 i	0.298	1.151	2 Hour
Reach-7 ii	0.4305	1.497	2 Hour
Reach-8	0.273	0.968	2 Hour
Reach-9	0.329	0.68	6 Hour
SUB C1 i	0.124	1.684	12 Hour
SUB C1 ii	0.162	0.274	6 Hour
SUB C10	0.972	0.663	2 Hour
SUB C11	0.142	0.179	2 Hour
SUB C12	0.207	1.545	2 Hour
SUB C13	0.046	0.105	2 Hour
SUB C14	0.495	0.218	6 Hour
SUB C15	0.025	0.28	12 Hour
SUB C2 i	0.776	1.352	12 Hour
SUB C2 ii	0.061	0.138	6 Hour
SUB C3 i	0.353	0.638	12 Hour
SUB C3 ii	0.366	0.759	6 Hour
SUB C4 i	0.298	1.155	2 Hour
SUB C4 ii	0.2211	0.974	2 Hour
SUB C4 iii	0.1325	0.61	2 Hour
SUB C5	0.273	0.97	2 Hour
SUB C6	0.329	0.682	6 Hour
SUB C7	0.725	1.347	6 Hour
SUB C8	0.375	0.836	6 Hour
SUB C9 i	0.158	0.432	6 Hour
SUB C9 ii	0.85	2.284	6 Hour

Reach-6	1.666	20.1
Reach-7 i	0.231	3.1
Reach-7 ii	0.299	3.9
Reach-8	0.295	3.5
Reach-9	0.332	4
SUB C1 i	0.16	1.9
SUB C1 ii	0.201	2.5
SUB C10	1.542	15.2
SUB C11	0.069	0.9
SUB C12	0.177	2.5
SUB C13	0.021	0.2
SUB C14	0.586	6.8
SUB C15	0.154	2.2
SUB C2 i	0.683	7.5
SUB C2 ii	0.045	0.6
SUB C3 i	0.328	4
SUB C3 ii	0.307	3.7
SUB C4 i	0.231	3.1
SUB C4 ii	0.214	2.9
SUB C4 iii	0.068	0.9
SUB C5	0.295	3.5
SUB C6	0.332	4
SUB C7	0.662	6.6
SUB C8	0.372	4.4
SUB C9 i	0.137	1.6
SUB C9 ii	0.471	4.7



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REV.	REVISIONS	REVISED BY	APP. BY	DATE
C	REVISED FOR REPORT	A.E.	A.E.	12/05/15
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A	ISSUED FOR REPORT	K.B.	K.B.	11/02/15

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PROJECT
STORM WATER MANAGEMENT STUDY

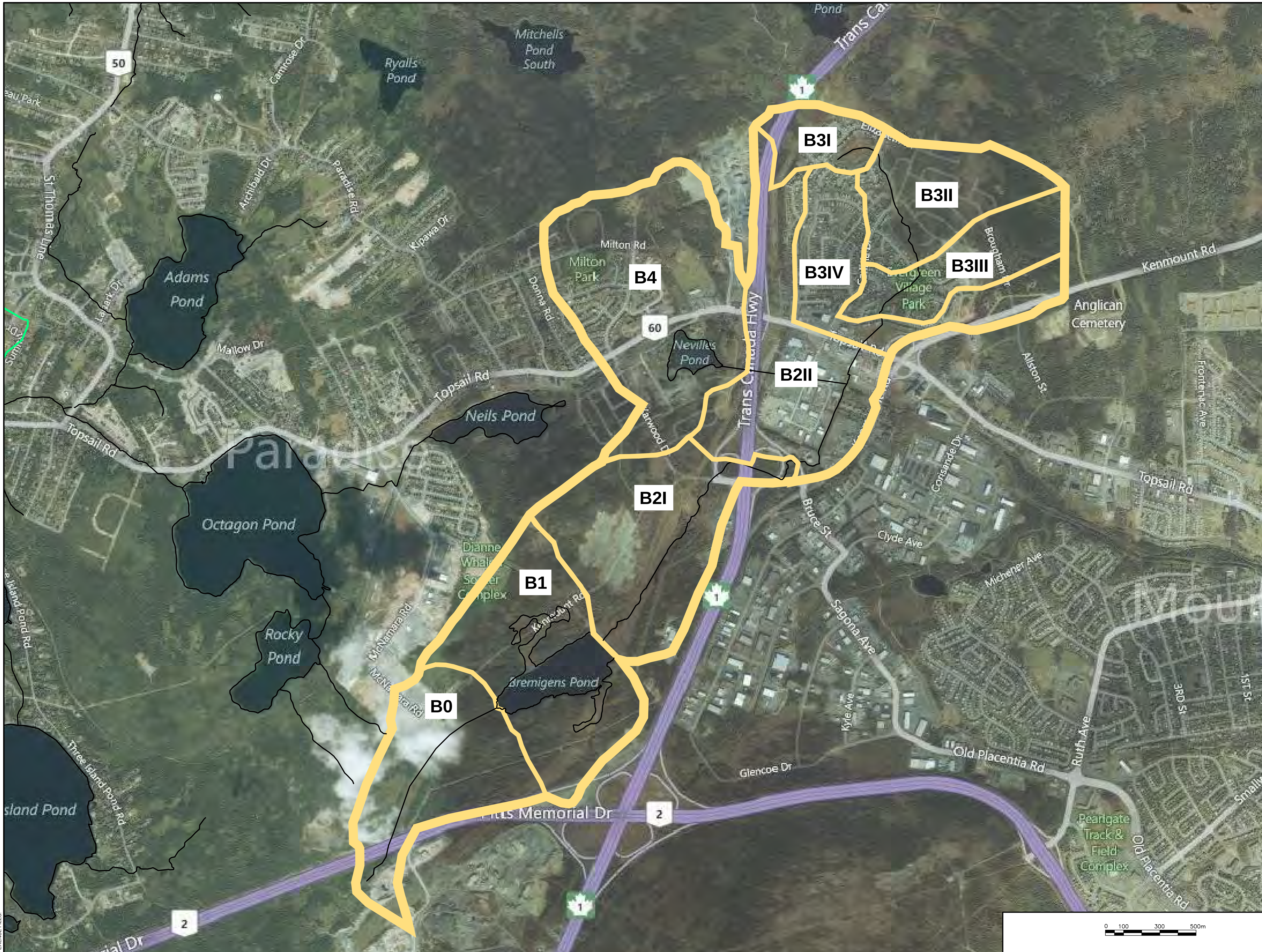
TITLE
BASIN A

DESIGNED BY K.B.	CHECKED BY K.B.	DATE 11/02/15
DRAWN BY J.J.B. & J.B.	APPROVED BY K.B.	DATE 11/02/15
SCALE AS SHOWN	BNG PROJ. No. 723487	
	CLIENT PROJ. No.	

DRAWING No. PR 1 – XX – CV – XX – 002

REV. C

REFERENCE FILES



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DRAWN BY	J.J.B. & J.B.	APPROVED BY	K.B.	DATE	11/02/15
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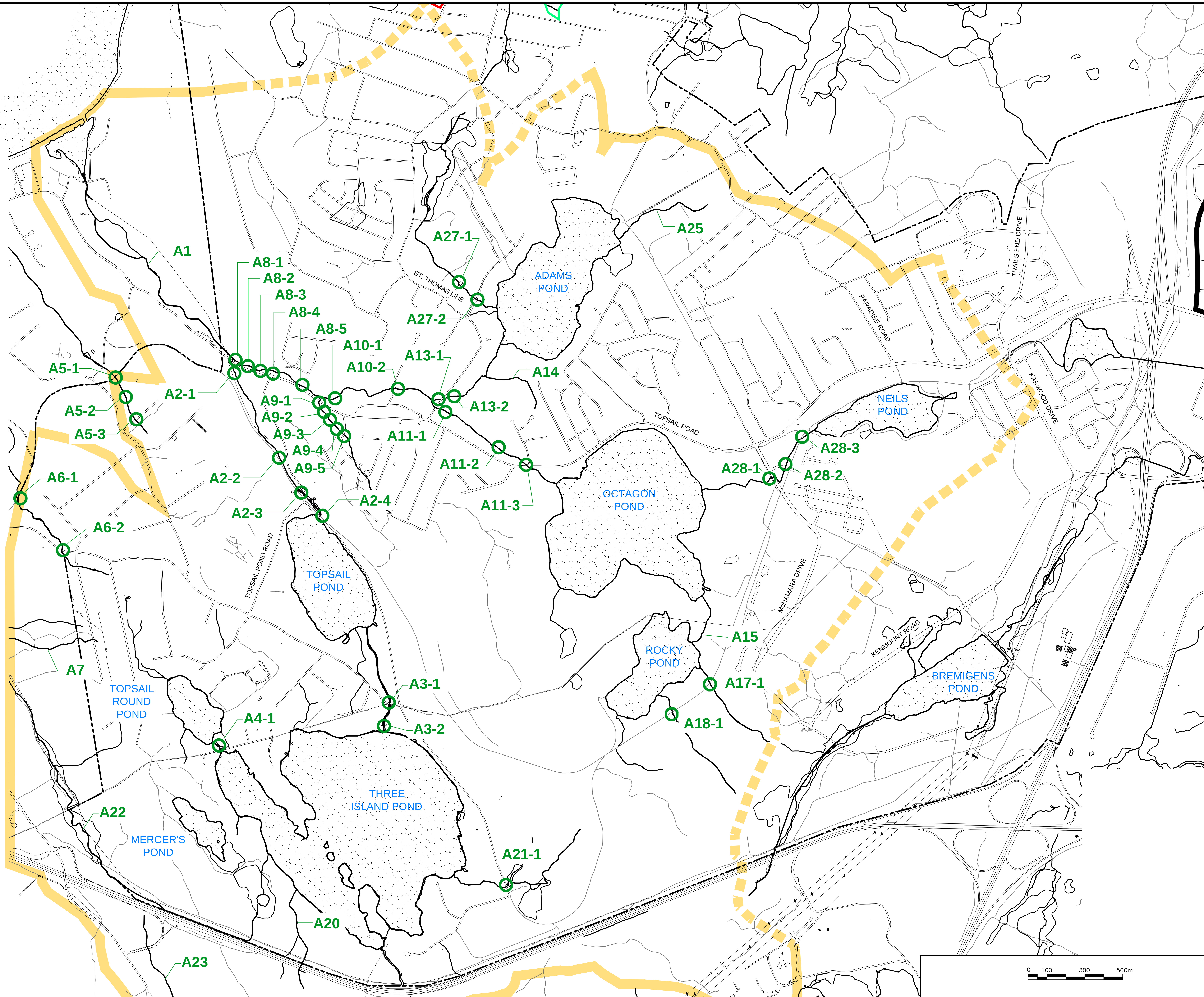
DRAWN BY	J.J.B. & J.B.	APPROVED BY	K.B.	DATE	11/02/15
SCALE	AS SHOWN	BNG PROJ. No.	723487		
		CLIENT PROJ. No.			

DRAWING No. **PR 1– XX –CV– XX –004**

REV. **D**



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LEGEND

CONTROL POINT LOCATION

A12-3

RIVER NUMBER

CONTROL POINT NUMBER

TOWN BOUNDARY

REV.	REVISIONS	REVISED BY	APP. BY	DATE
D	REVISED FOR REPORT	A.E.	A.E.	13/07/08
C	REVISED FOR REPORT	A.E.	A.E.	12/05/15
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TITLE

BASIN A
CONTROL POINTS

DESIGNED BY	K.B.	CHECKED BY	K.B.	DATE	11/02/15
DRAWN BY	J.J.B. & J.B.	APPROVED BY	K.B.	DATE	11/02/15
SCALE	AS SHOWN	BNG PROJ. No.	723487		
		CLIENT PROJ. No.			

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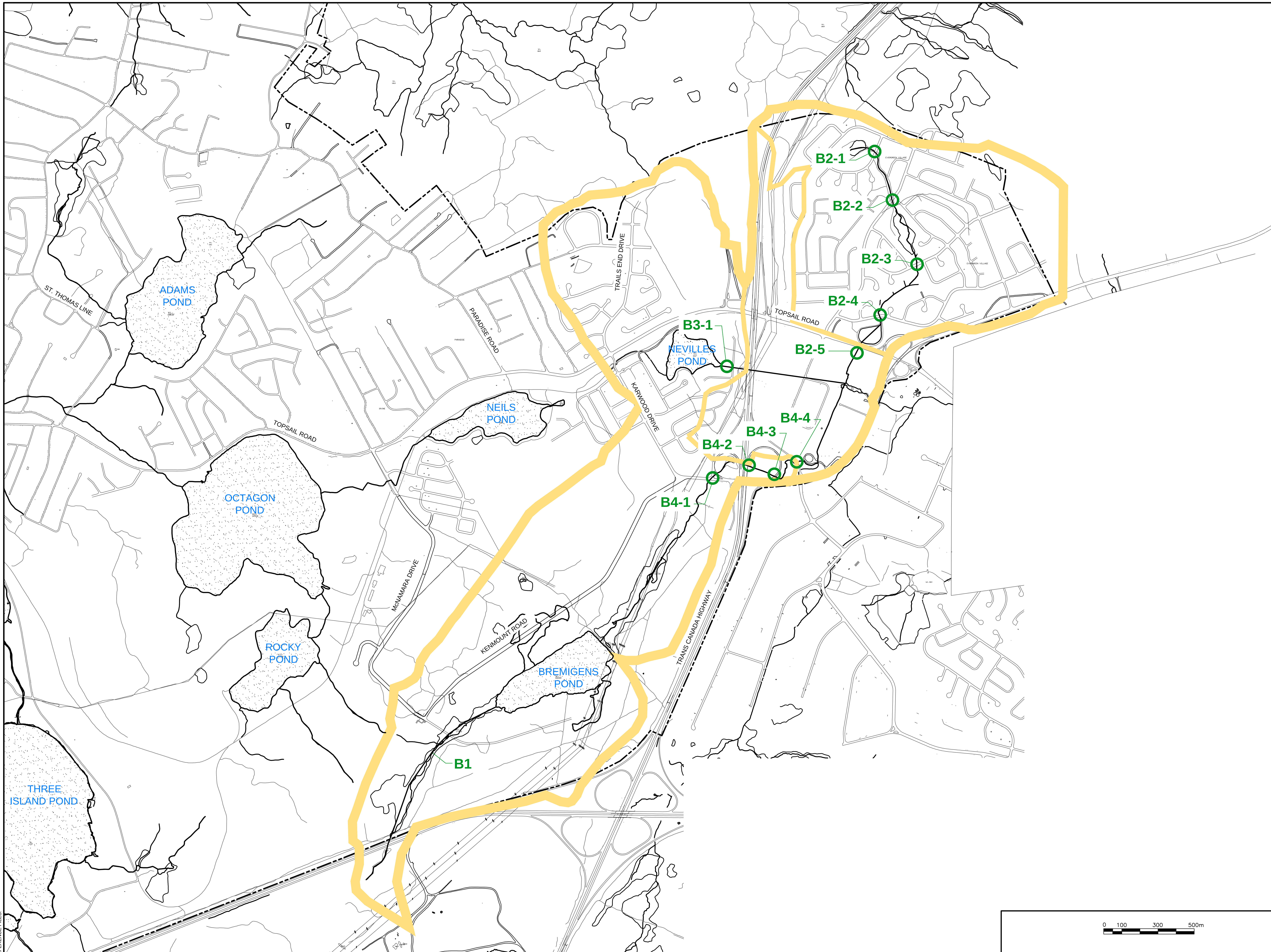
REV. D



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LEGEND

A12-3

CONTROL POINT LOCATION

RIVER NUMBER

CONTROL POINT NUMBER

TOWN BOUNDARY

REV.	REVISIONS	REVISED BY	APP. BY	DATE
B	REVISED FOR REPORT	A.E.	A.E.	12/05/15
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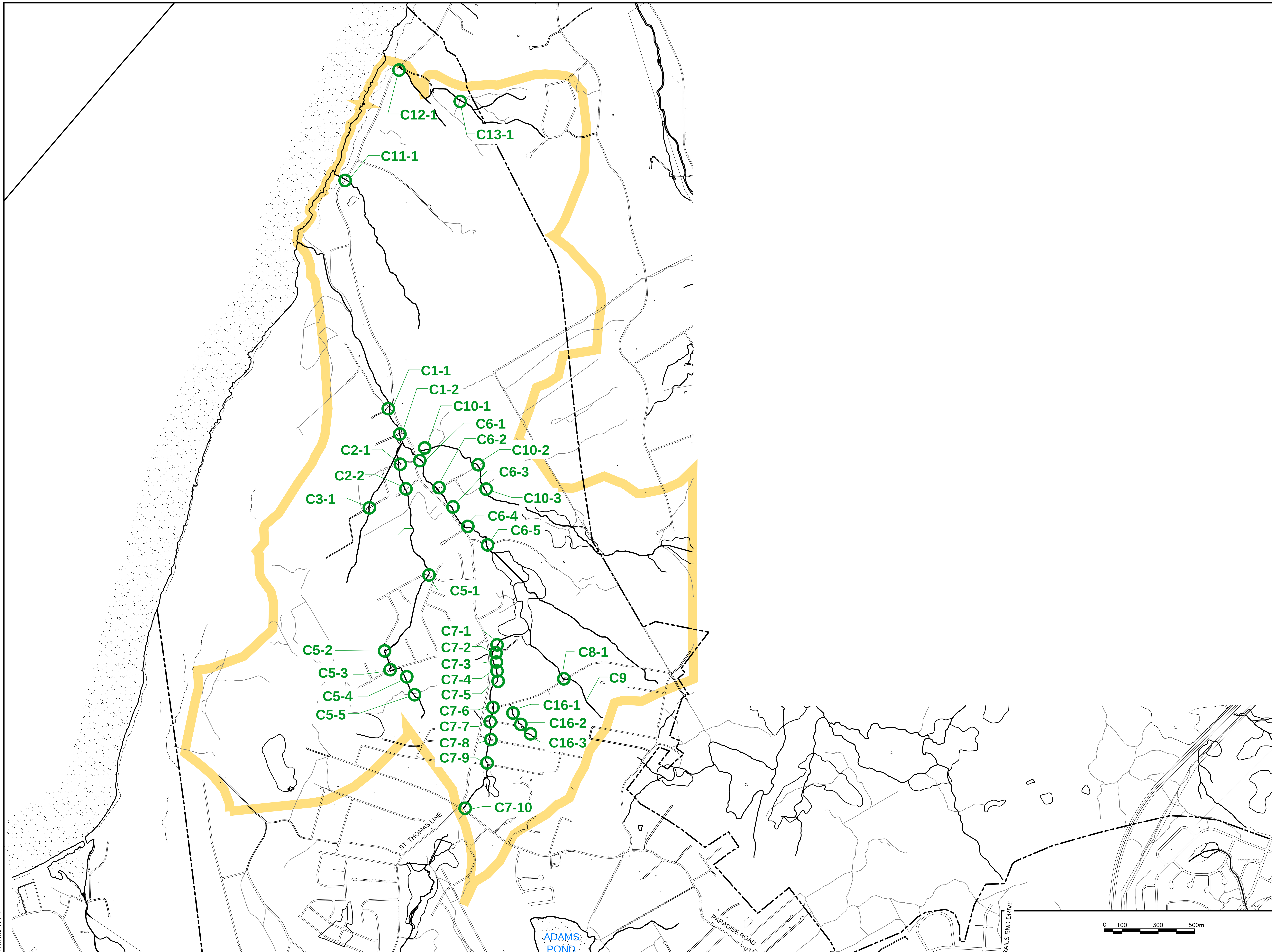
TITLE
BASIN B CONTROL POINTS

DESIGNED BY	K.B.	CHECKED BY		DATE	11/11/25
DRAWN BY	J.B.	APPROVED BY		DATE	11/11/25
SCALE	AS SHOWN	BNG PROJ. No.	723487		
		CLIENT PROJ. No.			

DRAWING No. PR 1 – XX – CV – XX – 006

REV. B

REFERENCE FILES



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LEGEND

A12-3

CONTROL POINT LOCATION

RIVER NUMBER

CONTROL POINT NUMBER

TOWN BOUNDARY

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C	REVISED FOR REPORT	A.E.	A.E.	13/07/08
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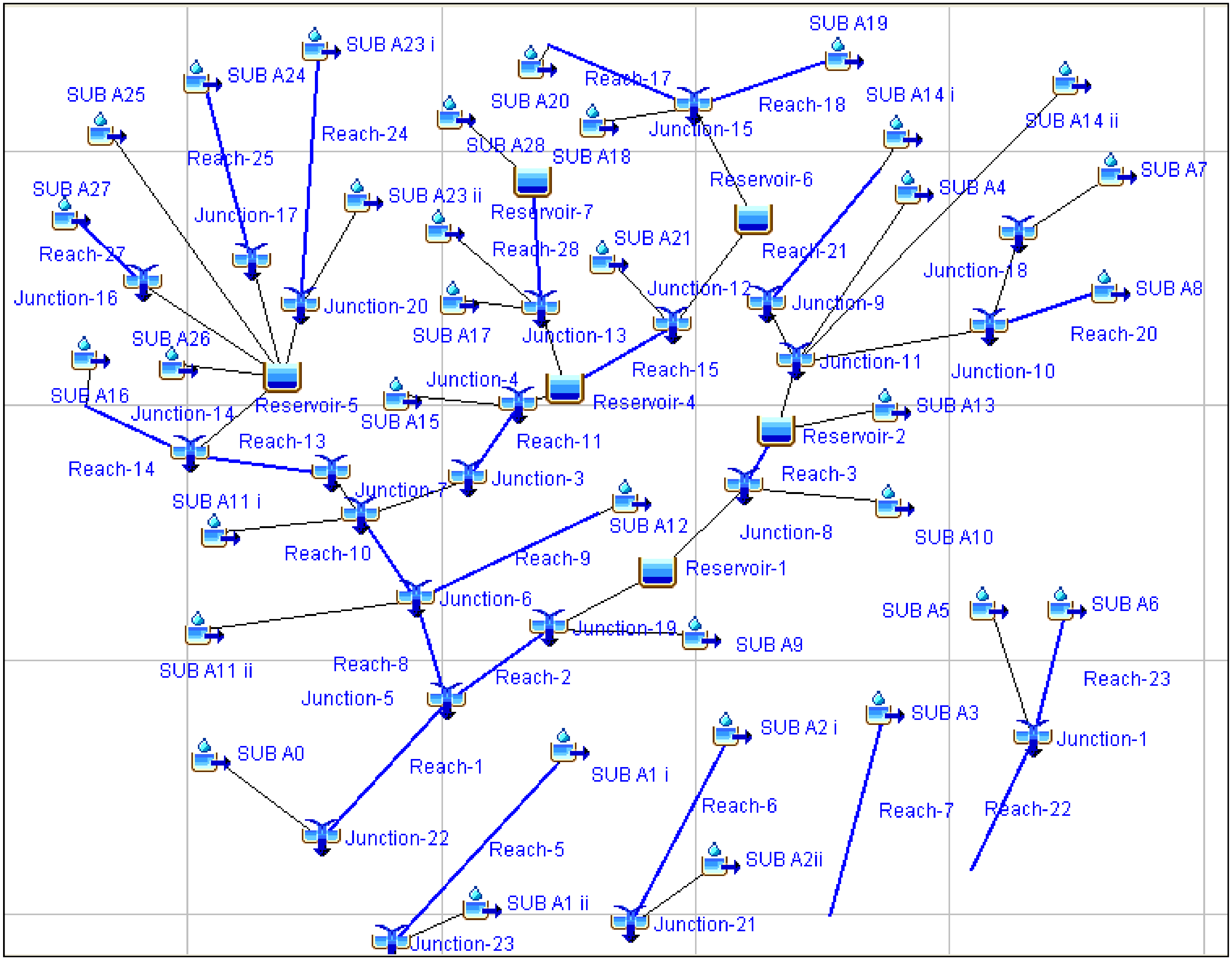
TOWN OF PARADISE NEWFOUNDLAND & LABRADOR

PROJECT
STORM WATER MANAGEMENT STUDY

TITLE
BASIN C CONTROL POINTS

DESIGNED BY	K.B.	CHECKED BY		DATE	11/11/25
DRAWN BY	J.B.	APPROVED BY		DATE	11/11/25
SCALE	AS SHOWN	BNG PROJ. No.	723487		
		CLIENT PROJ. No.			
DRAWING No.	PR 1– XX –CV–XX –007			REV.	C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Reservoir-6
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Reservoir-5
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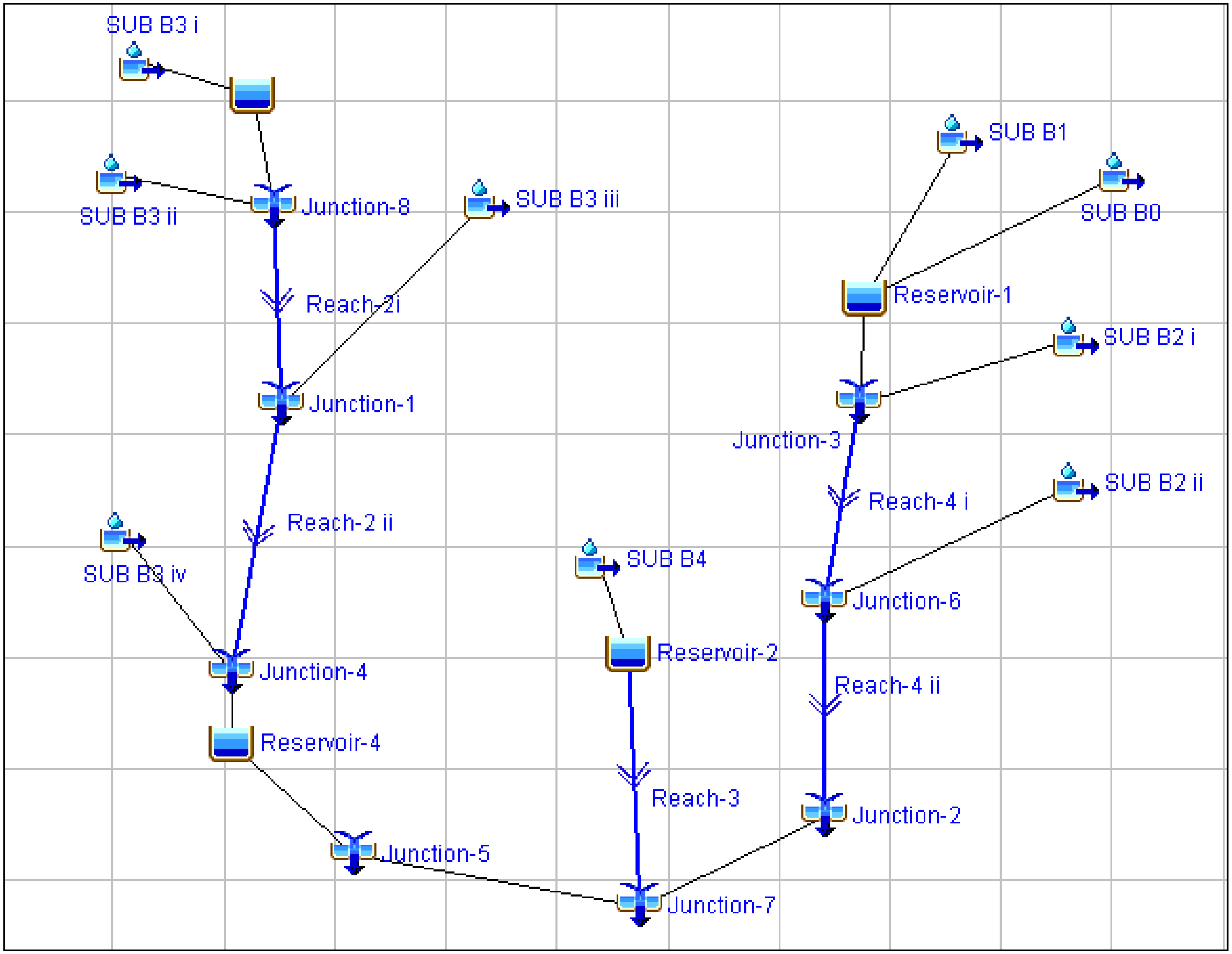
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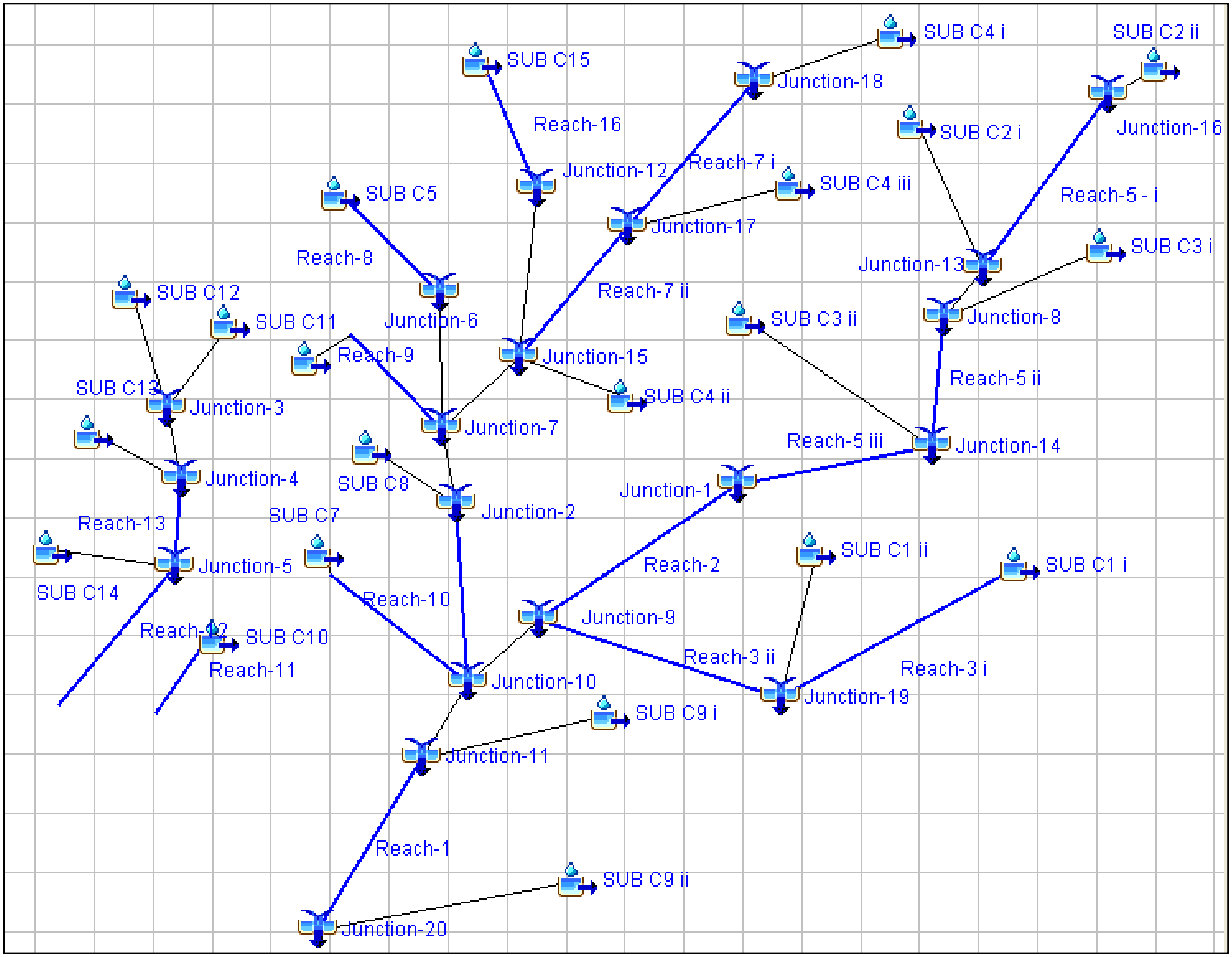

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Reach-3 i
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Junction-10
Junction-19
SUB C9 i
Junction-11
Reach-1
SUB C9 ii
Junction-20

APPENDIX C

Watershed Characteristics

Basin A Watershed Characteristics					
No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
1	Ocean at Topsail Beach	122.2	20	316.3	6.9
2	Topsail Rd Bridge near No. 1973 - Town Boundary (S)	2.0	5	29.8	5.1
3	Behind No. 1973 Topsail Rd	9.0	10	93.9	5.1
4	Penstock Crossing Behind No. 32 Topsail Pond Rd	5.7	15	69.8	6.4
5	First Bridge Behind No. 50 Topsail Pond Rd	4.0	25	36.7	7.4
6	Driveway Bridge - No. 52 Topsail Pond Rd	9.1	25	64.6	5.2
7	Topsail Pond Rd Control Structure	1.1	55	29.2	1.5
8	Topsail Pond Rd Bridge near Three Island Pond Rd	3.1	50	56.2	0.3
9	Topsail Pond	51.7	60	250.2	2.8
10	Topsail Pond Rd / Buckingham Dr Intersection	12.9	30	92.7	9.0
11	Buckingham Dr near Angel's Rd Intersection	3.9	50	57.4	6.8
12	Buckingham Dr near No. 317	8.2	50	116.9	6.8
13	Buckingham Dr River Crossing	1.6	40	42.0	4.0
14	Three Island Pond	227.9	60	557.3	5.2
15	Topsail Round Pond	54.0	50	264.7	4.4
16	Pond West of Three Island Pond	21.2	45	229.0	4.5
17	Culvert Across Peacekeeper's Way (West-1)	14.0	10	115.7	6.7
18	Culvert Across Peacekeeper's Way (West-2)	12.1	5	81.5	3.7
19	Culvert Across Peacekeeper's Way (From Three Arm Pond)	1.7	20	24.4	0.9
20	Three Arm Pond	385.4	40	621.8	3.0
21	Paddy's Pond	1142.4	45	1188.2	3.2
22	Thomas Pond	4292.8	10	2101.7	1.3
23	Cochrane Pond Campground	12.0	40	84.5	2.5
24	Cochrane Pond	1043.3	30	1295.7	2.3
25	No. 291 Three Island Pond Rd	64.9	5	256.9	4.5
26	Culvert Across Peacekeeper's Way (East)	61.9	30	173.7	7.1
27	Three Island Pond Rd near Shalloway Rd Intersection	10.9	35	67.3	7.8
28	No. 139 Three Island Pond Rd	1.2	50	25.7	6.1
29	No. 129 Three Island Pond Rd	1.5	40	35.2	3.1
30	No. 105 Three Island Pond Rd	21.0	15	102.5	7.5
31	No. 74 Three Island Pond Rd	3.5	25	48.9	11.1
32	No. 75 Three Island Pond Rd	11.4	10	81.1	11.3
33	No. 59 Three Island Pond Rd	4.3	30	58.6	7.9
34	Three Island Pond Rd near No. 28	6.3	45	65.8	7.3
35	No. 22 Topsail Pond Rd	1.9	65	56.2	10.9
36	Topsail Rd Bridge near No. 1973 - Town Boundary (SE)	1.4	65	42.4	4.6
37	Across Topsail Rd near No. 1956	5.0	35	51.5	9.5
38	Topsail Rd Bridge near No. 1973 - Town Boundary (NE)	0.2	60	12.6	12.7
39	No. 1973 Topsail Rd Bridge	6.0	30	47.1	8.8
40	No. 1960 Topsail Rd Driveway	1.1	75	34.5	6.7
41	No. 1956 Topsail Rd Driveway	0.3	65	25.6	4.7
42	No. 76 Brittany Dr Storm Sewer Outfall	8.5	55	100.0	11.8
43	Newdale Rd Bridge	1.0	50	31.0	8.1
44	Sedgewick St Storm Sewer Outfall	6.9	45	66.6	5.6
45	Sedgewick St River Crossing	3.0	20	44.9	10.5
46	Cuvlert across Topsail Rd at No. 1897	0.3	35	18.0	5.7
47	Culvert behind No. 1897 Topsail Rd (1)	0.3	65	15.3	2.4
48	Culvert behind No. 1897 Topsail Rd (2)	0.9	55	24.3	7.6
49	No. 1895 Topsail Rd Driveway	1.9	50	42.4	6.9
50	No. 10 Spracklin Blvd Storm Sewer Outfall	2.6	90	57.9	5.6

Basin A Watershed Characteristics					
No.	**Drainage Point**	**Area (ha)**	**Impervious (%)**	**Width (m)**	**Slope (%)**
51	No. 10 Spracklin Blvd Culvert	5.4	40	60.2	5.1
52	Spracklin Blvd Road Stub	1.8	95	43.4	4.2
53	Greenfields Pl near Spracklin Blvd	13.8	70	103.4	9.7
54	Spracklin Blvd / Ryder Pl Intersection	13.8	80	185.7	7.7
55	Topsail Rd near No. 1895	1.4	70	40.2	5.1
56	Topsail Rd near Twin Brooks Dr (W)	11.8	60	71.9	6.0
57	Culvert alongside Topsail Rd at No. 1904	6.4	70	51.3	8.1
58	Bridge across No. 1904 Topsail Rd Driveway	10.9	40	75.5	8.1
59	Summit Dr / Brittany Dr Intersection	7.3	60	64.0	11.6
60	Culvert across Brittany Dr	1.8	30	38.3	4.8
61	Summit Dr near No. 55	5.4	55	56.4	7.4
62	Summit Dr / Liberty Ln Intersection	4.4	80	35.7	9.3
63	Summit Dr near No. 107	13.7	65	83.3	10.6
64	Liberty Ln / Ridgewood Dr Intersection	2.0	85	27.9	8.3
65	Pinehill Pl River Crossing	10.6	50	93.0	5.3
66	Confluence behind No. 9 St. Thomas Line	7.7	80	40.4	11.6
67	No. 102 St. Thomas Line	1.4	75	55.5	6.2
68	Culvert across St. Thomas Line near No. 9 (S)	3.1	55	43.5	1.7
69	Hampton Pl Storm Sewer Outlet	3.3	90	121.8	10.4
70	Carlingford St River Crossing near No. 5	2.9	65	62.2	5.9
71	Sgt Donald Lucas Dr near Topsail Rd	9.4	85	107.6	4.1
72	Bridge under Topsail Rd near Christopher St Intersection	3.1	80	77.0	6.3
73	Octagon Pond	119.1	60	469.2	4.3
74	Rocky Pond	94.8	35	338.1	4.4
75	T'Railway behind Paradise Rec Centre (W)	11.8	80	136.1	6.7
76	McNamara Dr near No. 131	10.3	95	174.4	6.7
77	Octagon Pond East Inlet (S)	8.0	75	100.4	3.9
78	T'Railway behind Paradise Rec Centre (E)	8.4	85	187.8	4.8
79	McNamara Dr near No. 107	7.5	75	132.7	7.0
80	Octagon Pond East Inlet (N)	4.2	65	102.3	6.4
81	Paradise Town Hall Parking Lot	1.0	90	53.4	2.4
82	Rectangular Concrete Culvert behind Paradise Town Hall	1.0	64	34.7	9.0
83	Culvert next to Paradise Town Hall	1.1	70	32.9	3.6
84	Culvert across McNamara Dr near Burnaby St	2.1	65	52.9	3.4
85	Culvert across T'Railway near McNamara Dr	9.6	65	72.7	9.6
86	Burnaby St Storm Sewer Outfall	21.7	70	213.9	3.7
87	Burnaby River Crossing	2.2	80	46.2	2.8
88	Culvert across T'Railway behind Croydon St	0.5	25	28.7	1.6
89	Neil's Pond	54.0	70	184.2	5.7
90	Yellow Wood Dr Storm Sewer Outfall	15.1	65	103.6	8.0
91	Topsail Rd Outfall near No. 1662	30.5	75	227.8	7.5
92	Topsail Rd Outfall near Glenderek Dr	7.9	75	158.9	6.6
93	Culvert across St. Thomas Line near No. 9 (N)	0.2	65	20.0	2.0
94	Carlingford St Storm Sewer Outfall	5.7	75	71.1	4.9
95	Carlingford St River Crossing near No. 38	5.5	55	61.9	3.9
96	Nicholas Pl Storm Sewer Outfall	8.4	80	122.3	4.1
97	Adams Pond	105.0	70	347.1	1.6
98	Lanark Dr Storm Sewer Outfall near No. 11	6.3	95	83.9	2.4
99	Culvert across Lanark Dr near No. 11	1.6	75	46.8	1.1
100	Windmill Rd	2.6	80	74.1	3.0

Basin A Watershed Characteristics

No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
101	Copper Canyon Cl Storm Sewer Outfall	7.1	95	115.7	6.0
102	Copper Canyon Cl Culvert	24.0	50	97.1	2.8
103	No. 81 St. Thomas Line Outfall	25.7	80	168.1	4.2
104	Ridgewood Dr / Vambury St Intersection Ditch Inlet	2.7	70	33.0	11.7
105	Crimson St Storm Sewer Outfall	1.4	85	59.9	8.5
106	Acharaya Dr Storm Sewer Outfall	3.4	90	73.5	3.1
107	Tyrell Dr / St. Thomas Line Intersection	10.9	85	124.7	3.9
108	Camelot Cres Playground	5.9	20	63.9	8.2
109	Westport Dr Storm Sewer Outfall near St. Thomas Line	7.3	85	69.3	4.5
110	Ditch Inlet behind No. 12 Westport Dr	18.3	50	115.7	4.4
111	Plateau Park Storm Sewer Outfall near No. 65	10.4	75	102.2	7.1
112	Ditch near No. 74 Ashlin Cres	4.5	55	54.0	7.0
113	Westport Dr / St. Thomas Line Intersection	3.9	60	45.1	2.4
114	Lanark Dr Storm Sewer Outfall near Mountaineer Dr	4.0	90	71.2	5.9
115	Lanark Dr Storm Sewer Outfall near Rembrant Blvd	9.4	85	75.1	4.5
116	Adams Pond Inlet (E)	29.4	55	111.8	7.0
117	Fred W Brown Dr Storm Sewer Outfall	36.9	65	175.4	6.3
118	Kilbum Dr Storm Sewer Outfall	4.0	85	76.6	8.2
119	Lanark Dr Storm Sewer Outfall near Marble Ave	3.2	95	96.4	3.9
120	Mt Sylvester Pl Storm Sewer Outfall	0.6	95	42.1	4.3
121	Lanark Dr Storm Sewer Outfall near Glenderek Dr	21.9	90	275.7	5.0
122	Lanark Dr Storm Sewer Outfall near Blue Jay Pl	4.0	95	58.3	4.8

A	Area-Weighted Average	70.5	27	1444.2	2.6
A	Total	8596.0			

Basin B Watershed Characteristics					
No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
1	Waterford River under Kenmount Rd	9.1	65	137.5	0.2
2	Ditch Inlet at Kinsdale Rd / Wynnford Dr Intersection	1.7	85	35.0	1.5
3	Kinsdale Rd Catch Basin	1.4	40	62.6	2.0
4	Channel between St. Anne's Cres / Kinsdale Rd	7.3	80	109.2	0.9
5	Culvert at NL Highway Maintenance Depot	1.8	50	40.9	2.5
6	Culvert at Outer Ring Rd Eastbound Onramp	2.6	10	33.9	1.9
7	Culverts across Kenmount Rd from Donovan's Industrial Park	30.7	65	245.5	4.4
8	Culverts across Kenmount Rd from Outer Ring Road Ditch	21.6	50	80.7	6.0
9	Culvert inside Eastbound Outer Ring Rd / Kenmount Rd Offramp	3.1	15	61.1	4.2
10	Culvert inside Westbound Outer Ring Rd / Kenmount Rd Onramp	2.2	25	43.6	4.3
11	Kenmount Rd Culvert near NL Hydro Switchyard	50.0	20	200.4	3.6
12	Bremigan's Pond Dam Outlet	93.5	60	359.4	2.5
13	Bremigan's Pond Inlet	34.9	60	288.0	5.6
14	Bremigen's Blvd Culvert	59.9	20	207.8	4.5
15	Culvert under Westbound Outer Ring Rd / Kenmount Rd Offramp	3.0	40	84.7	8.4
16	Roundabout - Karwood Dr	2.3	90	64.5	10.1
17	Roundabout - Kenmount Rd	16.0	60	81.1	10.0
18	Culvert under St. Anne's Cres - SE	4.9	95	86.8	0.1
19	Culvert under St. Anne's Cres - SW	4.6	90	99.4	0.3
20	Culvert under St. Anne's Cres - NW	2.8	95	70.1	1.0
21	Culvert under St. Anne's Cres - NE	1.2	90	58.2	0.5
22	Culvert into St. Anne's Ind. Park from SW	4.2	40	51.3	2.9
23	Culvert under St. Anne's Industrial Park	7.5	50	99.8	5.0
24	Culvert across Outer Ring Rd near Neville's Pond	8.6	15	85.2	4.7
25	Bridge under Hollyberry Dr	7.7	60	98.1	3.5
26	Neville's Pond Outlet	10.2	55	114.6	0.9
27	Neville's Pond Inlet at Sanderling Pl	0.4	30	36.2	7.4
28	Sanderling Pl - East	0.8	70	45.4	1.3
29	Sanderling Pl - West	2.5	70	81.8	3.1
30	Cloudberry Dr to Sanderling Pl	0.3	70	31.4	4.0
31	Kestrel Dr to Sanderling Pl	0.6	70	56.6	3.6
32	Cloudberry Dr/Kestrel Dr Intersection	0.5	70	39.3	7.6
33	Hollyberry Dr to Cloudberry Dr	0.2	70	34.1	5.6
34	Hollyberry Dr	0.2	70	28.9	5.4
35	Hollyberry Dr / Hudsonberry Dr	0.6	70	45.5	5.3
36	Karwood Dr / Hudsonberry Dr	0.4	70	24.9	3.3
37	Paradise Elementrary Parking Lot	0.8	70	48.6	2.9
38	Cloudberry Dr before Kestrel Dr	2.3	60	92.2	8.1
39	Paradise Elementrary Headwall	14.8	15	83.4	7.8
40	Karwood Dr/Cloudberry Dr	1.4	75	68.0	4.3
41	Neville's Pond Inlet - West	1.9	60	54.2	5.1
42	Manhole at bottom of Cormorant Pl	1.3	50	55.7	5.4
43	Ditch into Neville's Pond Inlet - West	2.0	50	42.8	6.4
44	Karwood Dr / Kestrel Dr Intersection	0.1	95	20.1	1.4
45	Karwood Dr South of Kestrel Dr	0.8	45	48.6	6.9
46	Goldfinch Dr West of Karwood Dr	0.3	50	38.5	2.0
47	Goldfinch Dr / Flamingo Dr	0.4	70	52.3	2.0
48	Flamingo Dr South of Goldfinch Dr	0.5	70	50.7	9.5
49	Flamingo Dr from Crane St	0.4	70	39.3	7.0
50	Crane St from Karwood Dr	0.4	70	48.3	4.4

Basin B Watershed Characteristics					
No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
51	Crane St / Karwood Dr	0.2	70	26.8	9.5
52	Hummungbird Rd/Karwood Dr	0.5	70	37.5	4.7
53	South of Crane St	0.5	70	50.0	9.2
54	Flamingo Dr from Goldfinch Dr	0.2	70	25.1	3.2
55	Flamingo Dr / Golfinch Dr	0.4	70	43.3	6.8
56	Goldfinch Dr / Mockingbird Dr	0.4	70	41.4	7.0
57	Between Topsail Rd / Goldfinch Dr	2.9	70	50.5	3.4
58	Goldfinch Dr Outlet	0.1	70	13.3	2.8
59	Mockingbird Dr - A	0.2	70	32.3	1.4
60	Mockingbird Dr - B	0.4	70	46.1	3.4
61	Mockingbird Dr - C	0.5	70	53.8	8.5
62	Mockingbird Dr - D	0.7	70	65.1	9.4
63	Mockingbird Dr - E	0.2	70	36.8	7.4
64	Goldfinch Dr Outlet	0.1	70	13.3	2.7
65	Goldfinch Dr West - A	0.2	70	25.9	2.3
66	Goldfinch Dr West - B	0.4	70	35.5	3.5
67	Goldfinch Dr West - C	0.3	70	37.7	9.7
68	Goldfinch Dr West - D	0.5	70	53.0	9.9
69	Goldfinch Dr West - E	0.4	70	47.6	7.8
70	Manhole in Karwood Dr / Topsail Rd Intersection - West	1.3	60	40.6	5.5
71	Manhole at Bottom of Donna Rd	5.9	60	82.4	8.5
72	Manhole in Trails End Dr / Topsail Rd Intersection - North	10.4	60	122.3	6.0
73	Culvert into Neville's Pond from Topsail Rd - North	2.5	60	49.1	3.3
74	Aurora Pl	17.7	65	165.4	6.2
75	South of Outer Ring Rd / Topsail Intersection	2.1	25	78.1	5.8
76	Between Neville's Pond and Topsail Rd	3.7	5	82.2	2.7
77	Topsail Rd in front of Irving	3.1	100	61.7	2.6
78	Ditch Inlet on Topsail Rd near Maloney's RV - West	6.2	25	54.4	3.9
79	Culvert near Maloney's RV off Topsail Rd	4.7	10	56.4	8.7
80	Manhole at end of Milton Rd	13.5	50	185.0	6.1
81	Ditch Inlet on Topsail Rd near Maloney's RV - East	1.4	15	22.7	4.5
82	Topsail Rd east of Outer Ring Rd	3.9	90	98.2	2.1
83	Ditch Inlet on Topsail Rd near Scotiabank	1.4	25	20.0	3.5
84	Culvert alongside Topsail Rd near Outer Ring Rd Eastbound Onramp - East	5.0	10	35.4	7.6
85	Culvert alongside Topsail Rd near Outer Ring Rd Eastbound Onramp - West	46.0	35	193.1	5.6
86	Ditch Inlet on Topsail Rd near Elizabeth Dr	3.2	15	29.5	7.2
87	Manhole at Iris Pl / Elizabeth Dr Intersection	24.3	60	244.6	4.3
88	Pennecon Culvert - Catch basin in parking lot	1.0	100	67.4	2.5
89	Pennecon Culvert -Manhole at Topsail Rd	3.0	100	86.1	0.5
90	Detention Pond on Topsail Rd near Canterbury Dr	8.6	15	44.4	3.2
91	Canterbury Dr Bridge - West	8.0	60	93.0	3.4
92	Elgin Park	4.4	10	107.6	5.0
93	Cardiff Pl Culvert Outlet	1.9	60	52.0	4.3
94	Ellesmere Ave Bridge - West	3.3	60	76.4	1.9
95	Ellesmere Ave Bridge - North (Between Horncastle/Dungarvan)	4.3	10	68.0	6.7
96	Carlisle Dr Bridge - From Storm Sewer	10.8	60	111.0	3.1
97	Carlisle Dr Bridge - From Channel	2.9	15	43.2	7.3
98	Stephanie Ave Culvert - South	2.6	60	76.3	2.4
99	Stephanie Ave Detention Pond	3.7	40	80.9	1.7
100	Stephanie Ave Culvert into Pond	13.6	50	275.0	2.7

Basin B Watershed Characteristics

No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
101	Stephanie Ave Culvert - North	1.6	60	56.7	1.2
102	North of Stephanie Ave	1.8	10	107.2	8.1
103	Gervase Pl Culvert Outlet	16.8	45	137.8	3.1
104	Gainsborough Pl Culvert Outlet	0.5	60	39.8	5.2
105	Clevedon Cres Manhole	23.8	50	203.3	3.2
106	Canterbury Dr Bridge - East	24.9	60	171.7	3.7
107	Kenmount Rd NE	49.0	10	162.4	5.5
108	Culvert between Kenmount Rd Westbound Offramp and Topsail Rd	1.8	40	44.4	4.6

B	Area-Weighted Average	7.2	45	178.6	4.3
B	Total	780.1			

Basin C Watershed Characteristics					
No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
1	Horse Cove Brook Outlet at Ocean	7.3	10	62.6	11.4
2	Topsail Rd Bridge near No. 1973 - Town Boundary (S)	2.0	5	29.8	5.1
3	Whelan's Cres River Crossing near St. Thomas Line	7.4	10	42.1	13.4
4	Squire's Rd River Crossing	28.4	10	119.7	7.6
5	Neary Rd River Crossing	2.3	15	35.6	9.9
6	Father Lacey Pl River Crossing	29.1	5	144.6	8.9
7	Seascape Dr Storm Sewer Outfall	15.8	50	140.9	7.2
8	Culvert across Seascape Dr near No. 29	5.6	25	71.1	11.0
9	Culvert across Howard Ave near No. 16	0.3	60	18.7	3.4
10	Culvert across Morgan Ave near No. 4	1.9	20	39.8	7.0
11	Culvert alongside Howard Ave near No. 2 (W)	1.9	40	26.4	4.9
12	Culvert across Picco Dr near Howard Ave Intersection (W)	69.1	5	265.8	4.2
13	Culvert alongside Howard Ave near No. 2 (E)	0.2	70	13.8	9.2
14	Culvert across Picco Dr near Howard Ave Intersection (E)	6.4	5	66.7	5.5
15	Ivydale Pl Storm Sewer Outfall	13.4	65	142.6	7.3
16	Culvert across Neary Rd at St. Thomas Line Intersection	9.2	45	79.3	10.8
17	Culvert across St. Thomas Line near No. 450 (N)	1.2	30	27.0	8.1
18	Culvert across St. Thomas Line near No. 442 (N)	6.4	5	56.2	12.0
19	Wetlands behind O'Brien's Way	7.6	5	93.8	8.9
20	Culvert across St. Thomas Line near No. 442 (S)	3.9	10	51.3	6.5
21	O'Brien's Way River Crossing	6.9	5	53.8	9.3
22	Wetlands North of Lawlor Rd	40.8	10	223.6	3.7
23	Wetlands East of Lawlor Rd	8.8	5	78.7	6.1
24	Culvert across No. 394 St. Thomas Line Driveway	3.9	5	43.0	7.4
25	Culvert across No. 380 St. Thomas Line Driveway	3.4	15	48.6	9.0
26	Lawlor Rd River Crossing	8.4	10	86.2	3.9
27	Wetlands between Deborah Lynn Heights and Lawlor Rd (E)	21.0	15	128.3	5.2
28	Wetlands near Deborah Lynn Heights (E)	15.4	30	145.7	7.7
29	Wetlands between Deborah Lynn Heights and Lawlor Rd (W)	23.5	10	172.7	3.7
30	Culvert across St. Thomas Line near Raymond's Ln	5.5	25	86.6	10.9
31	St. Thomas Line near No. 273	0.5	20	28.7	13.0
32	St. Thomas Line near No. 271	3.2	10	61.2	17.3
33	St. Thomas Line near Deborah Lynn Heights Intersection	2.1	15	49.7	16.4
34	St. Thomas Line near Quilty's Rd Intersection	2.0	25	44.6	18.2
35	St. Thomas Line near Byrne's Rd Intersection	3.8	35	80.8	17.9
36	St. Thomas Line near No. 205	6.6	30	123.8	8.6
37	Raymond's Ln River Crossing	0.5	20	25.4	4.2
38	Culvert across No. 292 St. Thomas Line Driveway	0.2	25	17.2	10.9
39	Culvert across No. 290 St. Thomas Line Driveway	0.4	25	19.7	8.5
40	Culvert across No. 282 St. Thomas Line Driveway	1.5	5	38.1	7.5
41	Culvert across Johnathon Dr	3.0	20	88.7	13.3
42	Culvert across Deborah Lynn Heights near St. Thomas Line (W)	0.9	35	52.6	3.2
43	Culvert across Quilty's Rd	2.3	40	50.5	12.0
44	Culvert across Byrne's Rd	4.5	35	81.1	11.9
45	Culvert across Hickey's Rd	6.7	25	92.9	8.5
46	Wetlands between Paradise Rd and Hickey's Rd (W)	1.6	15	63.1	6.5
47	Culvert across Paradise Rd near St. Thomas Line Intersection	3.1	15	76.4	10.2
48	Wetlands between Paradise Rd and Hickey's Rd (E)	6.3	15	122.8	6.5
49	Culvert across Paradise Rd near No. 359	0.7	25	37.2	6.0
50	Culvert across Deborah Lynn Heights near St. Thomas Line (E)	15.7	10	106.1	7.5

Basin C Watershed Characteristics

No.	Drainage Point	Area (ha)	Impervious (%)	Width (m)	Slope (%)
51	Culvert across Deborah Lynn Heights near No. 57	9.1	10	133.1	12.3
52	Cuvlert across St. Thomas Line near Whelan's Cres	25.8	5	92.0	8.5
53	Cuvlert across St. Thomas Line near No. 494	11.3	5	57.5	6.5
54	Cuvlert across St. Thomas Line near No. 544	2.8	20	49.0	3.5
55	Cuvlert across St. Thomas Line near No. 644	6.1	20	47.3	10.7
56	Cuvlert across St. Thomas Line near No. 650	17.9	5	85.0	7.4
57	Atlantica Dr Storm Sewer Outfall	5.3	85	116.8	5.0
58	Cuvlert across Atlantica Dr	69.7	5	221.2	10.7
59	Cuvlert across St. Thomas Line near No. 660	9.5	25	39.2	10.5
60	Cuvlert across St. Thomas Line near Stapleton's Rd Intersection	34.1	10	151.2	11.6
61	Cuvlert across St. Thomas Line near No. 676	2.2	15	25.5	9.9
62	Cuvlert across St. Thomas Line near No. 684	4.4	25	43.6	13.9
63	Cuvlert across St. Thomas Line near Moonlight Dr Intersection	28.5	20	102.3	13.7

C	Area-Weighted Average	10.3	14	140.9	8.2
C	Total	649.1			

APPENDIX D

Hydraulic Structures and Proposed Improvements

Basin A - Hydraulic Structures and Proposed Improvements											
No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
A-1	River Confluence near No. 1973 Topsail Rd	Concrete Bridge	3000	1600	8.70	y	15.17	y	Increase bridge to 5.5m x 1.6m	9.00	n
A-2	No. 22 Topsail Pond Rd	CMP Culvert	580	580	0.24	n	0.25	n	N/A	0.25	n
A-3	Penstock Crossing near Topsail Pond Rd	Penstock Crossing	6100	1700	5.79	n	8.99	n	N/A	8.99	n
A-4	No. 64 Topsail Pond Rd	Concrete Bridge	2300	1200	5.79	n	8.99	y	Private driveway: 5.5 x 1.2 bridge	8.99	n
A-5	Topsail Pond Rd near Three Island Pond Rd	CMP Open Arch	4350	1820	8.79	n	11.99	n	N/A	11.99	n
A-6	Topsail Pond Outlet	Concrete Bridge	5500	2300	8.79	n	11.99	n	N/A	11.99	n
A-7	Topsail Pond Outlet	CMP Open Arch	5890	1520	8.79	n	11.99	n	N/A	11.99	n
A-9	Buckingham Dr near Angel's Rd	CPP Culvert	750	750	0.38	n	0.52	n	N/A	0.52	n
A-10	No. 317 Buckingham Dr	CMP Culvert	600	600	0.73	y	1.04	y	2.0 x 0.6 Concrete Box	1.01	n
A-11	No. 28 Three Island Pond Rd	CMP Culvert	600	600	0.42	y	0.63	y	600 HDPE Culvert	0.63	n
A-12	No. 59 Three Island Pond Rd	Concrete Culvert	530	530	0.29	n	0.44	y	600 HDPE Culvert	0.44	n
A-13	No. 75 Three Island Pond Rd	CMP Culvert	640	640	0.70	y	1.08	y	2.0 x 0.6 Concrete Box	1.08	n
A-14	No. 74 Three Island Pond Rd	CMP Culvert	457	457	0.22	n	0.34	y	600 HDPE Culvert	0.34	n
A-15	No. 105 Three Island Pond Rd	CMP Culvert	600	600	1.01	y	1.60	y	2.3 x 0.6 Concrete Box	1.60	n
A-16	No. 129 Three Island Pond Rd	CMP Culvert	600	600	0.11	n	0.17	n	N/A	0.17	n
A-17	No. 139 Three Island Pond Rd	CMP Culvert	560	430	0.11	n	0.16	n	N/A	0.16	n
A-18	Three Island Pond Rd near Shalloway Rd	CMP Culvert	600	600	0.63	y	0.92	y	2.0 x 0.6 Concrete Box	0.92	n
A-19	Buckingham Dr River Crossing	2x CMP Culverts	2000	1500	8.90	n	12.49	n	N/A	12.49	n
A-20	Topsail Round Pond Outlet	CMP Culvert	800	800	0.64	n	0.84	n	N/A	0.84	n
A-21	Pond West of Three Island Pond Outlet	2x CPP Culverts	460	460	0.43	n	0.54	n	N/A	0.54	n
A-22	Pitts Memorial Dr	CMP Culvert	600	600	0.48	n	0.50	n	N/A	0.50	n
A-23	Pitts Memorial Dr	CMP Culvert	600	600	0.56	n	0.60	n	N/A	0.60	n
A-24	No. 291 Three Island Pond Rd	CMP Culvert	1200	1200	2.24	n	3.68	y	2.0 x 1.2 Concrete Box	3.67	n
A-25	No. 315 Three Island Pond Rd	CMP Culvert	750	750	0.17	n	0.18	n	N/A	0.18	n
A-26	Pitts Memorial Dr	CMP Culvert	600	600	0.35	n	0.37	n	N/A	0.37	n
A-27	Pitts Memorial Dr River Crossing	CMP Open Arch	4570	2740	10.87	n	14.33	n	N/A	14.33	n
A-28	Cochrane Pond Campground	L: Wood Box R: CMP Culvert	1200 2030	1200 1220	2.48	n	4.12	n	Not Examined [1]	4.12	n
A-29	Cochrane Pond Campground	L: CMP Culvert R: CMP Culvert	1780 1120	910 970	2.48	n	4.08	y	Not Examined [1]	4.08	y
A-30	No. 1973 Topsail Rd	Steel/Wood Bridge	3400	2470	17.31	y	21.60	y	Private driveway: 7.5 x 2.0 Concrete Box	28.97	n
A-31	No. 1960 Topsail Rd	L: CMP Culvert R: CMP Culvert	1800 1600	1300 1400	17.00	y	21.48	y	7.5 x 2.0 Concrete Box	28.48	n
A-32	No. 1956 Topsail Rd	Concrete Bridge	3600	1000	16.91	y	25.80	y	7.5 x 2.0 Concrete Box	28.36	n
A-33	Crossing Topsail Rd near No. 1956 Topsail Road	CMP Culvert	600	600	0.63	y	1.76	y	Flow re-directed by other improvements	0.57	n
A-34	Newdale Rd River Crossing	2x CMP Culverts	2000	1400	17.81	y	24.46	y	7.5 x 2.0 Concrete Box	27.89	n
A-35	Sedgewick St River Crossing	CMP Open Arch	7130	2000	16.40	n	22.11	n	N/A	25.59	n
A-36	No. 1899 Topsail Rd	2x CMP Culvert	900	900	2.82	n	3.13	y	4.0 x 1.2 Concrete Box	7.30	n

Basin A - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
A-37	No. 1899 Topsail Rd Yard	Concrete Box	1600	1000	4.52	y	7.40	y	4.0 x 1.3 Concrete Box	7.78	n
A-38	No. 1899 Topsail Rd Yard	CMP Culvert	900	900	4.41	y	7.20	y	4.0 x 1.3 Concrete Box	7.55	n
A-39	No. 1895 Topsail Rd Parking Lot	CMP Culvert	1200	1200	3.65	y	5.78	y	1.8 x 1.7 Concrete Box	6.01	n
A-40	Spracklin Blvd	CMP Culvert	1200	1200	3.68	y	5.34	y	2.0 x 1.5 Concrete Box	5.50	n
A-41	Greenfield's Pl	CPP Culvert	1500	1500	3.22	n	4.62	y	1.8 x 1.5 Concrete Box	4.72	n
A-42	No. 1904 Topsail Rd	CMP Culvert	900	900	0.85	n	2.67	y	Flow re-directed by other improvements	0.98	n
A-43	No. 1904 Topsail Rd	Concrete Bridge	6230	1420	12.58	n	16.05	n	N/A	17.14	n
A-44	Brittany Dr	CMP Culvert	600	600	0.13	n	0.19	n	N/A	0.23	n
A-45	Summit Dr at Liberty Ln	CPP Culvert	750	750	0.29	n	0.39	n	N/A	0.39	n
A-46	Ridgewood Dr at Liberty Ln	CPP Culvert	750	750	0.30	n	0.40	n	N/A	0.40	n
A-47	Pinehill Pl	CMP Open Arch	4700	1170	9.39	n	13.36	y	5.5 x 1.4 Concrete Box	13.56	n
A-48	St. Thomas Line near No. 9	L: CMP Culvert R: CMP Culvert	1900 1850	1270 1280	5.52	n	9.09	y	5.0 x 1.3 Concrete Box	9.71	n
A-49	Carlingford Dr East	CMP Open Arch	2430	1700	5.32	n	8.82	y	2.6 x 1.7 Concrete Box	9.40	n
A-50	Topsail Rd near Christopher St	Concrete Bridge	6040	2290	5.30	n	8.77	n	N/A	9.35	n
A-51	Path South of Paradise Rec Centre	CMP Culvert	1000	1000	2.83	y	3.79	y	1.6 x 1.4 Concrete Box	3.81	n
A-52	McNamara Dr near No. 131	Concrete Culvert	1000	1000	1.56	n	2.05	n	N/A	2.05	n
A-53	Octagon Pond East Inlet	2x CMP Culvert	600	600	6.44	y	10.19	y	Foot bridge	12.42	n
A-54	Road behind Paradise Rec Centre	CMP Culvert	1470	940	1.93	y	2.63	y	1.8 x 1.2 Concrete Box	2.82	n
A-55	Paradise Rec Centre	CMP Culvert	1200	1200	1.04	n	1.38	n	N/A	1.38	n
A-56	Paradise Rec Centre	CMP Culvert	1200	1200	1.04	n	1.38	n	N/A	1.38	n
A-57	McNamara Dr near No. 107	CMP Culvert	1000	1000	1.05	n	1.39	n	N/A	1.39	n
A-58	Neil's Pond Brook near Town Hall Parking Lot	CPP Culvert	800	800	3.27	y	5.00	y	Open Channel	6.51	N/A
A-59	Neil's Pond Brook rear of Town Hall Parking Lot	CMP Culvert	800	800	0.26	n	0.37	n	Open Channel	0.34	N/A
A-60	Neil's Pond Brook near NW corner of Town Hall Parkir	Concrete Box	1300	870	3.53	y	5.37	y	Open Channel	6.85	N/A
A-61	Neil's Pond Brook north of Town Hall Parking Lot	Concrete Culvert	900	900	3.47	y	5.51	y	Open Channel	6.71	N/A
A-62	McNamara Dr River Crossing	2x CMP Culverts	900	900	4.29	y	6.10	y	6.0 x 1.6 Concrete Box	6.56	n
A-63	T'Railway at McNamara Dr	CPP Culvert	760	760	0.96	n	1.33	n	N/A	1.33	n
A-64	Burnaby St	CMP Culvert	1200	1200	1.07	n	1.41	n	N/A	2.09	n
A-65	Downstream of Neil's Pond Outlet	CMP Open Arch	1370	740	1.15	n	1.28	y	1.4 x 1.0 Concrete Box	2.04	n
A-66	Neil's Pond Outlet	Concrete Bridge	3800	600	1.03	n	1.25	n	N/A	2.02	N/A
A-67	St. Thomas Line near No. 9	L: CMP Culvert R: CMP Culvert	1850 1760	1250 1250	2.92	n	3.96	n	N/A	4.01	n
A-68	Carlingford Dr West	L: CMP Culvert R: CMP Open Arch	1650 1540	1150 980	2.35	n	3.72	n	N/A	3.94	n
A-69	Lanark Dr River Crossing at Adam’s Pond Outlet	3x HDPE Culverts	1000	1000	1.57	n	3.67	n	N/A	3.75	n
A-69-AP	Adam's Pond Walking Trail	HDPE Culvert	900	900	1.57	y	3.75	y	No Upgrade Recommended [2]	3.75	y

Basin A - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
A-70	Lanark Dr near No. 11	3x Concrete Boxes	1500	1200	6.66	n	8.12	n	N/A	9.15	n
A-71	Windmill Rd	CMP Culvert	1300	900	6.59	y	8.00	y	4.5 x 1.5 Concrete Box	8.99	n
A-72	Copper Canyon Close	CMP Open Arch	2250	1730	6.11	n	7.15	n	N/A	8.04	n
A-74	St. Thomas Line between Westport Dr and Tyrell Dr	CMP Culvert	1140	860	3.57	n	3.88	n	N/A	5.80	n
A-75	Across Westport Dr near St. Thomas Line	CPP Culvert	600	600	0.40	y	0.78	y	1200 HDPE Culvert	0.43	n
A-76	Stonewall Dr Stream Crossing	CMP Culvert	1040	940	1.40	n	1.77	n	N/A	1.77	n
A-77	Plateau Park near Stonewall Dr	CMP Culvert	600	600	0.45	n	0.58	n	N/A	0.58	n
A-78	Plateau Park near Stonewall Dr	CPP Culvert	600	600	0.45	y	0.66	y	750 HDPE Culvert	0.64	n
A-79	Ashlen Cres Ditch	CPP Culvert	600	600	0.51	n	0.65	n	N/A	0.65	n

Notes:
1. Structure outside Town Boundary - No Improvements Examined
2. It is not recommended to upgrade the walking trail structure, as doing so will result in increased occurance of undersized structures downstream

Basin B - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
B-1	Waterford River at Kenmount Rd	CMP Open Arch	10860	7520	27.14	n	35.00	n	N/A	35.00	n
B-2	Waterford River at Kenmount Rd	L: 3x HDPE Culverts R: Steel Footbridge	600 7000	600 1500	26.86	n	34.61	y	Raise/Remove Bridge	34.61	N/A
B-3	NL Highway Maintenance Depot	2x CMP Culverts	1550	1470	11.16	y	13.93	y	Not Examined [1]	13.93	y
B-4	Outer Ring Rd Eastbound Onramp	L: CMP Culvert R: CMP Culvert	2000 1550	1420 1220	11.12	n	13.89	y	Not Examined [1]	13.89	y
B-5	Kenmount Rd from Donovan's Industrial Park	L: CMP Culvert R: CMP Culvert	1930 2080	1270 1120	4.93	n	6.93	n	N/A	6.93	n
B-6	Outer Ring Rd Eastbound Offramp at Kenmount Rd	L: CMP Culvert R: CMP Culvert	2000 1580	1420 1020	9.70	n	12.20	y	Not Examined [1]	12.20	n
B-7	Outer Ring Rd near Kenmount Rd	L: CMP Culvert R: CMP Culvert	1650 2030	1070 1400	9.70	n	12.26	n	N/A	12.26	n
B-8	Outer Ring Rd Westbound Offramp at Kenmount Rd	CMP Culvert	600	600	0.31	n	0.42	n	N/A	0.42	n
B-9	Outer Ring Rd Westbound Onramp at Kenmount Rd	L: CMP Culvert R: CMP Culvert	2000 1700	1420 1170	9.66	n	14.80	y	Not Examined [1]	14.80	y
B-10	Kenmount Rd near NL Hydro Switchyard	L: CMP Culvert R: CMP Culvert	2110 1580	1300 1120	9.66	n	14.80	y	Not Examined [1]	14.80	y
B-11	Bremigen's Blvd	CMP Open Arch	3020	1850	2.31	n	3.39	n	N/A	3.39	n
B-12	St. Anne's Industrial Park	2x CMP Culverts	1500	1500	11.95	y	17.84	y	Bioswale along T'Railway	12.65	y
B-13	St. Anne's Industrial Park	CMP Culvert	1000	1000	0.39	y	0.75	y	Bioswale along T'Railway	0.54	y
B-14	Into St. Anne's Industrial Park	CMP Culvert	900	900	13.74	y	15.11	y	Bioswale along T'Railway	16.77	y
B-15	Outer Ring Rd near Neville's Pond	CMP Culvert	2400	2400	8.97	n	12.50	n	N/A	12.50	n
B-16	Hollyberry Dr	CMP Culvert	3100	1800	7.55	n	10.54	n	N/A	10.54	n
B-17	Karwood Dr near T'Railway	CMP Culvert	1320	1000	3.30	n	3.90	n	N/A	3.90	n
B-18	Neville's Pond Inlet	CMP Culvert	450	450	1.87	y	1.90	y	Not Examined	1.90	y
B-19	St. Anne's Industrial Park	CMP Culvert	1500	1500	12.12	y	16.27	y	Bioswale along T'Railway	10.54	y
B-20	Outer Ring Rd Eastbound Offramp at Topsail Rd	CMP Culvert	1000	1000	3.91	y	5.23	y	Not Examined [1]	5.23	y
B-21	Topsail Rd at Outer Ring Rd Overpass	CMP Culvert	1000	1000	3.59	y	4.91	y	Not Examined [1]	4.91	y
B-22	Outer Ring Rd Westbound Onramp at Topsail Rd	CMP Culvert	820	820	2.23	y	2.58	y	Not Examined [1]	2.58	y
B-23	Shelby St	HDPE Culvert	450	450	1.88	y	2.21	y	Not Examined	1.94	y
B-24	T'Railway near Shelby St	CMP Culvert	900	900	1.89	y	2.21	y	Not Examined	1.94	n
B-25	Outer Ring Rd Eastbound Onramp at Topsail Rd	CMP Culvert	600	600	4.24	y	5.72	y	Not Examined [1]	5.72	y
B-26	Topsail Rd at Outer Ring Rd Overpass	CMP Culvert	1000	1000	2.02	y	2.30	y	Not Examined [1]	2.30	y
B-27	Outer Ring Rd Westbound Offramp at Topsail Rd	CMP Culvert	660	380	2.67	y	3.83	y	Not Examined [1]	3.83	y
B-28	No. 1309 Topsail Rd Parking Lot	CMP Culvert	1650	1650	4.62	n	5.18	n	N/A	5.42	n
B-29	Canterbury Dr	2x CMP Culverts	1000	1000	5.73	y	9.55	y	3.5 x 2.5 Concrete Box	11.81	n
B-30	Elgin Park	Steel Footbridge	9870	1450	8.30	n	10.50	n	N/A	10.54	n

Basin B - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
B-31	Ellesmere Ave	2x CMP Culverts	1500	1500	6.36	n	7.83	y	3.0 x 1.5 Concrete Box	7.83	n
B-32	Carlisle Dr	CMP Culvert	1500	1500	2.47	n	2.85	n	N/A	2.85	n
B-33	Stephanie Ave	CMP Culvert	1500	1500	0.96	n	1.17	n	N/A	1.17	n
B-34	Elizabeth Dr near Stephanie Ave	CMP Culvert	750	750	0.17	n	0.25	n	N/A	0.25	n

Notes: 1. Department of Transportation and Works Structure - Not in Study Scope

Basin C - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
C-1	No. 544 St. Thomas Line	CMP Culvert	600	600	0.18	n	0.27	n	N/A	0.27	n
C-2	No. 494 St. Thomas Line	CMP Culvert	600	600	0.32	y	0.52	y	2x 600 HDPE Culverts	0.53	n
C-3	Whelan Cres near St. Thomas Line	L: CPP Culvert C: CMP Culvert R: CMP Culvert	750 1600 1200	750 1200 1200	20.18	y	29.28	y	7.0x2.1 Concrete Box	30.91	n
C-4	St. Thomas Line near Whelan Cres	CMP Culvert	600	600	0.66	y	1.08	y	1.5x0.6 Concrete Box	1.08	n
C-5	Squires Rd	2x CMP Culverts	2100	1500	19.90	y	28.83	y	7.0x2.1 Concrete Box	30.48	n
C-6	No. 11 Neary Rd	CMP Culvert	1250	1150	4.21	y	5.04	y	5.6x1.2 Concrete Box	11.55	n
C-7	Neary Rd near St. Thomas Line	CMP Culvert	610	610	7.28	y	10.17	y	1.6x1.3 Concrete Box	3.70	n
C-8	Father Lacey Pl	CMP Culvert	1550	1250	10.10	y	13.04	y	5.6x1.3 Concrete Box	13.13	n
C-9	Seascape Dr	CMP Open Arch	3660	1520	2.87	n	3.98	n	N/A	3.98	n
C-10	Howard Ave near No. 16	CMP Culvert	2000	2000	2.00	n	3.22	n	N/A	3.22	n
C-11	Howard Ave near No. 16	CMP Culvert	2000	2000	1.71	n	2.70	n	N/A	2.70	n
C-12	Morgan Ave near No. 4	CMP Culvert	750	750	0.15	n	0.22	n	N/A	0.22	n
C-13	Howard Ave near No. 2	Concrete Box	1200	1200	1.55	n	2.46	n	N/A	2.46	n
C-14	Picco Dr near Howard Ave	CPP Culvert	1200	1200	1.43	n	2.27	n	N/A	2.27	n
C-15	Howard Ave near No. 2	CPP Culvert	900	900	0.28	n	0.52	n	N/A	0.52	n
C-16	Picco Dr near Howard Ave	CPP Culvert	900	900	0.27	n	0.50	n	N/A	0.50	n
C-17	St. Thomas Line near No. 450	CMP Culvert	600	600	0.11	n	0.16	n	N/A	0.16	n
C-18	St. Thomas Line near No. 450	CMP Culvert	2000	1200	0.78	n	1.39	n	N/A	1.39	n
C-19	No. 440 St. Thomas Line Driveway	CMP Culvert	1800	1250	7.97	y	11.78	y	4.2x1.8 Concrete Box	13.85	n
C-20	O'Brien's Way	CMP Culvert	1450	1450	7.75	y	11.46	y	4.2x1.8 Concrete Box	13.58	n
C-21	No. 394 St. Thomas Line	CMP Culvert	1450	1450	6.18	y	9.12	y	4.2x1.5 Concrete Box	11.18	n
C-22	No. 380 St. Thomas Line	CMP Culvert	1700	1300	6.01	y	8.85	y	4.4x1.4 Concrete Box	10.92	n
C-23	Lawlor's Rd	2x CMP Culverts	1620	1150	5.81	y	8.55	y	5.4x1.2 Concete Box	10.67	n
C-24	Across St. Thomas Line near Raymond's Ln	CMP Culvert	750	750	1.18	y	1.60	y	1.6x0.7 Concrete Box	1.61	n
C-25	Raymond's Ln	CMP Culvert	750	750	2.32	y	3.83	y	3.0x1.0 Concrete Box	4.73	n
C-26	No. 292 St. Thomas Line	2x Steel Culverts	750	750	2.31	y	3.81	y	3.0x1.0 Concrete Box	4.70	n
C-27	No. 290 Thomas Line	2x Steel Culverts	750	750	2.30	y	3.81	y	3.0x1.0 Concrete Box	4.69	n
C-28	No. 282 Thomas Line	CMP Culvert	1000	1000	2.30	y	3.80	y	3.0x1.0 Concrete Box	4.67	n
C-29	Johnathon Dr	CPP Culvert	1230	1230	2.31	y	4.01	y	2.2x1.2 Concrete Box	4.57	n
C-30	St. Thomas Line near No. 271	CMP Culvert	600	600	0.34	n	0.58	n	N/A	0.25	n
C-31	Deborah Lynn Heights near St. Thomas Line	CPP Culvert	750	750	0.59	n	0.96	n	N/A	0.96	n
C-32	Deborah Lynn Heights near St. Thomas Line	CMP Culvert	1600	1000	0.59	n	0.96	n	N/A	0.96	n
C-33	Deborah Lynn Heights near St. Thomas Line	2x CPP Culverts	900	900	2.03	n	2.69	n	2.4x1.0 Concrete Box	3.45	n
C-34	Quilty's Rd	L: CMP Culvert R: CMP Culvert	1100 700	900 700	1.98	n	2.64	y	2.6x0.9 Concrete Box	3.36	n

Basin C - Hydraulic Structures and Proposed Improvements

No.	Location	Existing Structure			Scenario 1		Scenario 2		Scenario 3		
		Type	Span (mm)	Rise (mm)	Peak Flow (m^3/s)	Undersized (y/n)	Peak Flow (m^3/s)	Undersized (y/n)	Proposed Improvement	Peak Flow (m^3/s)	Undersized (y/n)
C-35	Byrne's Rd	L: CPP Culvert R: CMP Culvert	750 1300	750 900	1.86	n	2.46	y	2.4x0.9 Concrete Box	3.14	n
C-36	Hickey's Rd	CPP Culvert	1200	1200	1.59	n	2.13	y	1.6x1.2 Concrete Box	2.73	n
C-37	No. 205 St. Thomas Line	CMP Culvert	600	600	0.61	y	0.86	y	900 HDPE Culvert	0.86	n
C-38	Paradise Rd near St. Thomas Line	CMP Culvert	600	600	0.26	n	0.38	n	N/A	0.38	n
C-39	Paradise Rd near No. 359	CMP Culvert	600	600	1.02	n	1.30	n	N/A	1.33	n
C-40	Paradise Rd near No. 359	CMP Culvert	600	600	0.07	n	0.11	n	N/A	0.11	n
C-41	St. Thomas Line near No. 644	CMP Culvert	750	750	0.35	n	0.54	n	N/A	0.54	n
C-42	St. Thomas Line near No. 650	CMP Culvert	1800	1550	2.60	n	4.05	n	N/A	4.05	n
C-43	Atlantica Dr	CMP Culvert	1400	1400	1.70	n	2.78	n	N/A	2.78	n
C-44	St. Thomas Line near No. 660	CMP Culvert	1000	1000	0.51	n	0.74	n	N/A	0.74	n
C-45	St. Thomas Line at Stapleton's Rd	CMP Culvert	800	800	1.18	y	1.91	y	2.0x0.8 Concrete Box	1.92	n
C-46	St. Thomas Line near No. 676	CMP Culvert	600	600	0.13	n	0.21	n	N/A	0.21	n
C-47	St. Thomas Line near No. 684	CMP Culvert	600	600	0.32	n	0.48	y	750 HDPE Culvert	0.48	n
C-48	St. Thomas Line near Moonlight Dr	CMP Culvert	600	600	1.32	y	1.96	y	2.0x0.8 Concrete Box	1.97	n

APPENDIX E

Cost Estimates

Project: Stormwater Management Plan
Client: Town of Paradise
Date: November, 2019
Estimate: Basin A Improvements - Proposed Structure Upgrades

OPINION OF PROBABLE COST

No.	Location	Proposed Improvement	Length (m)	Subtotal	Contingency (20%)	Engineering (15%)	Subtotal	HST (15%)	Estimate Total	Estimate Total (Rounded)	Comments
A-1	River Confluence near No 1973 Topsail Road	Increase bridge to 5.5m x 1.6m	13	$469,800.00	$93,960.00	$70,470.00	$634,230.00	$95,134.50	$729,364.50	$730,000	
A-4	54 Topsail Pond Road	Private driveway: 5.5 x 1.2 bridge	5	$256,900.00	$51,380.00	$38,535.00	$346,815.00	$52,022.25	$398,837.25	$399,000	Replace Bridge at Driveway of 54 Topsail Pond Road.
A-10	317 Buckingham Drive	2.0 x 0.6 Concrete Box	11	$58,800.00	$11,760.00	$8,820.00	$79,380.00	$11,907.00	$91,287.00	$92,000	
A-11	28 Three Island Pond Rd	600 HDPE Culvert	12	$12,400.00	$2,480.00	$1,860.00	$16,740.00	$2,511.00	$19,251.00	$20,000	
A-12	59 Three Island Pond Rd	600 HDPE Culvert	12	$12,400.00	$2,480.00	$1,860.00	$16,740.00	$2,511.00	$19,251.00	$20,000	
A-13	75 Three Island Pond Road	2.0 x 0.6 Concrete Box	9	$48,100.00	$9,620.00	$7,215.00	$64,935.00	$9,740.25	$74,675.25	$75,000	
A-14	74 Three Island Pond Road	600 HDPE Culvert	18	$18,600.00	$3,720.00	$2,790.00	$25,110.00	$3,766.50	$28,876.50	$29,000	
A-15	105 Three Island Pond Road	2.3 x 0.6 Concrete Box	16	$88,000.00	$17,600.00	$13,200.00	$118,800.00	$17,820.00	$136,620.00	$137,000	
A-18	Three Island Pond near Shalloway Road	2.0 x 0.6 Concrete Box	12	$64,200.00	$12,840.00	$9,630.00	$86,670.00	$13,000.50	$99,670.50	$100,000	
A-24	291 Three Island Pond Road	2.0 x 1.2 Concrete Box	16	$86,300.00	$17,260.00	$12,945.00	$116,505.00	$17,475.75	$133,980.75	$134,000	
A-30	1973 Topsail Road	Private driveway: 7.5 x 2.0 Concrete Box	5	$175,200.00	$35,040.00	$26,280.00	$236,520.00	$35,478.00	$271,998.00	$272,000	Gravel Driveway
A-31	1960 Topsail Road	7.5 x 2.0 Concrete Box	9	$317,100.00	$63,420.00	$47,565.00	$428,085.00	$64,212.75	$492,297.75	$493,000	Paved Driveway
A-32	1956 Topsail Road	7.5 x 2.0 Concrete Box	5	$175,200.00	$35,040.00	$26,280.00	$236,520.00	$35,478.00	$271,998.00	$272,000	Gravel Driveway
A-34	Newdale Road River Crossing	7.5 x 2.0 Concrete Box	6	$211,400.00	$42,280.00	$31,710.00	$285,390.00	$42,808.50	$328,198.50	$329,000	
A-36	1907 Topsail Road	4.0 x 1.2 Concrete Box	24	$234,500.00	$46,900.00	$35,175.00	$316,575.00	$47,486.25	$364,061.25	$365,000	
A-37	1899 Topsail Road	4.0 x 1.3 Concrete Box	9	$96,900.00	$19,380.00	$14,535.00	$130,815.00	$19,622.25	$150,437.25	$151,000	Grassed Area
A-38	1899 Topsail Road	4.0 x 1.3 Concrete Box	6	$63,500.00	$12,700.00	$9,525.00	$85,725.00	$12,858.75	$98,583.75	$99,000	Paved Driveway
A-39	1895 Topsail Road	1.8 x 1.7 Concrete Box	20	$133,300.00	$26,660.00	$19,995.00	$179,955.00	$26,993.25	$206,948.25	$207,000	Paved Parking Lot
A-40	Spracklin Boulevard	2.0 x 1.5 Concrete Box	32	$208,000.00	$41,600.00	$31,200.00	$280,800.00	$42,120.00	$322,920.00	$323,000	
A-41	Greenfields Place	1.8 x 1.5 Concrete Box	26	$160,300.00	$32,060.00	$24,045.00	$216,405.00	$32,460.75	$248,865.75	$249,000	
A-47	Pinehill Place	5.5 x 1.4 Concrete Box	11	$183,200.00	$36,640.00	$27,480.00	$247,320.00	$37,098.00	$284,418.00	$285,000	
A-48	Near 9 St. Thomas Line	5.0 x 1.3 Concrete Box	18	$262,000.00	$52,400.00	$39,300.00	$353,700.00	$53,055.00	$406,755.00	$407,000	
A-49	Carlingford Drive East	2.6 x 1.7 Concrete Box	16	$134,700.00	$26,940.00	$20,205.00	$181,845.00	$27,276.75	$209,121.75	$210,000	

Project: Stormwater Management Plan
Client: Town of Paradise
Date: November, 2019
Estimate: Basin A Improvements - Proposed Structure Upgrades

OPINION OF PROBABLE COST

No.	Location	Proposed Improvement	Length (m)	Subtotal	Contingency (20%)	Engineering (15%)	Subtotal	HST (15%)	Estimate Total	Estimate Total (Rounded)	Comments
A-51	Path south of Paradise Recreation Centre	1.6 x 1.4 Concrete Box	8	$47,100.00	$9,420.00	$7,065.00	$63,585.00	$9,537.75	$73,122.75	$74,000	Gravel Path
A-53	Octagon Pond East Inlet	Non-Restrictive Footbridge	18	$53,200.00	$10,640.00	$7,980.00	$71,820.00	$10,773.00	$82,593.00	$83,000	Replace Footbridge at Octagon East Outlet.
A-54	Road Behind Paradise Recreation Centre	1.8 x 1.2 Concrete Box	170	$1,002,300.00	$200,460.00	$150,345.00	$1,353,105.00	$202,965.75	$1,556,070.75	$1,557,000	Gravel Path
A-58	Neil's Pond Brook	Open Channel	9	$3,175.00	$635.00	$476.25	$4,286.25	$642.94	$4,929.19	$5,000	Removal of Culverts on Neil's Pond Brook.
A-59	Neil's Pond Brook	Open Channel	120	$42,000.00	$8,400.00	$6,300.00	$56,700.00	$8,505.00	$65,205.00	$66,000	
A-60	Neil's Pond Brook	Open Channel	13	$9,175.00	$1,835.00	$1,376.25	$12,386.25	$1,857.94	$14,244.19	$15,000	
A-61	Neil's Pond Brook	Open Channel	35	$24,525.00	$4,905.00	$3,678.75	$33,108.75	$4,966.31	$38,075.06	$39,000	
A-62	McNamara Drive River Crossing	6.0 x 1.6 Concrete Box	21	$538,600.00	$107,720.00	$80,790.00	$727,110.00	$109,066.50	$836,176.50	$837,000	
A-65	Downstream of Neil's Pond Outlet	1.4 x 1.0 Concrete Box	18	$177,000.00	$35,400.00	$26,550.00	$238,950.00	$35,842.50	$274,792.50	$275,000	Gravel Path
A-71	Windmill Road	4.5 x 1.5 Concrete Box	12	$164,100.00	$32,820.00	$24,615.00	$221,535.00	$33,230.25	$254,765.25	$255,000	
A-75	Across Westport Drive near St. Thomas Line	1200 HDPE Cuvlert	22	$38,800.00	$7,760.00	$5,820.00	$52,380.00	$7,857.00	$60,237.00	$61,000	
A-78	Plateau Park near Stonewall Drive	750 HDPE Culvert	84	$92,300.00	$18,460.00	$13,845.00	$124,605.00	$18,690.75	$143,295.75	$144,000	Undeveloped/Grass (near house)

This opinion of probable costs is presented on the basis of experience, qualifications and best judgement. It has been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments and the like are beyond the control of CBCL Limited. We cannot warrant or guarantee that actual costs will not vary significantly from the opinion provided.

Project: Stormwater Management Plan
Client: Town of Paradise
Date: November, 2019
Estimate: Basin B Improvements - Proposed Structure Upgrades

OPINION OF PROBABLE COST

Item	Location	Proposed Improvement	Length (m)	Subtotal	Contingency (20%)	Engineering (15%)	Subtotal	HST (15%)	Estimate Total	Estimate Total (Rounded)	Comments
B-12	St. Anne's Industrial Park	Bioswale along T'Railway	400	$92,400.00	$18,480.00	$13,860.00	$124,740.00	$18,711.00	$143,451.00	$144,000	Complete 400m of Bioswale
		1.8 x 1.8 Concrete Box @ 1345 Topsail Road	16	$107,500.00	$21,500.00	$16,125.00	$145,125.00	$21,768.75	$166,893.75	$167,000	Remove CMP cuvlert and install box culvert
		1.8 x 1.8 Concrete Box @ St. Annes Crescent	14	$94,100.00	$18,820.00	$14,115.00	$127,035.00	$19,055.25	$146,090.25	$147,000	Remove CMP cuvlert and install box culvert
B-13	St. Anne's Industrial Park	Bioswale along T'Railway									Estimate completed under item B-12
B-14	Into St. Anne's Industrial Park	Bioswale along T'Railway									Estimate completed under item B-12
B-19	St. Anne's Industrial Park	Bioswale along T'Railway									Estimate completed under item B-12
B-29	Canterbury Dr	3.5 x 2.5 Concrete Box	34	$381,900.00	$76,380.00	$57,285.00	$515,565.00	$77,334.75	$592,899.75	$593,000	
B-31	Ellesmere Ave	3.0 x 1.5 Concrete Box	23	$211,900.00	$42,380.00	$31,785.00	$286,065.00	$42,909.75	$328,974.75	$329,000	

This opinion of probable costs is presented on the basis of experience, qualifications and best judgement. It has been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments and the like are beyond the control of CBCL Limited. We cannot warrant or guarantee that actual costs will not vary significantly from the opinion provided.

Project: Stormwater Management Plan
Client: Town of Paradise
Date: November, 2019
Estimate: Basin C Improvements - Proposed Structure Upgrades

OPINION OF PROBABLE COST

Item	Location	Proposed Improvement	Length (m)	Subtotal	Contingency (20%)	Engineering (15%)	Subtotal	HST (15%)	Estimate Total	Estimate Total (Rounded)	Comments
C-2	494 St. Thomas Line	2x 600 HDPE Culverts	16	$27,700.00	$5,540.00	$4,155.00	$37,395.00	$5,609.25	$43,004.25	$44,000	
C-3	Whelan Crescent near St. Thomas Line	7.0 x 2.1 Concrete Box	15	$507,600.00	$101,520.00	$76,140.00	$685,260.00	$102,789.00	$788,049.00	$789,000	
C-4	St. Thomas Line near Whelan Crescent	1.5 x 0.6 Concrete Box	14	$67,900.00	$13,580.00	$10,185.00	$91,665.00	$13,749.75	$105,414.75	$106,000	
C-5	Squires Road	7.0 x 2.1 Concrete Box	16	$539,800.00	$107,960.00	$80,970.00	$728,730.00	$109,309.50	$838,039.50	$839,000	
C-6	11 Neary Road	5.6 x 1.2 Concrete Box	18	$294,600.00	$58,920.00	$44,190.00	$397,710.00	$59,656.50	$457,366.50	$458,000	
C-7	Neary Road near St. Thomas Line	1.6 x 1.3 Concrete Box	15	$84,300.00	$16,860.00	$12,645.00	$113,805.00	$17,070.75	$130,875.75	$131,000	Roadside
C-8	Father Lacey Place	5.6 x 1.3 Concrete Box	12	$203,100.00	$40,620.00	$30,465.00	$274,185.00	$41,127.75	$315,312.75	$316,000	
C-19	440 St. Thomas Line	4.2 x 1.8 Concrete Box	25	$311,600.00	$62,320.00	$46,740.00	$420,660.00	$63,099.00	$483,759.00	$484,000	
C-20	O'Brien's Way	4.2 x 1.8 Concrete Box	13	$161,400.00	$32,280.00	$24,210.00	$217,890.00	$32,683.50	$250,573.50	$251,000	
C-21	394 St. Thomas Line	4.2 x 1.5 Concrete Box	9	$109,100.00	$21,820.00	$16,365.00	$147,285.00	$22,092.75	$169,377.75	$170,000	Paved Driveway
C-22	380 St. Thomas Line	4.4 x 1.4 Concrete Box	6	$78,500.00	$15,700.00	$11,775.00	$105,975.00	$15,896.25	$121,871.25	$122,000	Gravel Driveway
C-23	Lawlor's Road	5.4 x 1.2 Concete Box	16	$258,000.00	$51,600.00	$38,700.00	$348,300.00	$52,245.00	$400,545.00	$401,000	
C-24	St. Thomas Line across near Raymond's Lane	1.6 x 0.7 Concrete Box	26	$129,100.00	$25,820.00	$19,365.00	$174,285.00	$26,142.75	$200,427.75	$201,000	
C-25	Raymond's Lane	3.0 x 1.0 Concrete Box	14	$115,400.00	$23,080.00	$17,310.00	$155,790.00	$23,368.50	$179,158.50	$180,000	Gravel Driveway/Road
C-26	292 St. Thomas Line	3.0 x 1.0 Concrete Box	4	$33,600.00	$6,720.00	$5,040.00	$45,360.00	$6,804.00	$52,164.00	$53,000	Gravel Driveway
C-27	290 St. Thomas Line	3.0 x 1.0 Concrete Box	4	$33,600.00	$6,720.00	$5,040.00	$45,360.00	$6,804.00	$52,164.00	$53,000	Gravel Driveway
C-28	282 St. Thomas Line	3.0 x 1.0 Concrete Box	16	$134,100.00	$26,820.00	$20,115.00	$181,035.00	$27,155.25	$208,190.25	$209,000	Paved Driveway
C-29	Johnathan Drive	2.2 x 1.2 Concrete Box	4	$26,300.00	$5,260.00	$3,945.00	$35,505.00	$5,325.75	$40,830.75	$41,000	
C-33	Deborah Lynn Heights near St. Thomas Line	2.4 x 1.0 Concrete Box	13	$88,000.00	$17,600.00	$13,200.00	$118,800.00	$17,820.00	$136,620.00	$137,000	
C-34	Quilty's Road	2.6 x 0.9 Concrete Box	16	$112,100.00	$22,420.00	$16,815.00	$151,335.00	$22,700.25	$174,035.25	$175,000	
C-35	Byrne's Road	2.4 x 0.9 Concrete Box	15	$94,000.00	$18,800.00	$14,100.00	$126,900.00	$19,035.00	$145,935.00	$146,000	
C-36	Hickey's Road	1.6 x 1.2 Concrete Box	13	$70,500.00	$14,100.00	$10,575.00	$95,175.00	$14,276.25	$109,451.25	$110,000	
C-37	205 St. Thomas Line	900 HDPE Culvert	17	$26,300.00	$5,260.00	$3,945.00	$35,505.00	$5,325.75	$40,830.75	$41,000	

Project: Stormwater Management Plan
Client: Town of Paradise
Date: November, 2019
Estimate: Basin C Improvements - Proposed Structure Upgrades

OPINION OF PROBABLE COST

Item	Location	Proposed Improvement	Length (m)	Subtotal	Contingency (20%)	Engineering (15%)	Subtotal	HST (15%)	Estimate Total	Estimate Total (Rounded)	Comments
C-45	St. Thomas Line at Stapleton's Road	2.0 x 0.8 Concrete Box	20	$112,800.00	$22,560.00	$16,920.00	$152,280.00	$22,842.00	$175,122.00	$176,000	
C-47	near 684 St. Thomas Line	750 HDPE Culvert	17	$19,300.00	$3,860.00	$2,895.00	$26,055.00	$3,908.25	$29,963.25	$30,000	
C-48	St. Thomas Line near Moonlight Drive	2.0 x 0.8 Concrete Box	15	$84,700.00	$16,940.00	$12,705.00	$114,345.00	$17,151.75	$131,496.75	$132,000	

This opinion of probable costs is presented on the basis of experience, qualifications and best judgement. It has been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments and the like are beyond the control of CBCL Limited. We cannot warrant or guarantee that actual costs will not vary significantly from the opinion provided.