City of Parksville

Storm Drainage Master Plan

MARCH 2016

KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

PARKSVILLE, BC

KOERS
& ASSOCIATES
ENGINEERING LTD.
Consulting Engineers

P.O. BOX 790
194 MEMORIAL AVENUE
PARKSVILLE, B.C. V9P 2G8
Phone: (250) 248-3151
Fax: (250) 248-5362
kael@koers-eng.com
www.koers-eng.com

March 11th, 2016
1346 - Final Report SDMP

City of Parksville
PO Box 1390
100 E Jensen Avenue
Parksville, BC V9P 2H3

Attention: **Ms. Rosa Telegus, P Eng.**
Development Engineer

Ladies and Gentlemen:

Re: City of Parksville
STORM DRAINAGE MASTER PLAN – Final Report

Koers & Associates is pleased to submit three bound copies and an electronic copy in pdf format of our "City of Parksville Storm Drainage Master Plan, March 2016"

This study examines the adequacy of the storm drainage system under two conditions:

- Current Conditions, and
- Full Build-Out of the Official Community Plan (projected to occur in Year 2072).

The conclusions and recommendations of this study are meant to complement the existing capital projects plan, and should be reviewed on a regular basis and if site investigations (such as video inspection) reveal differences between the model input data (pipe diameter, material type, pipe slope) and actual conditions. If there are major departures in the assumed flows, changes to development patterns or land use, or adjustments to the City's boundaries, then the model should be updated regularly as such developments occur, and as additional flow monitoring data comes available in case adjustments to the proposed works are required.

We thank you for the opportunity to have worked on this assignment. We will be pleased to assist the City with updating the model as development proposals are being considered and with helping to implement the study recommendations on future capital projects.

Please call if you have any questions.

Yours truly,

KOERS & ASSOCIATES ENGINEERING LTD.

R Cave

Chris Holmes, P Eng	Richard Cave, AScT	Rob Hoffman, P Eng
Project Engineer	Sr. Design Technologist	Project Manager

Enclosure

STORM DRAINAGE MASTER PLAN

Table of Contents

Page

Cover Letter

EXECUTIVE SUMMARY .... **iv**

1 INTRODUCTION **1**
1.1 Authorization 1
1.2 Background & Previous Studies 1
1.3 Study Objectives 2
1.4 Scope of Work 2
1.5 Acknowledgements 4

2 EXISTING SYSTEM **5**
2.1 General 5
2.2 Collection System 5
2.3 Outfalls 6
2.4 Storm Water Detention & Treatment Systems 7
2.4.1 Existing Facilities 7
2.4.2 Bylaw Requirements 7

3 STORMWATER MANAGEMENT **10**

4 FLOW MONITORING **12**
4.1 City Owned flow Monitoring Equipment 12
4.2 Monitoring Sites 12
4.3 Rainfall Events 13
4.4 Runoff Volumes 14
4.5 Runoff Flows 15
4.6 Local Soils (Impact on Runoff Volume & Flows) 17

5 CLIMATE CHANGE **18**

6 DESIGN CRITERIA **19**
6.1 Minor and Major Systems 19
6.2 Rainfall 19
6.2.1 Historical Storms 19
6.2.2 IDF Curve Update 19
6.2.3 Hyetographs 20

Table of Contents, Cont'd

7 COMPUTER MODEL 22
7.1 Computer Model 22
7.2 XP-SWMM Model Overview 22

8 COMPUTER MODEL DATA ENTRY & CALIBRATION 24
8.1 Data Collection & Entry 24
8.2 Calibration Model 27
8.3 Existing Conditions 28
8.4 OCP Build-Out 29
8.4.1 Municipal Boundary Expansions 29
8.4.2 RDN Controlled Lands 29
8.4.3 E&N Railway Corridor 29

9 MODELLING RESULTS 30
9.1 General 30
9.2 Undersized Pipes and Pipes to be Monitored 30
9.2.1 Existing Conditions 31
9.2.2 Existing System at OCP Build-Out 31
9.2.3 Proposed System at OCP Build-Out 31
9.3 Groundwater Infiltration 32
9.4 Stormwater Runoff Treatment 32

10 CAPITAL PLANNING 34
10.1 Condition Risk Analysis 34
10.2 Capacity Risk Analysis 35
10.3 Capital Planning Priority 36
10.4 Infrastructure Condition Verification 36

11 RECOMMENDED IMPROVEMENTS & COST ESTIMATES 37

12 CONCLUSIONS 38

13 RECOMMENDATIONS 44

TABLES

1 Pipe Lengths and Age by Diameter 5
2 Outfall Locations and Catchment Areas 6
3 Existing Stormwater Detention & Retention Systems 7
4 Flow Monitoring Sites 13
5 Rainfall Intensity and Return Periods, Jan 10-11 & Feb 15-16, 2014 13
6 Maximum Volume, Jan 10 - 11 & Feb 15 - 16, 2014 15

STORM DRAINAGE MASTER PLAN

Table of Contents, Cont'd

TABLES

7 Maximum Flow, Dec 5, 2013 to March 21, 2014 17
8 Rainfall Intensity Comparison between City of Parksville Existing Design IDF Curves and Proposed Curves 20
9 Manning's 'n' Values 24
10 Land Use and Percent Impervious 25
11 Soil Types & SCS Curve Number 26
12 Time of Concentration 27
13 Storm Drain Undersized Under Existing & OCP Build-Out Conditions .... (after page) 31
14 Cost Estimates .... (after page) 37

FIGURES **(after page)**

1 Existing Storm Drainage System 5
2 Outfall Catchment Areas 6
3 Flow Monitoring Sites 12
4 Parksville Proposed IDF Curves & Historical Storms 19
5 Synthetic Storm Rainfall Distribution Comparison 21
6 1 hour 10 year & 100 Year Design Rainstorm 21
7 6 hour 10 year & 100 year Design Rainstorm 21
8 DMH 1322 Metered vs Computer Modeled Flows 27
9 DMH 879 Metered vs Computer Modeled Flows 28
10 DMH 916 Metered vs Computer Modeled Flows 28
11 Existing Land-Use (Zoning) Plan 29
12 Official Community Plan 29
13 Projected Areas of Growth 29
14 Drawing Sheet Location Key Plan 31

Figures listed below are located in Appendix A

1 Storm Drainage Flow Monitoring Sites Catchment Areas
2 DMH 879 (153 Corfield Rd) Daily Volume (Dec 2013 – March 2014)
3 DMH 916 (256 Mills St) Daily Volume (Dec 2013 – March 2014)
4 DMH 1322 (281 Chestnut St) Daily Volume (Dec 2013 – March 2014)
5 DMH 879 (153 Corfield Rd) Daily Flow (Dec 2013 – March 2014)
6 Daily Unit Flow vs Rainfall (Dec 2013 – March 2014)
7 DMH 916 (256 Mills St) Daily Flow (Dec 2013 – March 2014)
8 DMH 1322 (281 Chestnut St) Daily Flow (Dec 2013 – March 2014)
9 Suggested Future Flow Monitoring Site

City of Parksville

STORM DRAINAGE MASTER PLAN

Table of Contents, Cont'd

Figures listed below are located in Appendix C

1-1 General Location Map
1-2 Little Qualicum River Hatchery, 1981 – 2010 Climate Normal
2-1 Definition of Climate Variability & Climate Change
3-1 Parksville Regional Climate Stations – Normalized Maximum Annual Daily Rainfall
3-2 Comox Airport Climate Stations – Normalized Maximum Annual Daily Rainfall
4-1 Recommended Updated City of Parksville IDF Curves
4-2 Recent Extreme Rainfall Events
5-1 Future Climate (2050s) Adjusted IDF Curve Example

DRAWINGS (located in pocket at end of report)

1346 – 1 to 7 Existing System at Existing Conditions & OCP Build-Out
1346 – 8 to 14 Proposed System & OCP Build-Out

APPENDICIES

A Technical Memorandum No. SDFM (Storm Drainage Flow Monitoring)

B Technical Memorandum No. SD-2 (Storm Drainage Design Criteria)

C Technical Memorandum Task 7 (IDF Curves & Climate Change)

D Technical Memorandum No. SS-1 & SD-1 (Software Evaluation)

E XPSWMM Technical Literature

EXECUTIVE SUMMARY

<u>Study Objectives</u>

The main objectives of this Storm Drain Master Plan were to develop a calibrated computer model that:

- Integrates with the City's GIS database to reflect current conditions; and
- Identifies the infrastructure required to accommodate future development in accordance with the City's OCP.

The computer model was to be used by engineering staff to:

- Review the impact of proposed development on the downstream infrastructure and establish the extent of off-site works and services that will be required;
- Set priorities for the capital works program and develop the annual budget;
- Identify Development Cost Charge projects and the allocation of project costs;
- Assist in reviewing the potential impact of climate change, and
- Assess the potential impact of the failure of key infrastructure components.

<u>Growth Projections</u>

The projected OCP build-out is just under 22,000, compared to the 2014 estimated population of just over 12,000. The OCP build-out population is notably lower than the 41,600 service population projected in the 1998 study. Based on available growth projections, it is estimated that the OCP Build-out would occur around Year 2072 (57 years).

<u>Computer Model & Calibration</u>

The program XP-SWMM was chosen for modelling the storm drainage system. It is a comprehensive program used for modelling sanitary sewer and storm drainage collection systems or combined systems, river systems, and floodplains. It can carry out extended time simulations and present modelling results in a variety of graphical and tabular forms. The program interfaces with AutoCAD and the City's GIS program, MapGuide.

As part of the study, flows were recorded in the field at five monitoring sites (manholes) during a 3 ½ month period (December 2013 and February 2014). The data was used to calibrate the computer model. The model calculated slightly higher peak flows than the peak flows recorded at the three monitoring sites.

<u>Modelling Results</u>

In general, the findings of the conveyance capacity of the storm drain network can be summarized as follows:

- Under current conditions:
  - A total of 3,076 m (4.1% of the collection system) are undersized to convey the 10 year design storm. These pipes are calculated to be flowing more than 100% full.

- At OCP Build-out:
  - The length of main undersized to convey the 10 year design storm increases to 4,137 m (5.5% of the collection system).

Climate Change
Climate Change model forecasts vary significantly based on the input data and assumptions made in developing the models. There are known weather cycles that can span from a few years (El Nino/El Nina) to several decades (the Pacific Decadal Oscillation). It is thought that we may have recently shifted from a warm to a cool PDO phase which could result in more extreme precipitation for the Parksville region. Extreme precipitation for the area is forecast to increase by 15% to 50% for the hourly events, which is the governing Time of Concentration for the City's storm drainage network, excluding Romney and Shelly Creeks with their approximate 6 hour Time of Concentration.

It has been put forward that, given the uncertainty in the change of future extreme rainfall intensities, drainage system resilience be improved through the use of a percentage full limit for pipe design which requires increasing to the next available pipe size if design flow results in pipe flowing more than a set limit, for example 75% to 80% full. Applying a more than 80% full design criteria, results in several upgrading works throughout the City.

Sea Level Rise
The Provincial Government has recommended that average sea level rise of 1 m by year 2100 and 2 m by 2200 be used for coastal flood planning.

Design Criteria
It is recommended the City's design standard for the conveyance capacity of gravity mains be based on 80% full. The current standards do not include conveyance capacity criteria.

Storm Drain Outfalls
Of the City's 24 stormwater outfalls, two have catchments that originate outside of the City's boundaries; Romney Creek and Shelley Creek.

Increasing flow in Romney Creek can have the greatest impact on the City's infrastructure as the majority of the creek is completely enclosed within the City. There are two culverts on Romney Creek that act as choke points for larger storm events: the 600 mm diameter CSP culvert under the E&N Victoria Line and the 900 mm x 1200 mm concrete arch under the E&N Port Alberni line. The City should be diligent and proactive to ensure that the impacts of upgrading these culverts are understood. This will require a co-ordinated effort with the RDN, Island Corridor Foundation, and Southern Rail. If the culverts are upsized, the impact on the City's infrastructure will need to be re-assessed to determine if development of a stormwater detention system is warranted to ensure the increase in the peak flow is restricted to the safe conveyance capacity of the enclosed storm drain system. This study assumes that these culverts will continue to limit the flows that enter the City.

Shelly Creek is an open water course as it passes through the City except at the three road crossings (Hamilton, Butler and Martindale roads) where it is culverted. Modelling shows the Butler and Martindale Road culverts to be 107% and 115% full, respectively, under a 10 year design storm event. Under larger storm events the surcharging will increase. As a fish bearing watercourse, the provincial Riparian Area Regulations (RAR) govern any works done within the riparian area.

Only one of the City's storm drain outfalls, at the top end of Renz Road, discharges to lands outside of the City's boundaries.

Eleven outfalls discharge onto the foreshore either directly into Parksville Bay (7) or west of the Bay (4).

Nine outfalls discharge either directly into a watercourse that discharges into the Englishman River Estuary (3), to ground within the estuary (2), into Shelly Creek (3) or in to the Englishman River (1). The City's OCP includes guidelines for diverting stormwater away from the Englishman River Estuary, as well as avoiding discharges to watercourses.

There are three municipal outfalls discharging into Craig Bay.

The City's OCP includes guidelines for incorporating on-site stormwater management techniques and providing stormwater treatment for groundwater protection. The City should consider expanding this to include first flush treatment of runoff prior to discharge to the environment, where it can cost effectively and appropriately be accommodated.

Groundwater Infiltration

There are few notably visible areas of very permeable soils within the City: the Craig Bay Area, the Herring Gull Industrial Park and in and around the area of Pioneer Crescent.

For Craig Bay it has been assumed for modelling purposes that there will be no further expansion of the municipal collection system and all future development in the area will be required to manage stormwater runoff on-site.

For the Pioneer Crescent area, there is already a developed municipal collection system. For the Herring Gull Industrial Park area, there is a network of ditching and culverts provided at driveway access points. Developments in this area should therefore be assessed on a case-by-case basis with the requirement to infiltrate stormwater as much as possible.

Video Inspection & Capital Planning

Video inspection of all mains in the storm drain system is recommended to confirm the current condition of each pipe, service connection and manholes. The information should be checked against the computer model and the City's database, and both updated as needed to reflect actual conditions. The computer model should be re-run as new information becomes available to assess if other works are required.

This video library would serve several purposes including:

- Verifying the pipe information the computer model uses,
- Assessing condition of older mains or ones with incomplete information,
- Prioritizing identified maintenance works,
- Coordinating the findings with road construction works to ensure the underground infrastructure is dealt with before the road is repaved, and
- Capital Planning.

This video inspection of 3.0 to 5.0 kms of pipe per year would result in the inspection of the 75 kms of pipe over a 15 to 25 year period.

1 INTRODUCTION

1.1 Authorization

The City of Parksville authorized Koers & Associates Engineering Ltd. to update the Master Plan for the City's storm drainage system. The work was to be carried out in accordance with Koers' proposal dated November 29, 2013 and was to include computer modelling analysis of the storm drainage system under current conditions and under future development conditions in accordance with the City's 2013 Official Community Plan – A Vision for the Future.

1.2 Background & Previous Studies

The City's current master drainage document was completed 17 years ago and was an update of the previous master drainage plan developed 29 years ago. A brief summary of each is presented below.

<u>Master Drainage Plan, 1986</u>
This report was completed in April 1986 by Ker Priestman & Associates Ltd. The City's population at the time was 6,000.

The report was developed with the aid of the computer model ILLUDAS (Illinois Urban Drainage Area Simulator). The findings were based on the ability of the drainage system to adequately convey a design storm event with a 10 year return period.

The study identified $5.5 million (in 1986 dollars) of improvement works including works to service future development. Most of the works were implemented by 1998.

<u>Storm Drainage Study, 1998</u>
The City's current Storm Drainage Study was completed in February 1998 by Koers & Associates Engineering Ltd. Both the City's boundary had grown since completion of the 1986 study and the population was now 10,000; an increase of 4,000, or 67%.

This was a city wide study that utilized the computer model MIDUSS (Microcomputer Interactive Design of Urban Stormwater Systems). The model was used to identify works required to meet runoff flows upon full build-out for the land-uses proposed by the Official Community Plan (OCP). The program provided the City with a workable tool to assess the impacts of new developments as the applications were submitted.

The storm drainage design criteria expanded to include a 10 year and a 100 year return period storm. All pipes were assessed on their ability to convey the 10 year storm. For events up to the 100 year storm, new roads and developments were to be graded such that the runoff could be conveyed along road surfaces and not flow onto private property. For existing developments, where surface flow routes were not available, the system was modelled to convey up to the 100 year storm, with an allowance for the HGL to surcharge 1.3 m above the top of the storm pipe.

A review of the City's rainfall Intensity-Duration-Frequency (IDF) curves was carried out. The IDF curves which had recently been developed for the Town of Qualicum Beach were found to be the best fit and were used for modelling.

The study included a brief analysis of existing stormwater quality and the sensitivity of the receiving environment at each outfall.

The study identified $2.2 million of work attributed to development (eligible for Development Cost Charges) as well as $2 million of pipe replacement and duplication projects. Both costs are in 1998 dollars.

Since 1998, the City has continued to grow with significant growth in tourism and accommodation units. The City's municipal boundaries are unchanged but the population has increased to over 12,000 in 2013; an increase of 2,000, or 20%.

During the past 17 years, there have been significant advances in computer modelling, the City's OCP has undergone several changes, and new development as well as re-development of existing properties has occurred. Public awareness and education of the impact of human activity on aquatic habitat and the wildlife it sustains has increased significantly. The issues associated with climate change including the threat of rising sea levels, changes in rainfall patterns and intensities, and the potential impact on municipal infrastructure need to be considered as part of planning for future development and ongoing operation and maintenance of infrastructure.

The City identified the need to update the 1998 drainage study to reflect the infrastructure changes that have occurred, incorporate the new OCP to determine what infrastructure works are required to accommodate the anticipated growth, and review the potential impacts of climate change.

1.3 Study Objectives

The City desires to update their storm sewer computer models to reflect current conditions and identify the infrastructure required to accommodate future development in accordance with the City's OCP.

The purpose of the computer model and Storm Drainage Master Plan is to assist engineering staff with:

1. Development applications, specifically to review system capacity downstream of the proposed development so that requirements for works and services can be set.

2. Completing the yearly budget process and setting priorities for the capital works program; identifying Development Cost Charge, developer, and capital works projects; and allocating project costs.

3. Assist in reviewing the potential impact of climate change.

4. Rank the criticality of each improvement and develop a prioritization list.

5. Prioritize future investigations, monitoring locations and modelling calibration exercise.

1.4 Scope of Work

To meet the objectives of the study, the scope of work in our proposal dated November 29, 2013 was adopted, and is summarized as follows:

1. Meet with City staff and confirm project scope of work, objectives and goals, confirm deliverables and identify information required.

2. Assist City staff with:

- Carrying out a flow monitoring program over the fall/winter months (2013/14) when the groundwater table is higher and rainfall events are more frequent compared to spring and summer.
- Identifying key flow monitoring sites, being mindful that monitoring can be expensive. Obtain quotes from a flow monitoring contractor to install and monitor the equipment and provide collected data to the City.
- Reviewing and analyzing flow data in conjunction with rainfall and catchment area characteristics. Summarize findings in a Technical Memorandum.

3. Collect and review available information including relevant past reports and studies, zoning and OCP maps, HYDRA computer model, electronic copies of the City's GIS storm drain system data, digital copies of record drawings, and population data.

4. Carry out an evaluation of several stormwater computer programs including:

- compatibility with the City's GIS program;
- ease of data entry/editing;
- program capabilities;
- model calibration using flow and rainfall data; user friendliness, model accuracy, reliability, and sensitivity;
- results presentation and graphical interfaces;
- technical support; purchase and annual licensing costs.

Findings and software purchase recommendations to be presented in a Technical Memorandum *(Technical Memorandum No. SS-1 & SD-1 submitted March 26, 2014).*

5. Compare flow data results against the City's current design standards. Recommend design criteria to be used for existing conditions and for OCP build-out. Present findings in a Technical Memorandum *(Technical Memorandum Nos. SDFW & SD-2 submitted August 29, 2014)*.

6. Develop a computer model of the existing storm sewer collection system using digital information for the City's MIDUSS model and GIS database, and information from record drawings. Calibrate the model using the flow monitoring data.

   i) Run model under existing conditions using the City's current design criteria. Run model using recommended design criteria (item 5). Compare results against problem areas identified by City staff. Identify upgrading works.

   ii) Meet with City staff to review future growth and redevelopment areas, projected development timelines, and anticipated population densities. Develop and run computer model for OCP build-out using the recommended design criteria (item 5). Identify upgrading works required to accommodate OCP build-out.

   iii) Present modelling results in a Technical Memorandum *(submitted as Master Plan report)*.

7. Undertake an infrastructure criticality analysis to assess the likelihood and consequence of failure of various components of the collection system (The findings are to be used to develop a priority list of the recommended capital projects.

8. Present findings in a Master Storm Drainage Plan report complete with;
   - tables, colour figures and maps;

- priority list of projects with construction cost estimates suitable for budgetary purposes;
- identification of whether project is a capital works, DCC, or development project;
- identification of whether project meets higher levels of government infrastructure funding programs;
- conclusions and recommendations.

1.5 Acknowledgements

Koers & Associates Engineering Ltd. acknowledges with thanks the assistance provided by the following City staff during the course of the study and preparation of this Sanitary Sewer Master Plan:

- Mr. Vaughn Figueira, P Eng. – Director of Engineering
- Ms. Rosa Telegus, P Eng – Development Engineer
- Mr. Blaine Russell, MCIP – Director of Community Planning
- Mr. Connor Bankes, Engineering Technologist
- Mr. John Diggins, AScT Engineering Technologist
- Ms. Barbara Silenieks, – Engineering Technologist
- Mr. Randy Hall – GIS Technician

2 EXISTING SYSTEM

2.1 General

The City of Parksville, situated on the east side of Vancouver Island, is located to the north of the City of Nanaimo and south of the Town of Qualicum Beach. The municipal boundaries encompass 17.4 km^2 with development occurring along an approximately 7 km long by 1.3 km wide area (9 km^2) which in general is bounded by the E&N railway to the south and the ocean to the north.

The City is divided by the Englishman River whose catchment area includes Mount Arrowsmith with a peak elevation of just over 1,800 m. West of the Englishman River, the topography in general slopes moderately from a maximum elevation of 60 m at the E&N railway until it reaches the foreshore of the ocean (Salish Sea / Strait of Georgia); resulting in an averaged slope of around 3%. The exception to this would be the 15 m to 20 m high coastal bluffs located on the west side of Parksville Bay. East of the river, a portion of the land slopes west into the river while the majority slopes east into the ocean at Craig Bay. The majority of this area has a maximum elevation of 25 m with moderate slopes.

2.2 Collection System

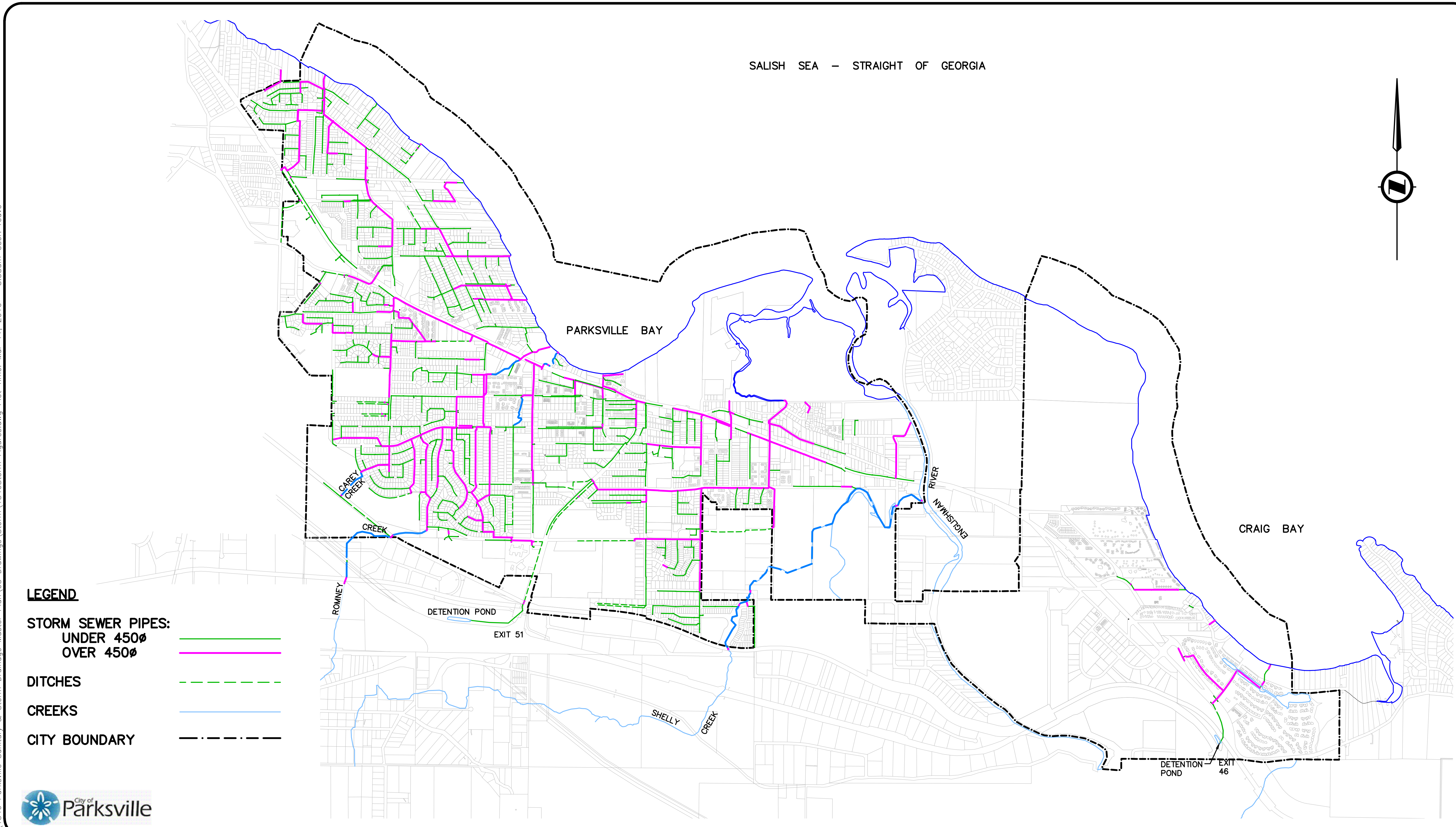
The City's collection system consists of 75 kilometres of storm drain mains in addition to roadside ditches and natural water courses. **Table 1** presents the total length and percentage of storm drain pipe in the City by pipe diameter and age. The collection system is shown on **Figure 1**.

Table 1 – Pipe Lengths and Age by Diameter

Age (years)	Pipe Diameter (mm)														
	150 - 200	**250 - 300**	**350 - 375**	**450**	**525**	**600 - 675**	**750**	**900**	**1050**	**1200**	**1350**	**1500 - 1650**	**2400**	**3050**	**Total**
0 - 10	61	2,371	1,132	867	670	438	1,028	682	71					48	7,369
11 - 30	1,800	**13,729**	4,261	4,579	1,820	5,692	2,757	3,422	676	498	1,257	320			40,810
31 - 60	920	7,825	1,459	2,251	485	1,084	1,172	471	521	489	359	214			17,250
?	815	3,819	1,169	1,419	665	1,312	306	112	76	125		5	63		9,887
Total	3,597	27,744	8,020	9,116	3,640	8,527	5,262	4,687	1,344	1,112	1,616	539	63	48	75,316
	Length as % of Total by Diameter														
0 - 10	0	3		1	1	1	1	1						0	10%
11 - 30	2	**18**	6	6	2	8	4	5	1	1	2	0			**54%**
31 - 60	1	10	2	3	1	1	2		1	1	0	0			**23%**
?	1	5	2	2	1	2	0	0	0	0		0	0		13%
Total	5%	**37%**	11%	12%	5%	11%	7%	6%	2%	1%	2%	1%	0%	0%	100%

There is just over 75 kilometres of storm drain pipe in the City. Slightly more than 50% is less than 450 mm in diameter and almost 90% is 750 mm in diameter or less.

More than 60% of the pipe was installed during the last 30 years and 87% of the pipe was installed during the last 60 years. The age of 13% of the pipe is not known.



KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

CLIENT	CITY OF PARKSVILLE
PROJECT	STORM DRAINAGE MASTER PLAN

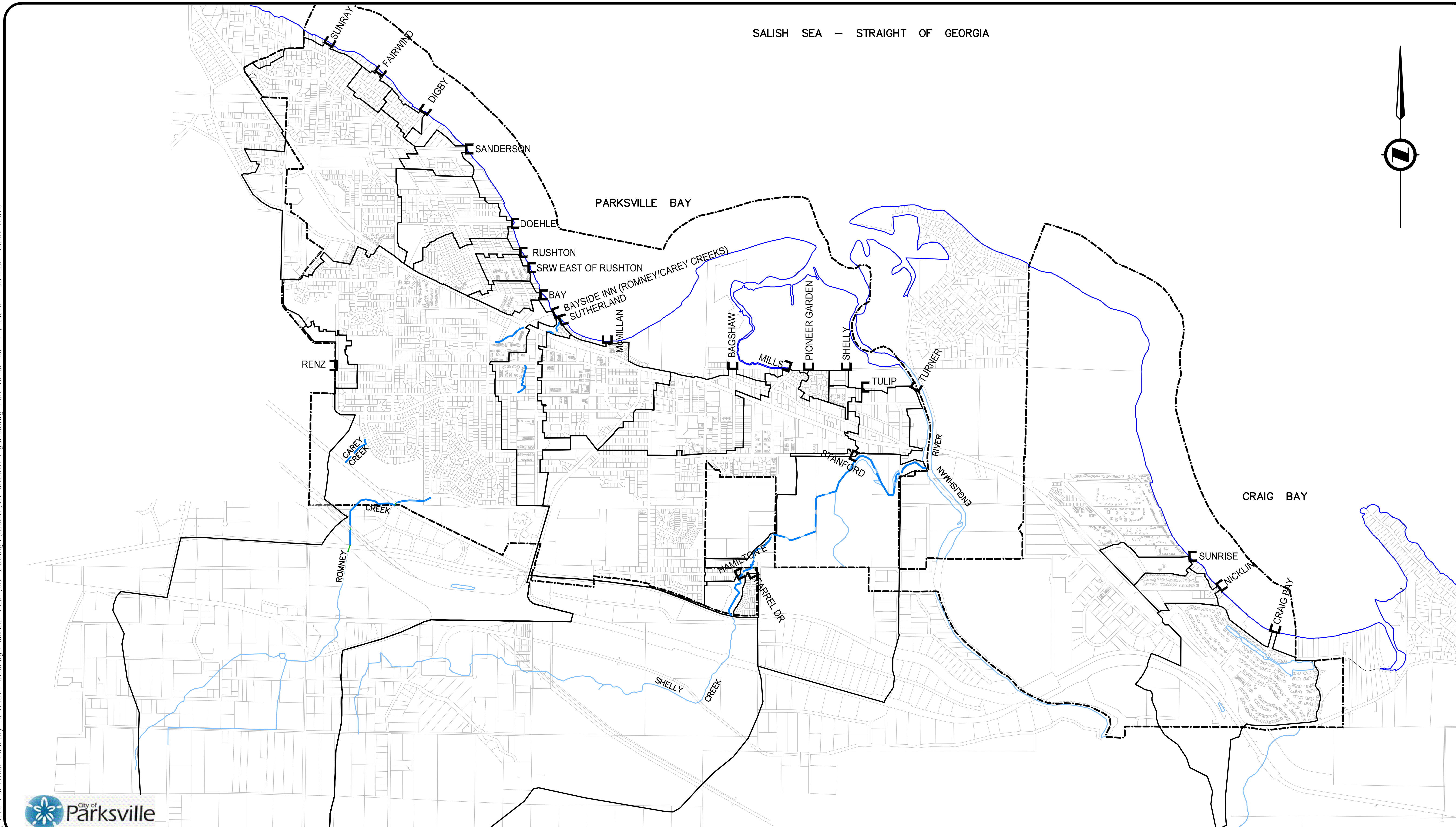
TITLE	COLLECTION SYSTEM		
APPROVED		SCALE	1:25,000
DATE	MAR 2015	DWG No.	FIGURE 1
PROJECT No.	1346		

2.3 Outfalls

There are a total of 23 storm drain outfalls within the City, discharging either onto the foreshore, into Shelly Creek, into the Englishman River or into the tidal estuary adjacent to the mouth of the Englishman River. There are also a few catchbasins at the south end of Renz Road that discharge west to lands outside of the City boundary. The locations of the outfall and their catchment area are shown on **Figure 2**. Characteristics of each catchment area are presented in **Table 2**. They are presented in order of location moving from west to east across the City.

Table 2 – Outfall Locations & Catchment Areas

Location of Discharge	Outlet Pipe Diameter (mm)	Catchment Area (ha)
WEST of the Englishman River		
Renz Rd (north end)	varies	6.5
Foreshore West of Parksville Bay		
Sunray Rd	600	132.0
Fairwind Ave	300	5.5
Digby Ave	Ditch	5.0
Sanderson Rd	375	9.5
Foreshore of Parksville Bay		
Doehle Rd	600	35.0
Rushton Avenue (Future)	Ditch	23.5
330 Dogwood SRW - Shoreline	900	23.5
Bay Ave	300	7.5
375 Island Hwy West/ Mini-Golf (Carey Creek)	1,200	720.0
Sutherland Pl	Ditch	34.5
McMillan St	1,200	50.0
Englishman River Estuary		
Bagshaw – 293 Bagshaw SRW	1,000	44.0
Mills St	1,050	190.0
493 Pioneer Crescent SRW – Pioneer Estates	750	4.0
Shelley Rd	Ditch	1.5
Tulip Lane	Ditch	3.0
Shelly Creek (tributary to Englishman River)		
Hamilton Ave, east end	450	11.5
Farrell Dr, north end	600	4.0
Stanford Ave, east end	1,000	1.0
Shelly Creek (at Martindale)	2 x 1200	639 ha
Englishman River		
Englishman River, Turner Road	600	19.0
EAST of the Englishman River		
Foreshore of Craig Bay		
1175 Resort Drive SRW – Sunrise Ridge	375	9.0
Nicklin Rd	Ditch	11.5
Saltspring Pl (Craig Bay Estates)	500	95.0



KOERS
& ASSOCIATES
ENGINEERING LTD.
Consulting Engineers

CLIENT	CITY OF PARKSVILLE	TITLE	OUTFALL CATCHMENT AREAS
PROJECT	STORM DRAINAGE MASTER PLAN	APPROVED	
		DATE	MAR 2015
		PROJECT No.	1346
		SCALE	1: 25,000
		DWG No.	FIGURE 2

File: H:\6012 Parksville City\1346 Parksville Sanitary & Storm Drainage Master Plan\03 Drawings\Storm\1346Storm Figures.dwg Plot Time: Mar 11, 2016 – 8:40am User: rcave

2.4 Storm Water Detention & Treatment Systems

2.4.1 Existing Facilities

There are several types of storm water detention or treatment systems within the City. They are owned and operated by the Ministry of Transportation and Infrastructure (MoTI), the City, or private development. **Table 3** below lists the ones that we are aware of along with their characteristics and purpose. There may be other privately owned systems that we are not aware of.

Table 3 – Existing Stormwater Detention & Treatment Systems

Owner, Year Installed	Location	Description	Purpose	Storage Volume	
				(m^3)	(m^3/ha)
Saltspring Place Outfall (Craig Bay Estates)					
MoTI, 1995	Hwy 19 Exit 46 (Hwy 19a)	Detention Pond, 1 m deep	Peak Flow Reduction	1,500	91
Private, 1995	Craig Bay Estates	Detention Pond, 1 m deep (On-line/Wet)	Peak Flow Reduction & Improve Water Quality	13,100	138
Mills Street Outfall					
City, 1998	Corfield at Hamilton	Detention Pond, 1.3 m deep (On-line/Wet)	Peak Flow Reduction	70	18
Bayside Inn Outfall (Carey/Romney Creek)					
MoTI, 1995	Hwy 19 Exit 51 (Hwy 4a)	Detention Pond, 1 m deep	Peak Flow Reduction	8,800	846
City, 2013	Renz Park	Below Ground Chamber 2.25 m height	Peak Flow Reduction	435	41
Private, 2014	Oceanside Medical Centre (Hwy 4A)	Detention Pond, 1.6 m deep (Off-Line/Dry)	Peak Flow Reduction	300	100
Sunray Road Outfall					
Private, 2004±	Wembley Mall	Detention Pond, 1.5 m deep (Off-Line/Dry)	Peak Flow Reduction	120	60
2014		Catchbasin Flow Restrictor	Peak Flow Reduction	n/a	
2014		Rooftop Storage	Peak Flow Reduction	n/a	
2014		Oil/Water Separator	Improve Water Quality	n/a	
City, 2008	Pym St	Below Ground Chamber, 2.7 m height	Peak Flow Reduction	400	41

Note:
MoTI (BC Ministry of Transportation & Infrastructure)

2.4.2 Bylaw Requirements

OCP Requirements
The OCP contains goals for managing the quantity and quality of stormwater generated within the City before it is discharged from the storm drainage system back into the natural environment. These include:

- Improving storm water drainage quality prior to discharging into the environment (Chapter 6, Storm Drainage),
- Providing scientific information on climate change and the potential implications for municipal infrastructure (Objective 9),
- Mimicking pre-development runoff flows through rainwater capture, stormwater infiltration, and detention (Section 6.1, Goal 3)

- Minimizing non-essential impervious surfaces, and
- Specific guidelines for a number of the Development Permit Areas (DPAs) including:
  - DP 1-8 water conservation measures
  - DP 9 diverting storm water away from the Englishman River estuary
  - DP 11 incorporating on-site storm water management techniques
  - DP 13 avoiding discharges to watercourses
  - DP 16 providing storm water treatment for groundwater protection

Subdivision Servicing Bylaw

The City's Engineering Standards and Specifications include the following regarding stormwater runoff control, quality, and quantity:

D-1 DRAINAGE DESIGN CRITERIA

1.0 Introduction

Design and construction of storm drainage facilities are subject to the requirements of the Fish and Wildlife Branch of the Ministry of the Environment, Department of Fisheries and Oceans, and any other agencies having jurisdiction.

3.0 Drainage Design Methods and Flows

b) Storm Water Management Systems

Storm Water Management Systems shall incorporate such techniques as lot grading, surface infiltration, and sub-surface disposal, storage, or other acceptable methods, to limit the peak runoff from development.

7.0 Site and Lot Grading

e) Individual lot(s) will not be permitted to direct storm water discharge or drainage into any natural watercourse, park, or green belt area. Sheet flow may be permitted.

10.0 Detention Facilities

Large developments, generally independent of existing drainage facilities, or where the existing drainage system is known or proven to be inadequate, will be required to provide detention of storm water to the pre-development runoff flows.

Detention facilities will be designed with bottom drainage to ensure the facility is dry when not in use.

24.0 Rockpits

The use of rockpits in the Municipality is discouraged and will only be permitted at the discretion of the Municipal Engineer. Rockpits will only be considered in certain areas of the Municipality where it can be demonstrated that the subsoil conditions will provide a percolation rate equal to, or in excess of, twice the minor runoff flows.

Sanitary & Storm Sewerage System Bylaw No. 1319

This bylaw regulates the connection to the City's storm and sanitary sewer system and the type, quantity and quality of the sewage and storm water discharged into the systems.

The bylaw focuses mainly on discharge to the sanitary sewer system and detailed water quality requirements for the discharge of industrial wastewater to the sanitary sewer system as well as the requirement for the use of grease, oil and sand interceptors for vehicle repair and maintenance shops, service stations and vehicle or equipment washing facilities (see items 31, 32 and 33). This can be interpreted that stormwater runoff from these businesses is to be discharged to the sanitary sewer system; not the storm drainage system.

While the bylaw does not specifically regulate the quality of water discharged to the storm drainage system, item 48 does allow the City to disconnect, plug or seal a storm drain line if the discharge from the property if the discharge:

a) creates an immediate danger to any person;

b) endangers or interferes with the operation of the storm sewer system; or

c) contains pollutants which are harmful to the environment.

<u>Subdivision Servicing Bylaw</u> (out-dated references)
The City's Engineering Standards and Specifications include several references to outdated documents, reports and computer models as follows:

D-1 DRAINAGE DESIGN CRITERIA

1.0 <u>Introduction</u>
The first paragraph references the Storm Drainage Study dated February 1998 by Koers & Associates Engineering Ltd.

3.0 <u>Drainage Design Methods and Flows</u>
a) <u>Conventional Systems</u>
ii) Hydrograph Methods
2) makes reference to using the most current version of the MIDUSS program and parameters and terms of reference for analysis to be provided by the City. (The City's system is now modelled using XP-SWMM). It also notes other modelling systems such as ILLUDAS or OTHYMO may be considered for use at the discretion of the Municipal Engineer.

3 STORMWATER MANAGEMENT

Municipal governments are granted the authority for the operating of drainage infrastructure by the *Local Government Act.* With this authority, local governments can be held accountable for downstream impacts that result from changes to upstream drainage patterns. In addition, there are other government policies and regulations that come to bear on storm water quantity and quality such as the RDN Liquid Waste Management Plan, the Federal Fisheries Act and the Provincial Fish Protection Act, Streamside Protection Regulation.

The science of stormwater management continues to develop, and there are now a number of resources specific to BC that are available to guide municipalities and development in implementing Best Management Practises (BMPs). Some of these include:

- Land Development Guidelines for the Protection of Aquatic Habitat, Fisheries & Oceans Canada, September 1993
- Stormwater Planning: A Guidebook for British Columbia, Ministry of Water, Land and Air Protection, 2002
- Develop with Care 2014: Environmental Guidelines for Urban & Rural Land Development in BC
- Environmental Best Management Practices for Urban & Rural Land Development in BC, DRAFT, 2014
- Liquid Waste Management Plan, Nanaimo Regional District, 2014

A stormwater management plan is to incorporate BMPs that acknowledge stormwater is not just a drainage or flood management issue but also a resource for:

- Fish and other aquatic species,
- Groundwater recharge (for both stream summer flow and for potable water supply),
- Water supply, and
- Aesthetic and recreational uses.

As noted in **2.4.2 Bylaw Requirements**, the OCP contains goals for managing the quantity and quality of stormwater and the City's Subdivision Servicing Bylaw lists requirements for quality and quantity control.

The BMPs implemented for each land development project will vary depending on the receiving environment (river, ocean, or ground) and the capacity of the downstream infrastructure. Some of the more common BMPs for quantity control include:

- infiltration systems;
- detention pond;
- Low Impact Development (LID) design standards; and
- Green Building or LEED design standards.

Some of the more common BMPs for quality control include:

- bio-filters (vegetated swales);
- rain gardens;
- engineered wetlands; and
- oil/water separators.

Stormwater runoff quality data specific to the City of Parksville can be found in the following two reports:

- City of Parksville Storm Drainage Study, February 1998 by Koers & Associates Engineering Ltd.

- Water Quality Assessment for Aquatic Life (Englishman River and Estuary), 2012± City of Parksville.

Discussion regarding potential areas for ground water infiltration within the City is presented in **9.3 Groundwater Infiltration**. An overview of issues relating to stormwater runoff treatment in several areas of the City is presented in **9.4 Stormwater Runoff Treatment**.

4 FLOW MONITORING

4.1 City Owned flow Monitoring Equipment

The City owns two flow monitors. One has been installed at a permanent site since 2009. The other is a portable unit purchased in December 2013 in conjunction with the monitoring work done for this study. The permanent site is on the sanitary sewer main at SMH 720 located in the parking lot on the west side of the curling rink in the Community Park.

The portable equipment purchased in December 2013 is an ISCO 2150 area velocity meter. It measures depth of flow and velocity and can record: depth of flow, velocity, flow rate and total flow. It has a total weight of 7.4 lbs. and is powered by two 6 volt batteries. Its power life is listed at up to 15 months at a 15 minute data storage interval. Data can be viewed in the field using a laptop computer or the manufacturer's weatherproof computer module "Field Wizard".

4.2 Monitoring Sites

Flow monitoring was carried out by SFE Global Ltd. for the City of Parksville for the period Dec 5, 2013 to March 21, 2014 at three locations. The monitoring sites were established in conjunction with City and Koers & Associates project team members. The three monitoring sites were as follows:

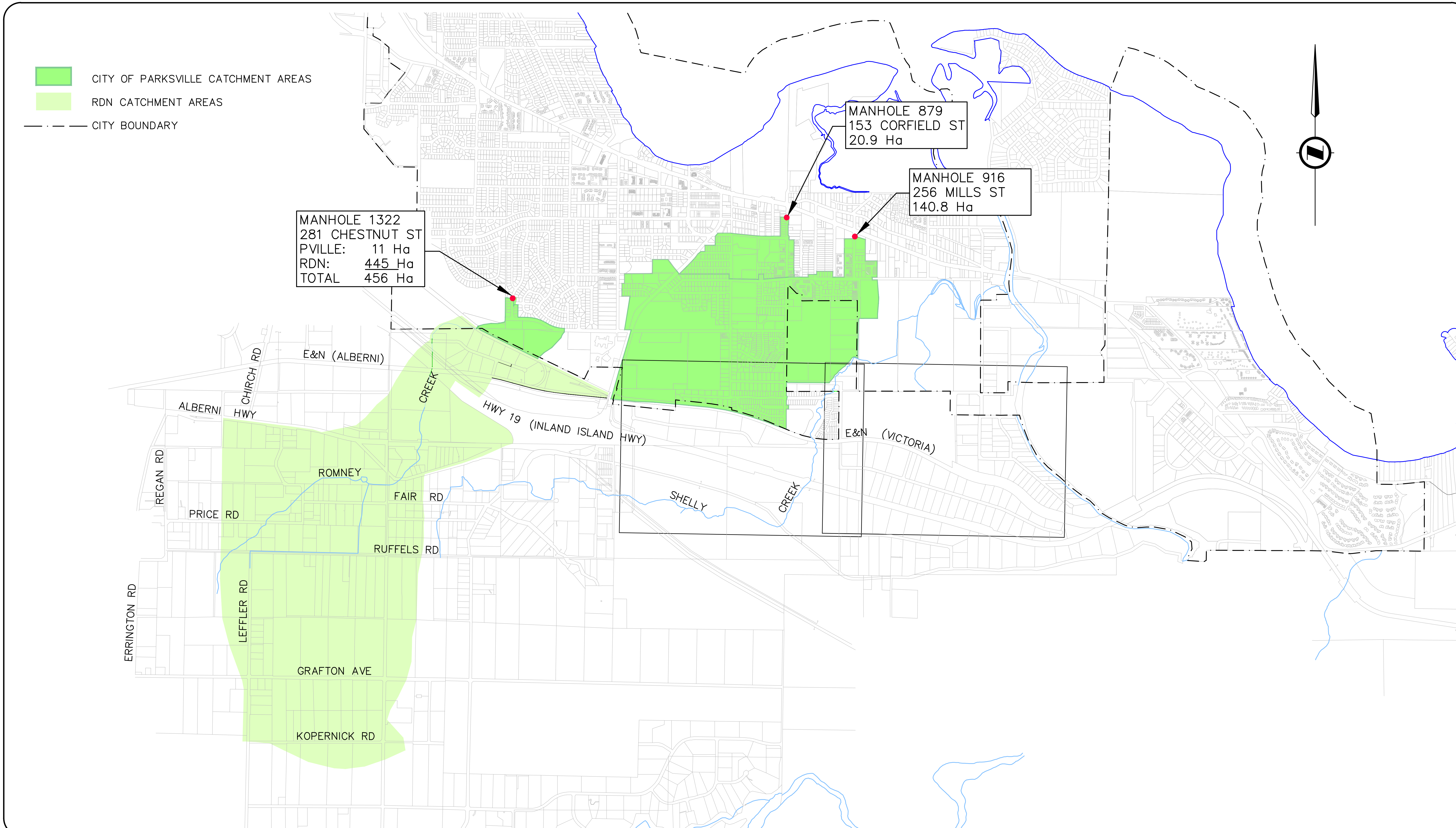
<u>DMH 879 (153 Corfield St)</u>
The Corfield site was selected because it is a smaller almost fully developed catchment (80%±) with a mixture of residential, commercial, institutional and park land uses. The extent of development in the area aided in calibrating runoff curve numbers in the computer model.

<u>DMH 916 (Mills St)</u>
The Mills site was selected as it is a larger catchment with a lot of residential development and has several large areas of undeveloped land. This assisted in calibrating runoff curve numbers for developed and undeveloped areas.

<u>DMH (281 Chestnut St)</u>
The Chestnut site was selected as it is a very large catchment and almost entirely outside of the City. This assisted in calibrating runoff curve numbers for rural developed lands that contribute flows into the City.

Originally four monitoring sites were proposed but this was cutback to three due to budget constraints. The fourth site was at the intersection of Morison Ave and Finholm St and its catchment area included one of the other monitoring site catchment areas.

The location and contributing catchment area for each of the three temporary monitoring sites is presented in **Table 4**. The location of each monitoring site and its catchment area boundary is shown on **Figure 3**. (also Appendix A – Figure 1).



KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers
P.O. Box 790, 194 Memorial Ave. Parksville, BC V9P 2G8 www.koers-eng.com Ph: 250-248-3151 Fax: 250-248-5362

CLIENT	CITY OF PARKSVILLE
PROJECT	STORM DRAINAGE MASTER PLAN

TITLE	FLOW MONITORING SITES & CATCHMENT AREAS		
APPROVED		SCALE	1:25,000
DATE	MAY 2014	DWG No.	FIGURE 3
PROJECT No.	1346		

Table 4 – Flow Monitoring Sites

Location	DMH No.	No. of Lots	Catchment Area (ha)		Description
			Total	Undeveloped	
153 Corfield St	879	105+	21	4.1	Mixed use including new and older single family residential, multi-family, commercial, institutional (civic centre).
256 Mills St	916	500+	141	61	Mainly single family residential with some multi-family, elementary school.
281 Chestnut St	1322	10+ (1)	456	0 (within City of Parksville)	11 ha of land inside City (single family residential, middle school, park) and 445 ha of Romney Creek catchment outside of the City's boundary in the Regional District of Nanaimo.

An overview of the flow monitoring results is presented below. Additional information is presented in **Appendix A** - Technical Memorandum No. SDFM (Storm Drain Flow Monitoring) including the flow monitoring report by SFE Global Ltd.

4.3 Rainfall Events

Rainfall in the City is recorded by two weather stations operated by the City; one at the Public Works Yard, operating since January 2005, the other at the Community Park flow measuring site which commenced recording rainfall in August 2009. The rainfall data referenced in this report is from the Community Park rain gauge, which is physically closer to the flow monitoring and recording sites.

There were a number of rainfall events during the 3 ½ month (Dec 2013 to mid-March 2014) monitoring period. The two largest events were as follows:

- January 10 - 11, 2014 18.8 mm + 26.4 mm = 45.2 mm
- February 15 - 16, 2014 16.6 mm + 27.4 mm = 44 mm

The intensities of the Jan 10-11 event were less than the Feb 15-16 event for the 5 minute to 6 hour durations but greater for the 12 hour and 24 hour durations. These two events were compared with the City's current IDF Curve to determine the storm's return period for various times of concentration. As noted in **Table 5**, both storms fall below a 5 year return period.

Table 5 - Rainfall Intensity and Return Periods for Jan 10/11, 2014 & Feb 15/16 2014

Duration	Jan. 10 – 11, 2014		Feb. 15 – 16 , 2014		Current IDF Curve 5 Yr. Return (mm/hr)
	Intensity (mm/hr)	Return Period	Intensity (mm/hr)	Return Period	
5 min	12.0	< 5 yr	12.0	< 5 yr	50
10 min	9.6	< 5 yr	12.0	< 5 yr	34
15 min	8.8	< 5 yr	11.2	< 5 yr	28
30 min	8.4	< 5 yr	9.6	< 5 yr	20
60 min	6.8	< 5 yr	8.4	< 5 yr	15
2 hour	5.4	< 5 yr	6.4	< 5 yr	10
6 hour	3.5	< 5 yr	4.2	< 5 yr	-
12 hour	2.47	< 5 yr	2.17	< 5 yr	-
24 hour	1.88	< 5 yr	1.3	< 5 yr	-

4.4 Runoff Volumes

The daily runoff volume was plotted for each monitoring site. Maximum day runoff volumes were recorded at two of the monitoring sites (DMH 879 and 281) during the Jan 10/11 rainfall event. At the third monitoring site (DMH 916), the maximum day volume was recorded during the Feb 15/16 event. A discussion of the findings for each site is presented below and is summarized in **Table 6**.

<u>DMH 879 (153 Corfield)</u>
For this, the smallest, catchment area monitored (21 ha), the maximum daily volume for the two storm events are nearly identical at 4,471 m^3 and 4,299 m^3 on January 11 and February 16, respectively, as shown in **Appendix A - Figure 2**. These equate to unit volumes of approximately 215 m^3/ha and 205 m^3/ha; respectively.

<u>DMH 916 (256 Mills)</u>
For this catchment area (141 ha), the maximum daily volume was recorded for the Feb 15/16 event at 21,790 m^3 compared to 13,216 m^3 for the January 10/11 storm as shown on **Appendix A - Figure 3**. These equate to unit volumes of approximately 155 m^3/ha and 95 m^3/ha; respectively.

The higher volumes from the February 15/16 event may have been in response to the higher volume of rain in the days before the February storm, which could have caused the soils to become saturated and thereby reduced their infiltration capacity. The lowered infiltration capacity combined with the higher rainfall intensities of the February event would have resulted in a larger volume of surface runoff compared to the January 10/11 event.

<u>DMH 1322 (281 Chestnut)</u>
For this, the largest catchment (456 ha) of the three monitoring sites, the Jan 10/11 event produced a much higher maximum day volume (21,002 m^3) than the Feb 15/16 event (15,358 m^3) as shown on **Appendix A - Figure 4**. These equate to unit volumes of 45 m^3/ha and 35 m^3/ha; respectively. This catchment area is part of the Romney Creek system.

The maximum volume occurring with the January event is opposite to the DMH 916 (256 Mills) site which recorded the maximum day volume for the February event. The exact reason for this reversal and the notably lower volume for the February event could not be determined from the data available. It may have been that the Jan 10/11 rainfall in this catchment area (Romney Creek) which originates outside of the City's boundary (nearly 2 kilometres inland towards the mountains) was different (i.e. a longer event with higher intensities) than what was recorded at the rain gauge (at the Community Park).

The maximum day volume of 21,002 m^3 is nearly identical to the 21,790 m^3 recorded for the DMH 916 (256 Mills) catchment which has an area that is less than 1/3rd the size (141 ha vs 456 ha). A discussion on potential reason(s) for the difference is discussed in **4.6 Local Soils**.

For the February event, the maximum volume was recorded on February 17 whereas it was recorded on February 16 at the other two sites. The recording of the maximum volume on February 17 for this monitoring site is suspected to reflect the majority of the rainfall (80%) and the peak intensities occurring between 2 pm and 6 pm on the 16th, in-conjunction with the longer time of concentration attributed to this much larger catchment compared to the catchment areas of the other two monitoring sites.

Table 6 – Maximum Volume, January 10 - 11 & February 15 - 16, 2014

DMH No.	Location	Catchment Area (ha)	Volume		Comments
			(m^3)	(m^3 per ha)	
January 10 - 11, 2014		45.2 mm	(18.8 mm + 26.4 mm)		
879	153 Corfield St	21	4,471	215	4 ha (20%) undeveloped
916	256 Mills St	141	13,216	95	61 ha (43%) undeveloped
1322	281 Chestnut St	456	21,002	46	Romney Creek, 445 ha (98%) outside City (within RDN)
February 15 - 16, 2014		44 mm	(16.6 mm + 27.4 mm)		
879	153 Corfield St	21	4,299	205	4 ha (20%) undeveloped
916	256 Mills St	141	21,790	155	61 ha (43%) undeveloped
1322	281 Chestnut St	456	15,358	35	Romney Creek, 445 ha (98%) outside City (within RDN)

4.5 Runoff Flows

The daily maximum, average and minimum flow were plotted for each site. The daily maximum was also converted to a unit flow per ha and graphed. A discussion of the findings for each site is presented below. **Table 7** presents the maximum flow recorded for each monitoring site.

DMH 879 (153 Corfield)
The majority (80%±) of the 21 ha of the catchment area is developed with a mixture of residential, commercial, institutional and park land uses, including 0.7 ha (3%) of park behind (south of) City Hall. Undeveloped areas include the 2.6 ha cleared and grubbed site on the south side of Jensen Ave between McCarter St and Corfield St.

While this site recorded the seconded highest flow (336 L/s) of the three monitoring sites, it had the highest unit flow per ha (16.1 L/s per ha). A plot of the daily maximum, minimum and average vs rainfall for the 3 ½ months of monitoring is shown in **Appendix A - Figure 5**. **Appendix A - Figure 6** shows the calculated daily average flow per ha vs rainfall for each site for the 3 ½ months of monitoring.

The calculated higher runoff flow per ha of catchment area is representative of the extent of the catchment's development, its storm drainage system, and its smaller size; all of which are expected to have resulted in a quicker runoff time, higher peak flows and less chance for infiltration compared to the two other monitoring sites.

For the January event, the peak flow (336 L/s) was recorded at 1:40 am on the 11^{th}. A review of the rainfall data suggests a Time of Concentration of 10 to 15 minutes for this catchment area based on the time delay between peak rainfall intensities and peak flow recorded at the flow meter.

For the February event, the peak flow (268 L/s) was recorded at 5:00 pm on the 16^{th}; five minutes after the peak rainfall intensity.

Monthly graphs of the recorded flow vs rainfall in 5 minutes increments is located in **Appendix A** in the SFE Global Sanitary and Storm Water Flow Monitoring, Winter 2013/14 report.

DMH 916 (256 Mills)
Slightly more than 40% (61 ha) of this 141 ha catchment area is not developed. This includes several large contiguously forested areas including 31 ha and 10 ha on the south and north sides of Despard Avenue between the Alberni Hwy and Craig Street. There are also 13 ha of mostly cleared but undeveloped land in the RDN south of Stanford Ave between Corfield St and Blower Rd.

The highest flow of the three monitoring sites was recorded at this site on February 16, 2014 at 897 L/s. The second highest flow at this site was recorded on January 11, 2014 at 533 L/s. A plot of the daily maximum, minimum and average vs rainfall for the 3 ½ months of monitoring is shown **Appendix A - Figure 7**.

The higher flows for the February 15/16 event are consistent with the higher rainfall intensities for all durations between 10 minutes and 6 hours compared to the January 10/11 event. In addition, there was a higher volume of rainfall in the days before the February 15/16 storm. The higher rainfall intensities and volume could have caused the soils to become saturated, resulting in a higher runoff flow and volume compared to January 10/11 storm event.

The recording of the highest flow at this site compared to the other two monitoring sites is not unexpected given the larger area of developed catchment compared to the other two.

When the highest recorded flow is divided by the site's catchment area, the maximum unit flow per ha is quite modest at only 6.4 L/s per ha. The daily unit flow per ha vs rainfall is shown on **Appendix A - Figure 6**. Monthly graphs of the recorded flow vs rainfall in 5 minutes increments is located in **Appendix A** in the SFE Global Sanitary and Storm Water Flow Monitoring, Winter 2013/14 report.

The lower calculated unit flow per ha compared to the smaller, more developed catchment area of DMH 879 (153 Corfield) is reflective of the large percentage (40%) of undeveloped mostly forested land of the DMH 916 (2456 Mills) catchment as well as its much larger area (141 ha vs 21 ha). The difference in soils between the two catchments may also have had an impact on runoff flows and volumes. A discussion on the local soils is presented in 4**.6 Local Soils**.

DMH (1322 Chestnut)
This monitoring site had the largest catchment area (456 ha) of the three monitoring sites and encompasses the headwaters of Romney Creek. The main stem of Romney Creek is shown to be almost 3.5 kms long on the RDN's interactive map with an elevation range of 160 m to an estimated 50 m, resulting in an average slope of 3%. Above (south of) the Alberni Highway, the terrain is flatter resulting a channel gradient around 1%.

Almost all of the catchment (98%) is located within electoral Area G of the RDN. The RDN land-use is rural, consisting of larger lots. The drainage system consists mostly of roadside ditches with driveway culverts. There is not a sanitary sewer collection system within this area of the RDN and it is understood all sewage is treated and disposed of on-site (to ground).

For this site, its maximum flow (328 L/s or 0.7 L/s per ha) was recorded during the January 10/11 storm event, whereas at the other two monitoring sites, the maximum flow was recorded during the February 15/16 storm event. During the February 15/16 event, the maximum recorded flow was 311 L/s. The daily unit flow per ha vs rainfall is presented in **Appendix A - Figure 6**. A plot of the daily maximum, minimum and average vs rainfall for the 3 ½ months of monitoring is shown **Appendix A - Figure 8**.

The rural nature of the catchment, combined with the soils (see **4.6 Local Soils)**, the rainstorm events and the amount of rainfall prior to the storm events are contributing factors to the very low unit flow per ha.

Table 7 – Maximum Flow, Dec 5, 2013 to March 21, 2014

DMH No.	(Location)	Catchment Area (ha)	Maximum Recorded Flow		Date & Time
			(L/s)	(L/s per ha)	
879	(153 Corfield St)	21	336	16.1	February 16 - 5:00 pm
916	(256 Mills St)	141	897	6.4	February 16 - 5:00 pm
1322	(281 Chestnut St)	456	328	0.7	January 11 - 6:10 am

4.6 Local Soils (Impact on Runoff Volume & Flows)

A surficial soil map published by the Geological Survey of Canada (1964) was used to identify the soil types in each of the flow monitoring catchments, analyze the results of the flow monitoring program, and determine run-off coefficients for pervious areas as part of developing the computer model.

<u>DMH 916 (256 Mills)</u>
The soils are predominately marine deposits of generally less than 1.5 m thick and ground moraine deposits, i.e., stony gravel, gravel, sand, silt, clay, stony loam, till, lenses of gravel, sand and silt – generally impermeable soils with low infiltration potential.

The map shows Quadra Sediments (sand, minor gravel; in part covered by remnants of till) in the southwest corner of the catchment as well as adjacent to and just beyond its southern boundary. This could infer more permeable soils which could result in lower runoff volumes and peak flows due to higher infiltration rates and storage capacity compared to the DMH 879 (153 Corfield) catchment which has similar soils but not the presence of the Quadra Sediments.

<u>DMH 879 (153 Corfield)</u>
The soils are predominately marine deposits of generally less than 1.5 m thick and ground moraine deposits, i.e., stony gravel; gravel; sand; silt; clay; stony loam; till; lenses of gravel; sand and silt - generally impermeable soils with low infiltration potential.

<u>DMH 1322 (281 Chestnut)</u>
The soils are predominately marine deposits of generally less than 1.5 m thick, but with a few areas showing marine deposits up to 9 m thick, and ground moraine deposits, i.e., stony gravel, gravel, sand, silt, clay, stony loam, till, lenses of gravel, sand and silt (i.e. soils with generally low permeability). Quadra Sediments (sand, minor gravel; in part covered by remnants of till) is noted around the E&N Alberni Line as well as at the gravel pit located to the west around Church Road between the E&N Port Alberni line and Hwy 4.

5 CLIMATE CHANGE

The overview of climate change, and the potential impact on municipal infrastructure and ocean levels, is addressed in **Appendix C - Technical Memorandum IDF Curve Update and Climate Change Impact**. This technical memorandum was prepared by Kerr Wood Leidal as a specialist sub-consultant to Koers. Their findings are summarized as follows:

- Climate models forecast change in total annual precipitation of between +2% to +11% by the 2050s, which is within the historical variability in annual precipitation for the baseline period from 1961 to 1990.
- Models forecast winters that are slightly wetter or about the same as the baseline period (-6% to +14% by the 2050s).
- Historical trend analysis of extreme daily and hourly precipitation for stations within the Parksville Region and Comox Airport indicate upward trends in extreme precipitation but are only statistically significant for certain stations over relatively short periods.
- A visual review of the extreme data indicates that Pacific Decadal Oscillation (PDO) cycle could play a role in extreme precipitation in the Parksville Region with the three largest events occurring during a cool PDO phase. However, additional records would be required to confirm the statistical significance of this observation.
- The PDO cycles between warm and cool phases about once in every 20 to 30 years. We may have recently shifted from a warm to cool PDO phase, which may have the effect of slowing warming trends in Southwestern BC for the next 20 to 30 years.
- Extreme precipitation in the Parksville Region is forecast to increase by:
  - 5% to 15% for daily maximum, and
  - 15% to 50% for hourly

  by the 2050s, based on recent analysis completed by Pacific Climate Impacts Consortium (PCIC).
- Given the uncertainty in the change of future extreme rainfall intensities, it is suggested drainage system resilience be improved through the use of a percentage full limit for pipe design which requires increasing to the next available pipe size if design flow results in pipe flowing more than a set limit, for example 75% to 80% full.
- Based on global sea level rise forecasts, the Provincial Government has recommended that average sea level rise of 1 m by year 2100 and 2 m by 2200 be used for coastal flood planning.
- For the Parksville region, a minimum Flood Construction Level (FCL) of 5.4 m (geodetic datum) has been calculated based on the recommended 1 m increase (to year 2100) plus allowances for wave effects and freeboard as per the provincial sea level rise guideline. However, detailed site specific analysis is recommended to establish FCL for specific coastal developments as wave effects and storm surge can be affected by local coastal processes.

6 DESIGN CRITERIA

6.1 Minor and Major Systems

The City's drainage system is defined to consist of two parts:

Minor System consisting of pipes and ditches sized for a 10 year return frequency.

Major System consisting of surface flood paths, roadways, and watercourses which convey flows of a 100 year return frequency. Major flood path routing is required wherever surface overland flows in excess of 50 L/s are anticipated.

This criterion was used in the computer modelling analyses to assess the capabilities of the storm drainage system under existing conditions and at OCP Build-Out when the City's population is expected to reach 22,000 by Year 2072.

6.2 Rainfall

The XPSWMM computer model can model rainfall based on an actual (historical) storm or a created (synthetic) storm. A brief discussion of historical storms, IDF curves and hyetographs (synthetic storms) is presented below.

6.2.1 Historical Storms

The City of Parksville has been automatically recording rainfall data in 5 minute increments at the Community Park weather station since its installation in June 2009. No winter storms with return periods greater than 5 years have been recorded during this time. The three most notable winter events are as follows along with a thunderstorm event in September 2013.

- Oct 1/2, 2009 — 2 year return period for 5 minute to 2 hour duration
- Nov 18/19, 2009 — 2 to 5 year return period for 6 hour to 24 hour duration
- Oct 21/22, 2014 — 2 year return period for 6 hour to 12 hour duration
- Sept 2, 2013 — Greater than 100 year return period for 5 minute to 2 hour duration

The September 2, 2013 event was a short (1/2 hour) duration thunderstorm. The storm dropped a total of 33.2 mm of rain between 7 pm to 7:30 pm. The heavy downpour appears to have been over a limited area due to the reports of localized flooding; most notably in the Corfield Rd/Hwy 19A intersection area. **Figure 4** presents each storm overlain on the proposed IDF curves for the City.

As there were no recorded storms with a return period meeting the City's 10 year and 100 year design standard, the City's IDF curves were used to analyze the storm drainage system.

6.2.2 IDF Curve Update

A review of the City's IDF curves, which were created in 1998, was carried out. The work was carried out by Kerr Wood Leidal (KWL) as a sub-consultant to Koers & Associates. A summary of the findings of the review is presented below and is based on KWL's Technical Memorandum; a copy of which is presented in **Appendix C - Technical Memorandum Task 7 (IDF Curves & Climate Change)**.

City of Parksville
Proposed IDF Curves & Historical Storms

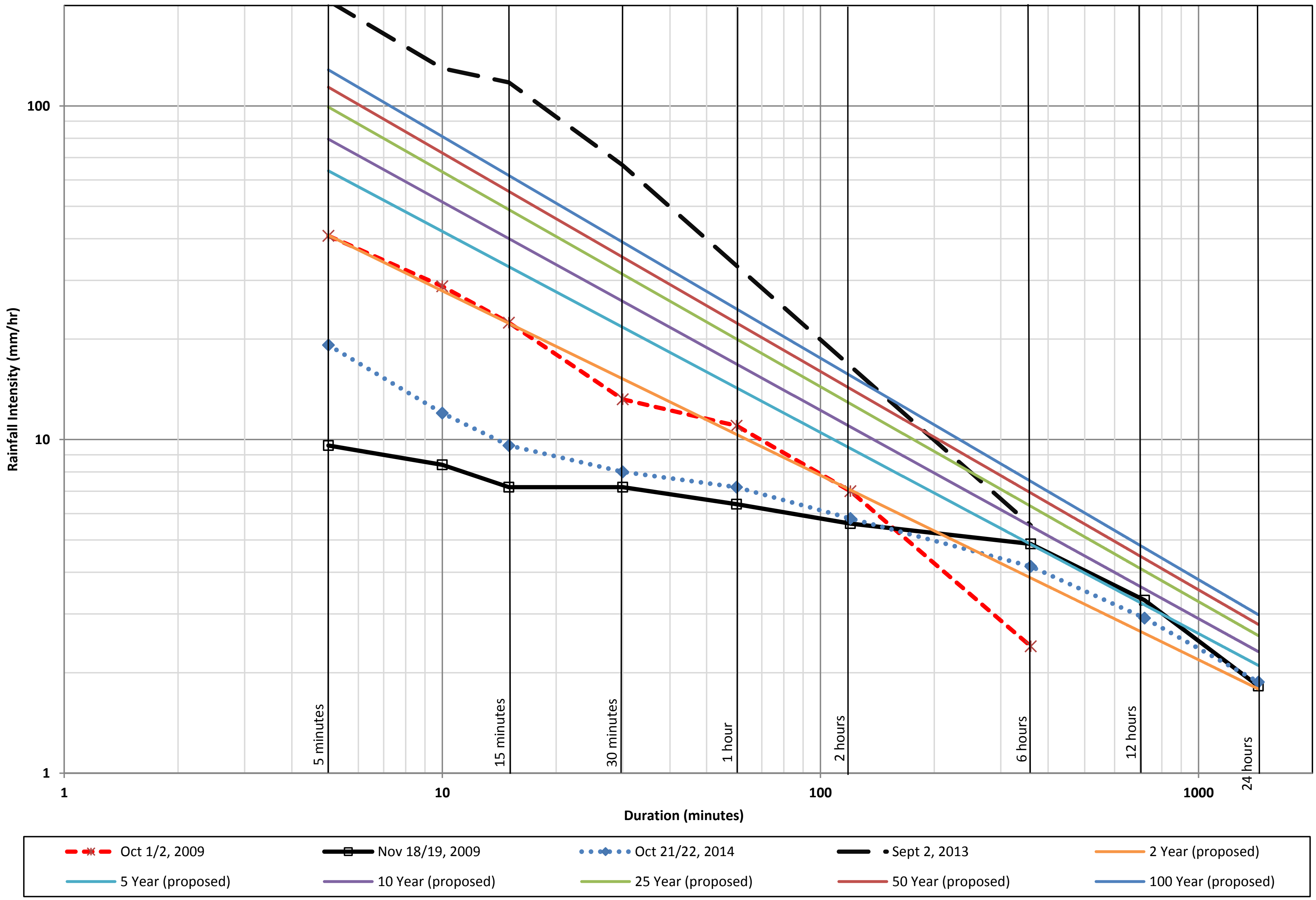


FIGURE 4

New IDF curves were calculated by factoring the Nanaimo City Yard station (10253G0) IDF most current data set available from Environment Canada to the City of Parksville South station (1025977). This spanned the 25 year period from 1980 to 2005. The values were adjusted to the City of Parksville based on the correlation between the rainfall data recorded at each station over the same time period (1983 to 1992).

The Environment Canada data for the Parksville station consists of hourly data for the 11 years between 1983 and 1992, while the Nanaimo City Yard station data set includes 5 minute intervals. While the City of Parksville Works Flowworks program has 5 minute measurements for the period 2005 to 2013, this is not considered to be a long enough period of time for IDF analysis, especially for return periods longer than 10 years.

A review of both the Nanaimo and Comox airport Environment Canada IDF curves, which span a longer time period than the Nanaimo City Yard station was carried out. The comparison found the Nanaimo City Yard station to have a better correlation with the Parkville data for the same record period (1983 to 1992).

A comparison of the rainfall intensities for the various duration and return periods shows the new curves to have a steeper slope, pivoting around the 40 to 50 minute range. As a result, the rainfall intensities for durations of less than one hour have increased by up to 34%, while those for durations of 1 hour or more have decreased by up to 31%. The City's existing IDF Curves, shown on the City's Standard Design Drawing D5, cover the durations from 5 minutes to 5 hours. The difference for each duration and return period between the existing and proposed curves is presented in **Table 8**. The existing curve intensities shown beyond 5 hour durations are based on extrapolations of the curves and are shown shaded.

Table 8 – Rainfall Intensity Comparison between City of Parksville Existing Design IDF Curves and Proposed Curves

Return Period	Rainfall Intensity (mm/hr) for varying durations								
	5 min	10 min	15 min	30 min	1 hr	2 hr	6 hr	12 hr	24 hr
Existing IDF Curves									
5 year	50.0	35.0	29.0	20.5	15.0	10.2	6.0	4.3	3.0
10 year	60.5	43.0	34.0	24.0	17.0	12.0	6.8	4.8	3.4
25 year	74.0	51.5	41.0	29.0	20.2	13.8	7.8	5.4	3.8
100 year	100.5	68.0	53.0	37.5	25.0	16.5	9.0	6.1	4.1
Proposed IDF Curves									
5 year	63.9	42.1	32.9	21.7	14.3	9.4	4.8	3.2	2.1
10 year	79.4	51.5	40.0	25.9	16.8	10.9	5.5	3.6	2.3
25 year	99.2	63.5	48.9	31.3	20.0	12.8	6.3	4.0	2.6
100 year	128.4	81.0	61.9	39.1	24.7	15.6	7.5	4.7	3.0
Change (%)									
5 year	28%	20%	14%	6%	-5%	-8%	-20%	-25%	-31%
10 year	31%	20%	18%	8%	-1%	-9%	-20%	-26%	- 1%
25 year	34%	23%	19%	8%	-1%	-7%	-19%	-25%	-31%
100 year	28%	19%	17%	4%	-1%	-6%	-17%	-22%	-31%

The proposed IDF curves are shown in **Figure 4**.

6.2.3 Hyetographs

The distribution (amount and intensity) of rainfall over time is called a hyetograph. Synthetic hyetographs have been developed across Canada and the USA based on the analysis of actual rainstorm events. Hyetographs vary across the country and require rainfall data to be created. This data comes from Intensity-Duration-Frequency (IDF) curves published by Environment Canada.

Four rainfall distribution patterns were considered:

- Atmospheric Environment Services Canada (AES)
- Soil Conservation Services (SCS)
- Huff - 1st, 2nd, 3rd and 4th Quartiles
- Chicago Distribution

AES hyetographs were developed specifically for Canada for 1 and 12 hour storm events. Hyetographs are available nationwide for various regions of the country including the coast of British Columbia.

SCS hyetographs were developed for various storm types, duration and regions of the United States. Four types have been developed (I, IA, II and III) with IA developed specifically for the pacific coast covering the states of Washington, Oregon, and the northern half and part of the middle of California. They reportedly can be applied to storm events from 1 to 48 hours.

Huff hyetographs were developed by the state of Illinois. The 1st quartile is for storms less than 6 hours durations. The 2nd quartile is for storm of 6 to 12 hours durations. The 3rd quartile is for storm of 12 to 24 hours and the 4th quartile is for storms longer than 24 hours.

The Chicago hyetograph have been found to result in higher peak flows compared to other hyetographs as the hyetograph is developed on the assumption that the design storm contains all the maximum intensities for various durations. It is not considered for this study as it does not represent rainfall patterns for the BC coast.

The AES hyetograph for coastal BC was modeled to assess the City's storm drainage system. The model was run with 30 minute, 1 hour, 2 hour, 6 hour and 12 hour duration storm events using the existing and proposed IDF curves for the 10 year and 100 year return periods. Peak flows experienced in each pipe were then compared. The analysis determined the 1 hour storm was the governing event, with the exception of the Romney Creek catchment (Bayside Inn outfall), where the 6 hour duration storm governed.

Figure 5 compares the AES and Huff 2nd quarter rainfall distribution for a 1 hour storm.

Figure 6 shows the one hour design rainfall for the 10 year and 100 year storm events.

Figure 7 shows the six hour design rainfall for the 10 year and 100 year storm events.

KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

Parksville
Synthetic Storm Rainfall Distribution Comparison
(10 Year, 1 hour Duration)

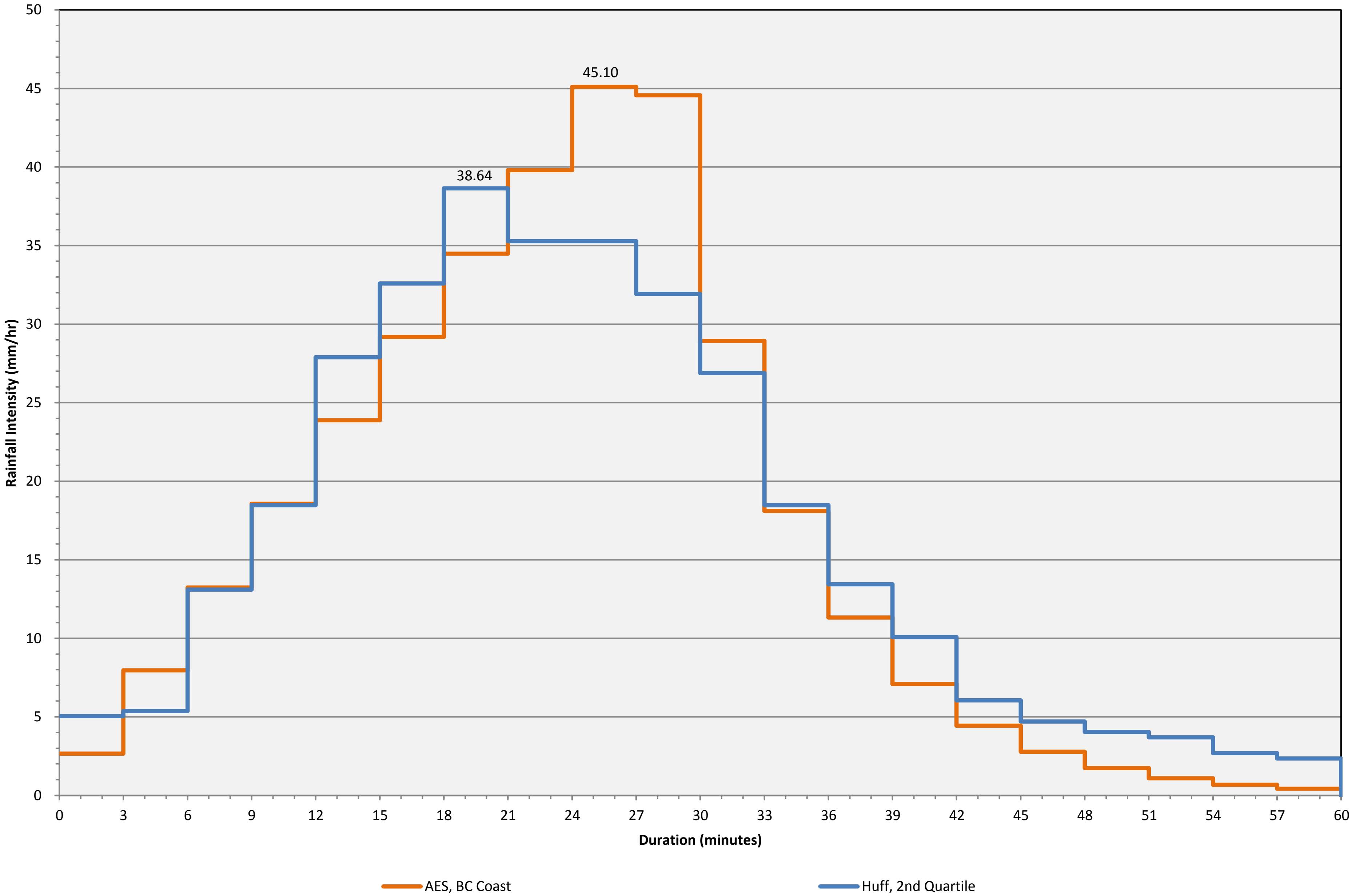


FIGURE 5

City of Parksville
1 Hour Design Rainfall
10 Year & 100 Year

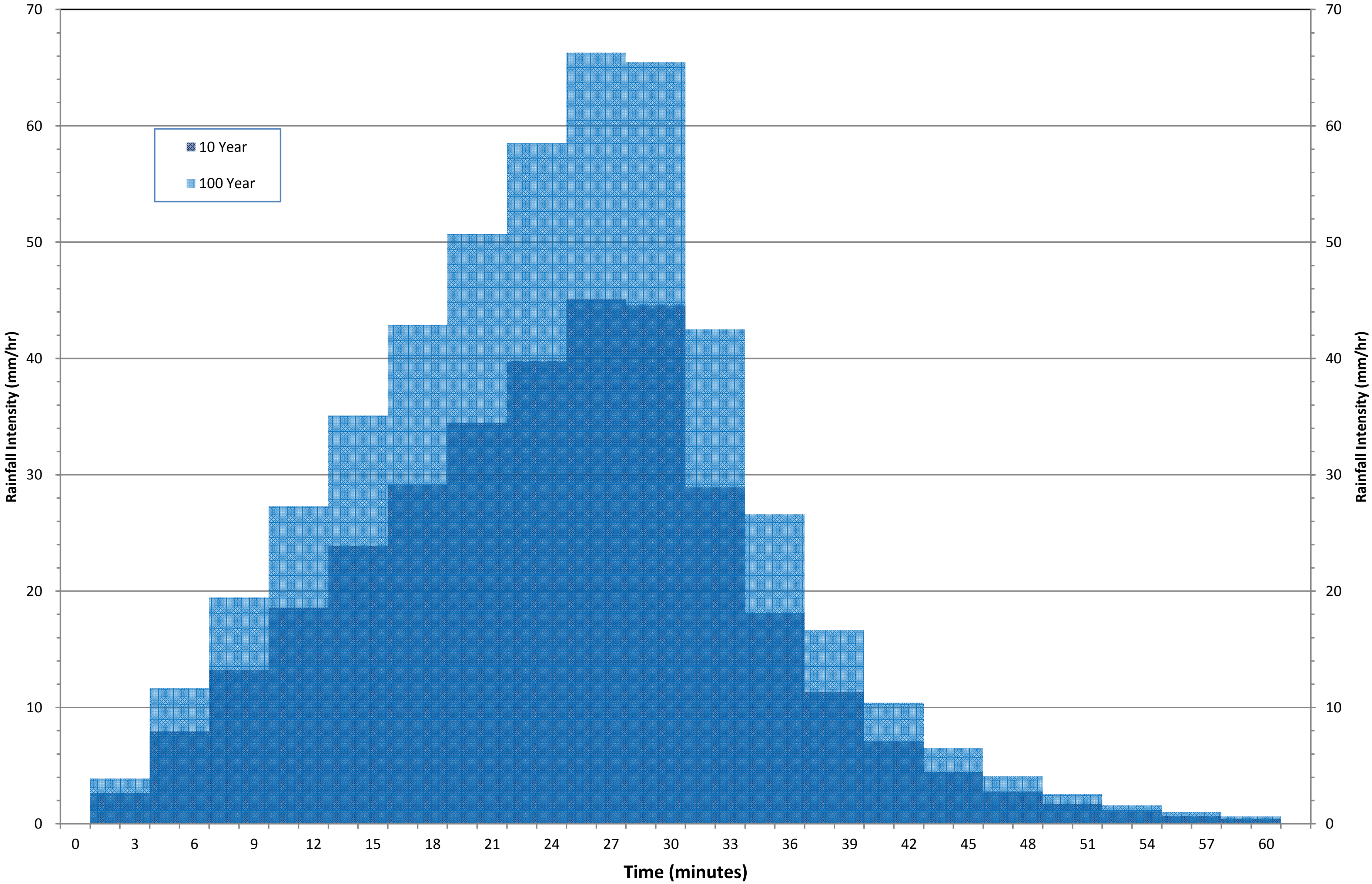


FIGURE 6

City of Parksville
6 Hour Design Rainfall
10 Year & 100 Year

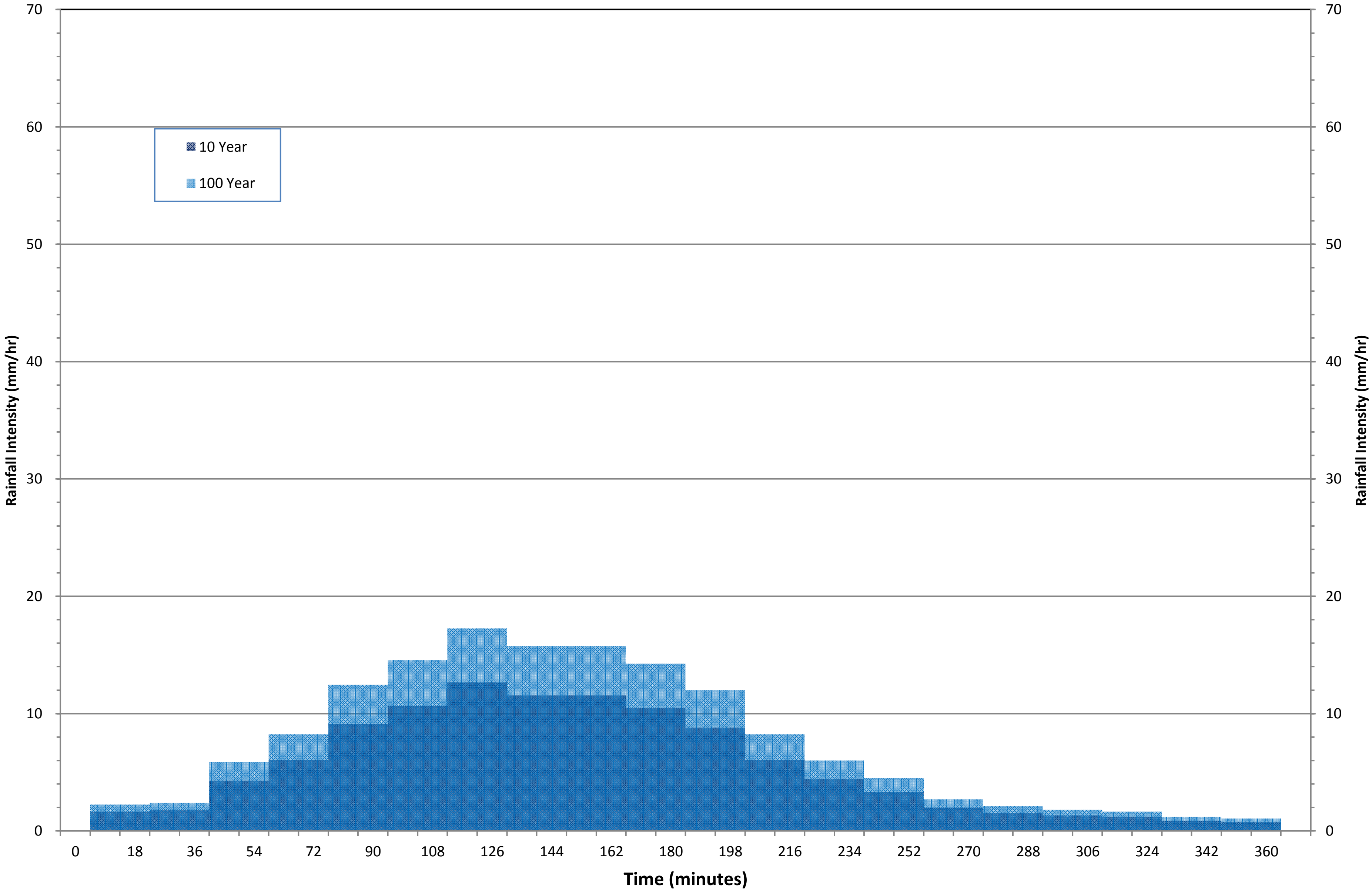


FIGURE 7

7 COMPUTER MODEL

7.1 Computer Model

As part of development of the Storm Sewer Master Plan, the City retained Koers to assist in selecting the most appropriate software for modelling and analyzing the City's storm and sanitary sewer collection systems for existing and future conditions and for analyses of development applications on an ongoing basis, as warranted.

The City had a computer model of the storm sewer system; a model developed by Koers using the computer software program MIDUSS as part of the 1998 Storm Drainage Study Update. The City also has pipe data within their GIS system (MapGuide) including location, size, inverts and MH rim elevations.

A review and analysis of five computer software programs was carried out. The programs reviewed were:

- XP-SWMM by XP Solutions
- PC-SWMM by CHI
- HYDRA by Pizer Inc.
- SewerGEMS by Bentley Systems Inc.
- Autodesk Sanitary by Autodesk

Upon completion of the review process, the City selected XP-SWMM for modelling of the storm drainage system as well as their sanitary sewer system.

The detailed review of the software programs and the rationale for selection of XP-SWMM is presented in **Appendix D - Technical Memorandum No. SS-1 & SD-1 (Software Evaluation)**.

7.2 XP-SWMM Model Overview

XP-SWMM (by XP Solutions) is a comprehensive software package that has been in use for over 25 years for planning, modeling and managing storm drainage and sanitary sewer systems. It is a powerful, user friendly graphical computer program that allows the user to easily change data parameter on an individual or global basis and to interact with the modelling input and output data both graphically and in tabular format. The program can interface with AutoCAD and GIS programs, including the City's MapGuide program.

The program can carry out real time simulations review and present model results through customizable animations. The program can be coupled with a two dimensional surface grid for comprehensive flood modeling and mapping. The program is used for the design and analysis of both synthetic and actual events.

The program has a number of stormwater uses, including, but not limited to:

- STORMWATER MANAGEMENT
  - Stormwater master planning
  - Collection system design & analysis
  - Detention facility sizing and optimization
  - Stormwater treatment analysis
  - Flood and hazard mapping

- HYDROLOGY
  - Modelling of actual and design rainfall events
  - Single event and continuous simulation
  - Groundwater infiltration and discharge
  - Temporary storage

- WATER QUALITY
  - Pollutant buildup and wash off
  - Street sweeping
  - Pollutant transport
  - Treatment analysis and optimization
  - Sediment transport
  - BMP and LID Analysis

The program can accommodate an almost limitless number of conduit shapes as well as changing roughness coefficients as a function of flow depth. Flow splitting/diversions can be used to direct flow by means of weirs or orifices.

Pump stations, which are more common with sanitary sewer systems than with storm drainage systems, can be represented as either an in-line lift station, or an off-line node representing a wet-well. Up to seven pumps may be assigned to a single pumping station, each with their own operating settings, including variable speed pumps. Pump curves, on/off levels and pumping rates based on wet well depth, pump curves and forcemain diameter and lengths can be entered to accurately model existing and proposed conditions.

Gates valves, flow regulators, moveable weirs and telemetry controlled pumps can be modelled using the Real Time Control (RTC) add-on module. The controls can be set using any combination of time and date variables, velocity and flow, depth and elevations, pump flows, weirs or orifices.

The program allows displaying of input and output data using layers which can be switched on or off. Background images, AutoCAD drawings or GIS data can be imported into the program for model development and analyses.

Customized tables can be generated for both data input and modelling results. Graphs of model results can be displayed for a single or multiple objects. Up to 16 graphs can be displayed on a single page. Results for any pipe can be viewed by clicking on the pipe. Digital Terrain Models (DTMs) can be incorporated into the model and used for animation of modeling results.

More detailed information on the computer program includes minimum operating system requirements are presented in **Appendix E - XPSWMM Technical Literature**.

8 COMPUTER MODEL DATA ENTRY & CALIBRATION

A calibrated model was developed and served as the basis for the analysis of the two scenarios; existing conditions, and at OCP Build-out. A discussion of how the models were developed is presented below.

8.1 Data Collection & Entry

Pipe Network
The City's storm drainage system was imported from their GIS database. The imported information included pipe diameter, pipe slope, pipe material, manhole rim and invert elevations, horizontal location of the pipes, manholes and pump stations.

Recent new development and upgrading works not yet incorporated into the City's GIS data were added to the model manually from the information on available record drawings.

Surface Roughness Coefficients
The Manning's 'n' values listed in **Table 9** were applied to the model to account for varying types of flow path (flow channel) surfaces. These are the values in the City's Engineering Standards and Specifications (page D-1.7).

Table 9 – Manning's 'n' Values

Surface Type	Manning's 'n'
Natural streams and grassed channels	0.050
Excavated ditches	0.030
Corrugated Steel Pipe	0.024
Asbestos Cement, Clay, or Concrete pipe	0.013
Concrete or Asphalt lined channels	0.013
PVC pipe	0.011

Cadastral
The City's cadastral was imported from their GIS database.

Catchment Areas
Catchment areas boundaries for each pipe were created digitally within the model based on how each property is serviced. The catchment area boundaries developed for MIDUSS model developed for the 1998 Storm Drainage Study served were also used as a reference.

Catchment areas of undeveloped properties that will require installation of new mains and how they will connect to the City's system, was established utilizing digital ground contour information from the City (LIDAR maps) and proposed development plans, if available.
Catchment area boundaries for the areas of Shelly and Romney Creek that are outside of the City were established based on the 1998 study in conjunction with local knowledge of ground contours in the Provincial and the RDN's on-line IMAP program.

Percentage of Impervious Area
The percentage of impervious area for each catchment area was entered manually based on the current land-use. A number of areas within the City were checked to confirm the overall % impervious for various land-uses. Calculations were performed using aerial photographs for large lot single unit residential, small lot single unit residential, multi-unit residential, and commercial. The percent impervious for each is presented in **Table 10**. These values were applied throughout the existing conditions and OCP build-out model and include allowances for impervious areas within the municipal roadway (paved road and sidewalks).

Table 10 – Land Use and Percent Impervious

Land Use	Percent Impervious
Single Unit Residential - large lot - small lot	 30% 45%
Multi-Unit Residential	65%
Commercial	80%
Institutional (not including playfields)	80%
Industrial	80%

Pervious Areas Runoff Coefficients
The Soil Conservation Service (SCS) method has been chosen for the modelling of the runoff capability (or the infiltration capability) of soils because it is the most common of the options available in the program and the inputs are easily obtainable or estimated. It is a simple and widely used method for estimating runoff from a rainfall event.

The method uses a soil runoff curve numbers (CNs) that have been developed based on the type of soil, the infiltration capability of the soil, the land use on the soil, and the depth of the seasonal high water table. To account for the variation in the infiltration capacity of soils, they are divided into four hydrologic soil groups (HSGs):

- Group A = Soils with high infiltration rates (>7.5 mm/hour) even when thoroughly wetted.
- Group B = Soils with moderate infiltration rates (4 to 7.5 mm/hour) when thoroughly wetted.
- Group C = Soils with slow infiltration rates (1 to 4 mm/hour) when thoroughly wetted.
- Group D = Soils with very slow infiltration rates (<1 mm/hour) when thoroughly wetted.

In developing the storm drain computer model for the City of Parksville, soil classification data was obtained from the 1:40,000 scale colour soil map published by the Geological Survey of Canada (1964). The map indicates four types of soils within Parksville. The locations indicated on the map are consistent with current observations. These four types are listed in **Table 11** along with the selected soil group and CN chosen for modelling purposes. The CN values chosen for modelling purposes were based on a wettest Antecedent Moisture Condition (AMC 3) signifying soils are saturated from previous rains and have the highest runoff potential. The antecedent moisture condition reflects the moisture of the soil before the rainfall event. AMC 1 represents dry conditions with minimal runoff. AMC 2 represents average conditions.

Table 11 – Soil Types and SCS Curve Number

Soil Type	Area	Soil Group & Curve Number for AMC 3
MARINE DEPOSITS Stony gravel, gravel, sand, silt, clay, stony loam with thicknesses generally less than 1.5 m underlain by till, lenses of gravel, sand and silt.	- West, south & southeast of Parksville Bay	C, 83
SHORE DELTAIC & FLUVIAL DEPOSITS Gravel, sand, silt, clay, peat	- Englishman River Estuary area - Pioneer Crescent area - Herring Gull Industrial Park - Englishman River (east and west sides) south of Hwy 19a - South of Craig Bay	A, 70
Areas of Bedrock Outcrop and outcrop interspersed with patches of thin overburden	East side of Englishman River: -south of Hwy 19a to Rathtrevor Rd - between Hwy 19 and E&N railway	D, 90
TERRACED FLUVIAL DEPOSITS Gravel and sand commonly underlain by silt and clay	- South of Englishman River Estuary - West of Craig Bay	B, 78

For Parksville, a CN value of 83 was selected with the exception of lands around the Englishman River Estuary, Pioneer Crescent area and Craig Bay which were assigned a value of 70 to reflect the sandy soils and their higher infiltrative capacity. A soil with a low CN will generate less runoff compared to a soil with a higher CN.

<u>Time of Concentration (Tc)</u>

The time it takes for surface runoff to enter the City's storm drainage system is the Time of Concentration. It can be calculated by several methods within the SWMM program based on the length it has to travel, the slope of the surface it travels over, and the roughness of the surface. The flow path includes both developed and undeveloped surfaces such as front, back and side yards, landscaped areas, paved surfaces, sidewalks, driveways, roads, rooftops, gutters and individual storm drain service connections. Entering the data for each catchment in the City is not practical or necessary for a model of this size. Instead, a time value based on the size of the catchment area was used for catchments up to 10 ha. For greater than 10 ha, the Bransby Williams formula was used to calculate the Tc. This formula:

$$Tc = 0.605 \frac{L}{S^{0.2} A^{0.1}}$$

Where:

T_c = Time of Concentration (hours)
L = Gross length of main channel (km)
S = Net slope of main channel (%)
A = Watershed area (km^2)

was used for the portions of the catchments located outside of the City's boundaries for Carey Creek, Romney Creek and Shelly Creek. The length and slope of the flow path (channel) and the contributing area for each was obtained from the 1998 Storm Drainage Study and from digital mapping from the RDN's IMAP program.

The Tc criteria used in the model are presented in **Table 12**.

Table 12 – Time of Concentration

Catchment Area	Time of Concentration (minutes)	
	Developed	Undeveloped
0 - 0.5 ha	5	10
0.5 - 1 ha	7.5	15
1 - 1.5 ha	10	20
1.5 - 2 ha	12.5	25
2 - 6 ha	15	30
6 - 10 ha	17.5	35
< 10 ha	Bransby Williams Formula	

Database Checking

The computer model was run to check the connectivity of the piping system and hydraulic grade line. System errors, such as pipe surcharging, revealed data entry errors, like incorrect pipe diameters, manhole rim or invert elevations, or different vertical datum on a few of the oldest sewer pipes. Field observations and record drawings were consulted and the model updated.

8.2 Calibration Model

The computer model for existing conditions was calibrated using the flow recorded during the winter flow monitoring program carried out between December 2013 and mid-March 2014 (3 ½ months). Data was recorded at the three sites previously discussed in section **4.2 Monitoring Sites**.

The following process was followed in calibrating the computer model:

.1 Rainfall data recorded at the City's Community Park station was entered into the model (in 5 minute increments).

.2 The model was run and hydrographs generated at each of the flow monitoring sites for the two largest rainstorm events (Jan 10-11, 2014 and Feb 15-16, 2014) as noted previously in section **3.3 Rainfall Events**.

The SWMM generated hydrographs were compared with the recorded hydrographs to establish compatibility. The modelled CN curve numbers were adjusted and the model rerun and the generated hydrographs compared again. This iterative process was repeated until the appearance of the two hydrographs was similar. The peaks of the SWMM hydrograph were higher than the recorded at two locations: the Chestnut St site and the Mills St site. A brief discussion of those findings at each of the three monitoring sites is presented below.

DMH 1322 (281 Chestnut St)

As shown in **Figure 8**, the leading edge of the model calculated flow resembles the recorded flow but then contains a decrease followed by a sharp rise to a peak notably higher than the recorded value and is followed by a sharp drop to below the recorded flow. The model calculated flow begins later and ends sooner than the recorded value. These differences are not unexpected given the large size (456 ha) and rural nature of the catchment area, and the information available to develop the catchment area boundaries, channel flow paths and Manning's "n" values.

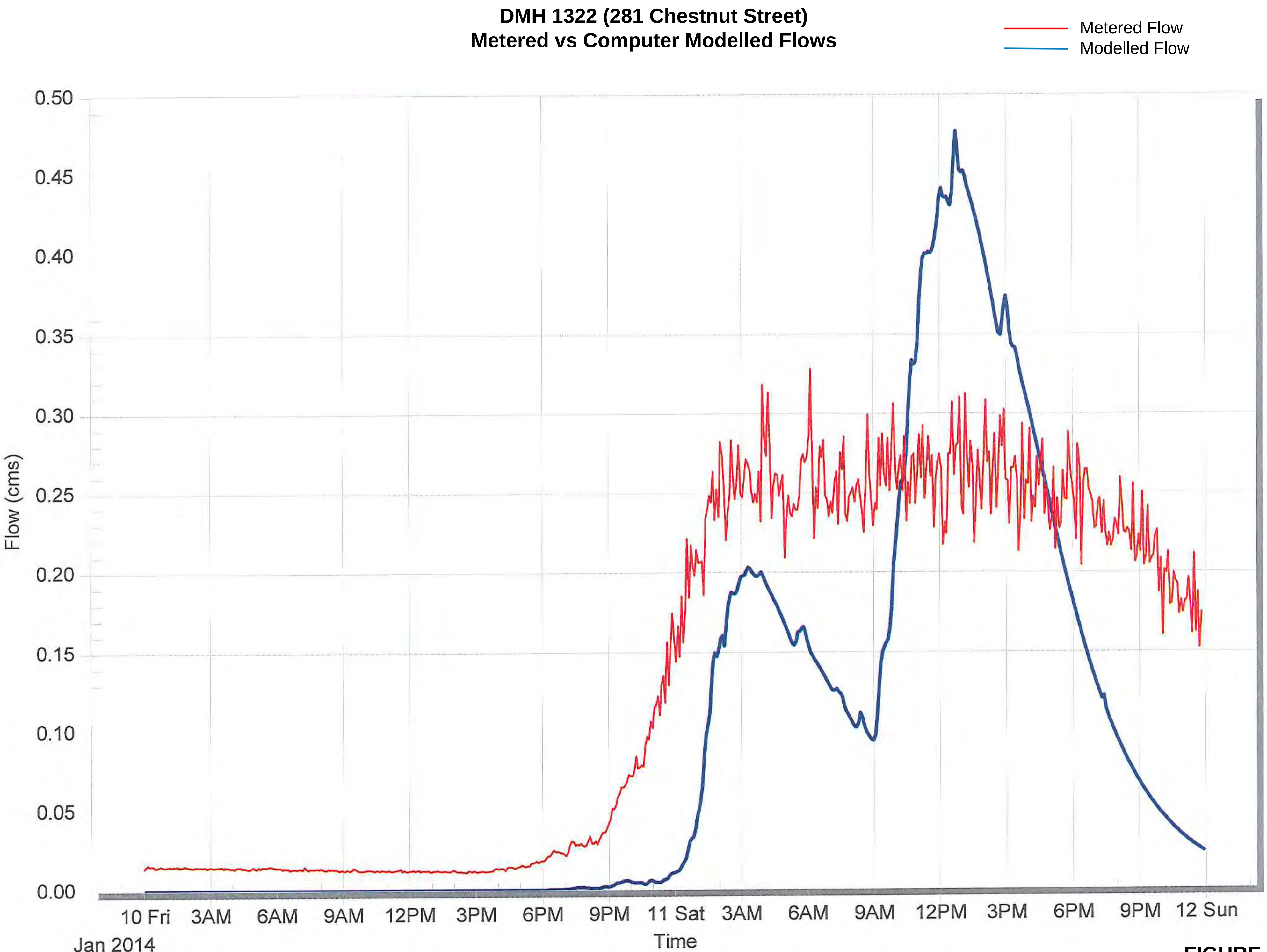

DMH 1322 (281 Chestnut Street)
Metered vs Computer Modelled Flows
Metered Flow
Modelled Flow
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Flow (cms)
10 Fri
Jan 2014
3AM
6AM
9AM
12PM
3PM
6PM
9PM
11 Sat
3AM
6AM
9AM
12PM
3PM
6PM
9PM
12 Sun
Time

FIGURE 8

The recorded base flows in the system are higher while peak flows are muted compared to the modelled data. This is most likely in response to localized areas of temporary storage within the system. Significant work would be necessary to refine the model to increase base flows and mute peak flows. Given that the calculated flows were higher than the recorded, it provides a measure of conservatism in assessing the capability of the enclosed storm drain system servicing Romney/Carey Creek.

The catchment area of the monitoring site is shown in **Appendix A Figure 1**.

<u>DMH 879 (153 Corfield St)</u>
Of the three monitoring sites, the model calculated flow most closely matched the recorded flow at this site as shown in **Figure 9**. This is perhaps reflective of the developed urban condition of the catchment and the extensive enclosed storm drainage system.

It is noted that the recorded flows decline at a slower rate than calculated by the model. This may be reflective of the 20% portion of the catchment that is undeveloped, and may be shedding rainfall at a slower rate than that calculated by the model.

The catchment area to the monitoring site is shown in **Appendix A Figure 1**.

<u>DMH 916 (256 Mills St)</u>
As shown in **Figure 10**, the leading edge of the model calculated flow resembles the recorded flow for the first 9 hours but then has much larger peaks and valleys than the recorded flow, which has two peak flows close together (between 12 am and 3 am), followed by a gradual decline.

As with the DMH 1322 (281 Chestnut St) monitoring site, the recorded base flows in the system are high while peak flows are muted compared to the modelled data. This is not unexpected given the larger size (141 ha) and 42% of the catchment that is undeveloped. The undeveloped lands are most likely shedding rainfall at a slower rate and there may be areas of localized low spots that are detaining runoff. It is expected that under OCP Build-Out recorded vs model calculated flows would be much closer, as the undeveloped lands would be fully developed. Given that the calculated flows were higher than the recorded flows, a better measure of conservatism in assessing the capability of the enclosed storm drain system

The catchment area to the monitoring site is shown in **Appendix A Figure 1**.

8.3 Existing Conditions

Using the calibrated computer model, the City current IDF curves were used to generate the rainfall intensities for a 10 year and 100 year storm event.

The CN values were adjusted to AMC 3 (wet) and the model run. All pipes flowing ≥100% full during the 10 year event were noted.

A 10 year and 100 year rainfall event was also developed using the proposed IDF curves. The resulting peak flows were nearly identical with those generated with the current IDF curves. This is reflective of the minimal difference in the IDF curves at the 1 hour period, (see **Table 8**), which was found to be the governing time of concentration for the City's storm drainage collection system, with the exception of the Romney Creek catchment (Bayside Inn outfall) and Shelly Creek where a four to 6 hour time of concentration governs due to the large rural, gentler sloped catchment areas in the RDN.

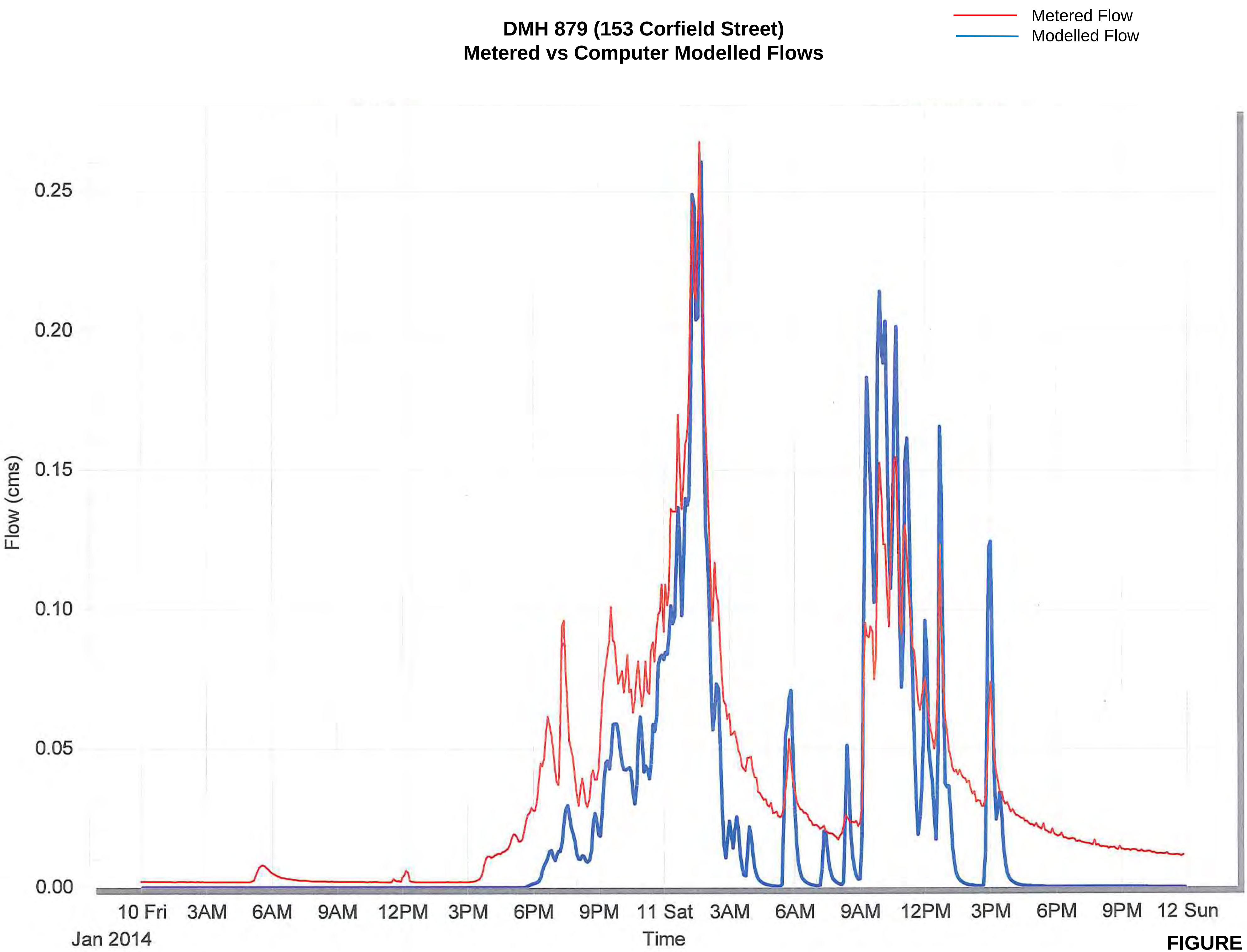

DMH 879 (153 Corfield Street)
Metered vs Computer Modelled Flows
Metered Flow
Modelled Flow
Flow (cms)
0.25
0.20
0.15
0.10
0.05
0.00
10 Fri
Jan 2014
3AM
6AM
9AM
12PM
3PM
6PM
9PM
11 Sat
3AM
6AM
9AM
12PM
3PM
6PM
9PM
12 Sun
Time

FIGURE 9

DMH 916 (256 Mills Street)
Metered vs Computer Modelled Flows

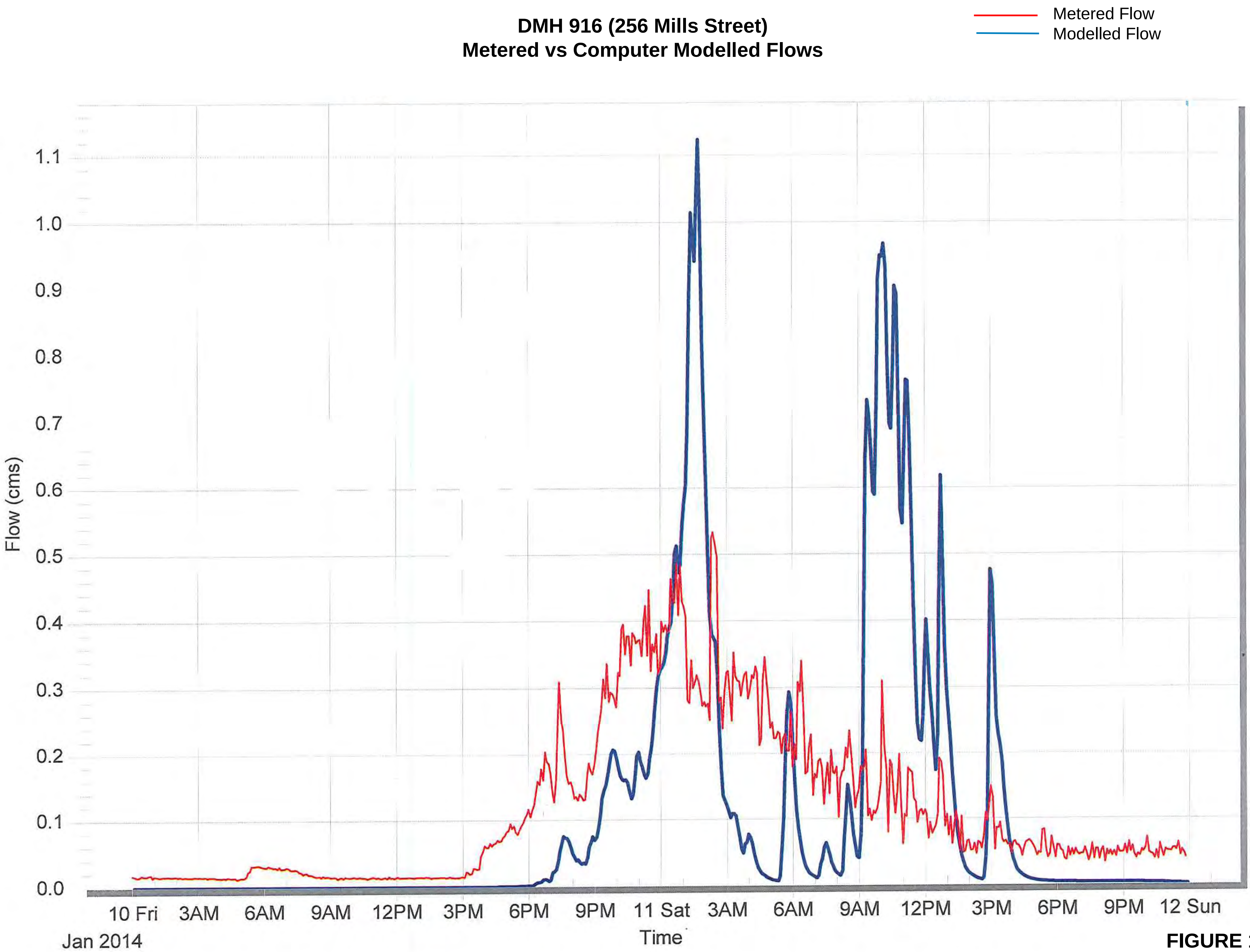


FIGURE 10

The computer model was run and pipes flowing ≥100% full for the 10 year event were identified.

A copy of the City's current land-use plan is presented in **Figure 11**.

8.4 OCP Build-Out

The OCP Build-Out model was developed from the Existing Conditions model.

Undeveloped lands identified for future development were assigned designated land-uses in accordance with the OCP and the corresponding percent imperviousness applied. Adjustments to percent imperviousness of areas to account for redevelopment/rezoning were also made. A copy of the OCP is presented in **Figure 12**. The areas where growth is anticipated and the resulting change in the percent imperviousness are shown in **Figure 13**.

The computer model was run and pipes under capacity for the 10 year event were identified.

8.4.1 Municipal Boundary Expansions

The OCP Build-Out model is based on the current municipal boundaries. No allowance has been made for boundary expansion of either developed or undeveloped lands within the RDN.

8.4.2 RDN Controlled Lands

The OCP Build-out condition model does not include any allowance for increasing flows from lands within the RDN that drain into the City of Parksville.

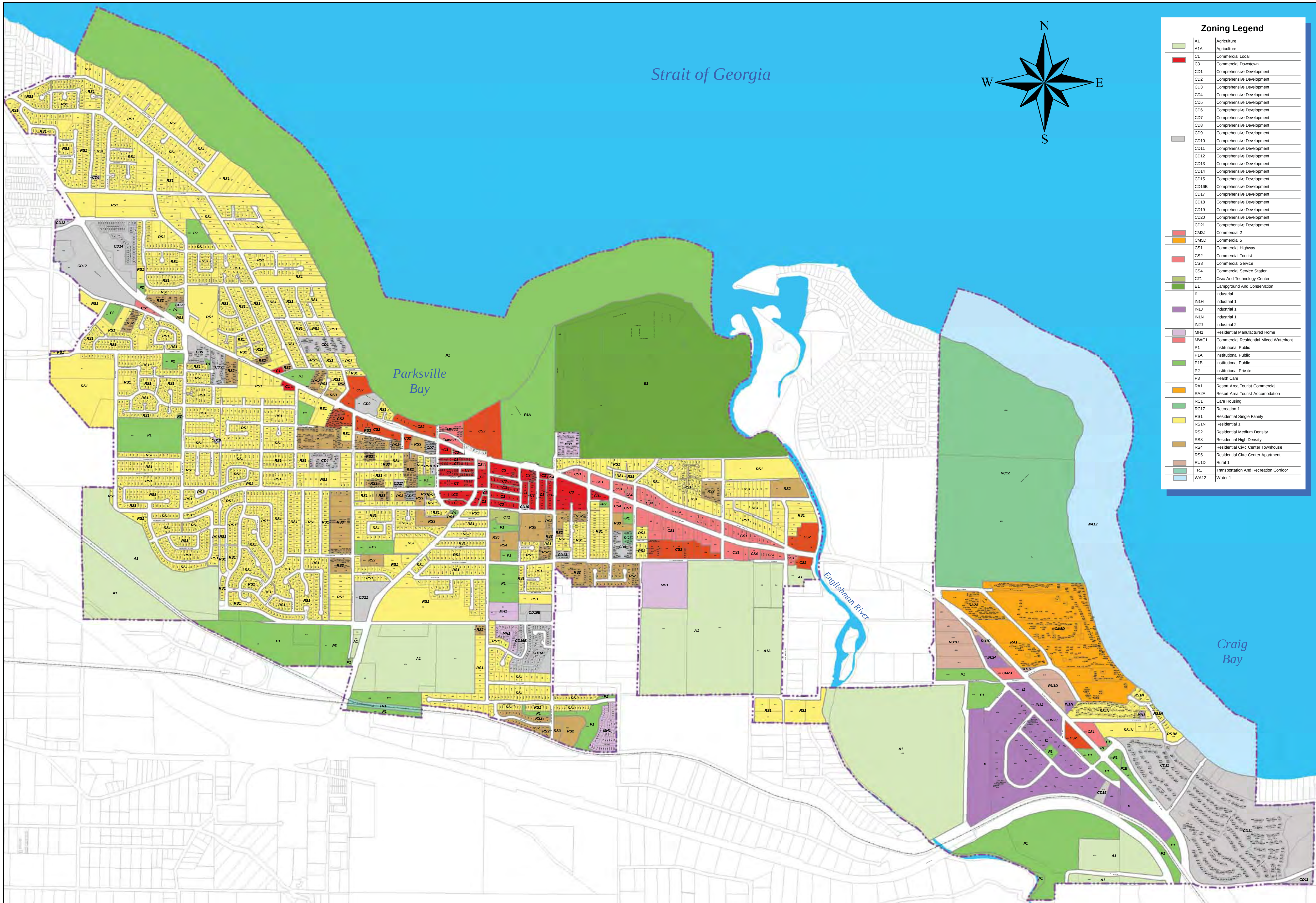
The largest drainage area outside of the City is Romney Creek with a total catchment at 720 ha. It is noted that more than 400 ha (55%) of its catchment is located within the RDN.

The 2nd largest catchment outside of the City is Shelly Creek at 650 ha. More than 570 ha (almost 90%) of its catchment is also located within the RDN.

It is noted that MoTI is responsible for subdivision approval and the maintenance of roads and drainage works outside of the City of Parksville while the RDN is responsible for zoning.

8.4.3 E&N Railway Corridor

It is also noted that the Island Corridor Foundation (ICF) owns the E&N railway which also serves as part of the City's southern boundary. There are several culverts under the tracks that drain into the city. The railroad bed which is built up on ballast in several locations has a tendency to act as a berm and can impound runoff during heavy rainfall events.



City of Parksville

Zoning and Development Bylaw, 1994, No. 2000

Consolidated to June 7, 2010 for Convenience Only

Map Amendments

Bylaw No.	Date of Adoption	Bylaw No.	Date of Adoption
2000.1	October 3, 1994	2000.34	October 7, 2002
2000.2	October 28, 1994	2000.37	May 5, 2003
2000.4	February 17, 1997	2000.38	August 18, 2003
2000.5	January 20, 1997	2000.39	December 15, 2003
2000.7	November 12, 1996	2000.40	March 15, 2004
2000.11	April 21, 1997	2000.41	April 5, 2004
2000.12	November 12, 1996	2000.44	October 3, 2005
2000.13	October 27, 1997	2000.47	June 20, 2005
2000.15	October 6, 1997	2000.48	July 5, 2006
2000.17	May 7, 2001	2001.49	February 4, 2008
2000.19	July 19, 1999	2000.50	August 9, 2006
2000.21	January 17, 2000	2000.51	August 9, 2006
2000.22	March 6, 2000	2000.52	November 6, 2006
2000.23	June 19, 2000	2000.53	October 2, 2006
2000.24	December 17, 2001	2000.55	April 2, 2007
2000.27	December 17, 2001	2000.56	August 20, 2007
2000.28	October 15, 2001	2000.58	August 20, 2007
2000.29	March 19, 2001	2000.61	July 21, 2008
2000.33	August 7, 2002	2000.77	May 3, 2010

FIGURE 11

DISCLAIMER

THIS INFORMATION IS PROVIDED FOR CONVENIENCE ONLY AND IS NOT THE OFFICIAL OR LEGAL VERSION OF ANY CITY DOCUMENT.

Project Location: I:\USERS\GIS\Map Projects\Zoning.mxd

PDF Location: I:\USERS\GIS\PDF Maps\Zoning.PDF

Map Creation Date: June 28, 2010

Map Created By: R. Hall

275 137.5 0 275 550 Meters

Schedule "B" - Future Landuse Map

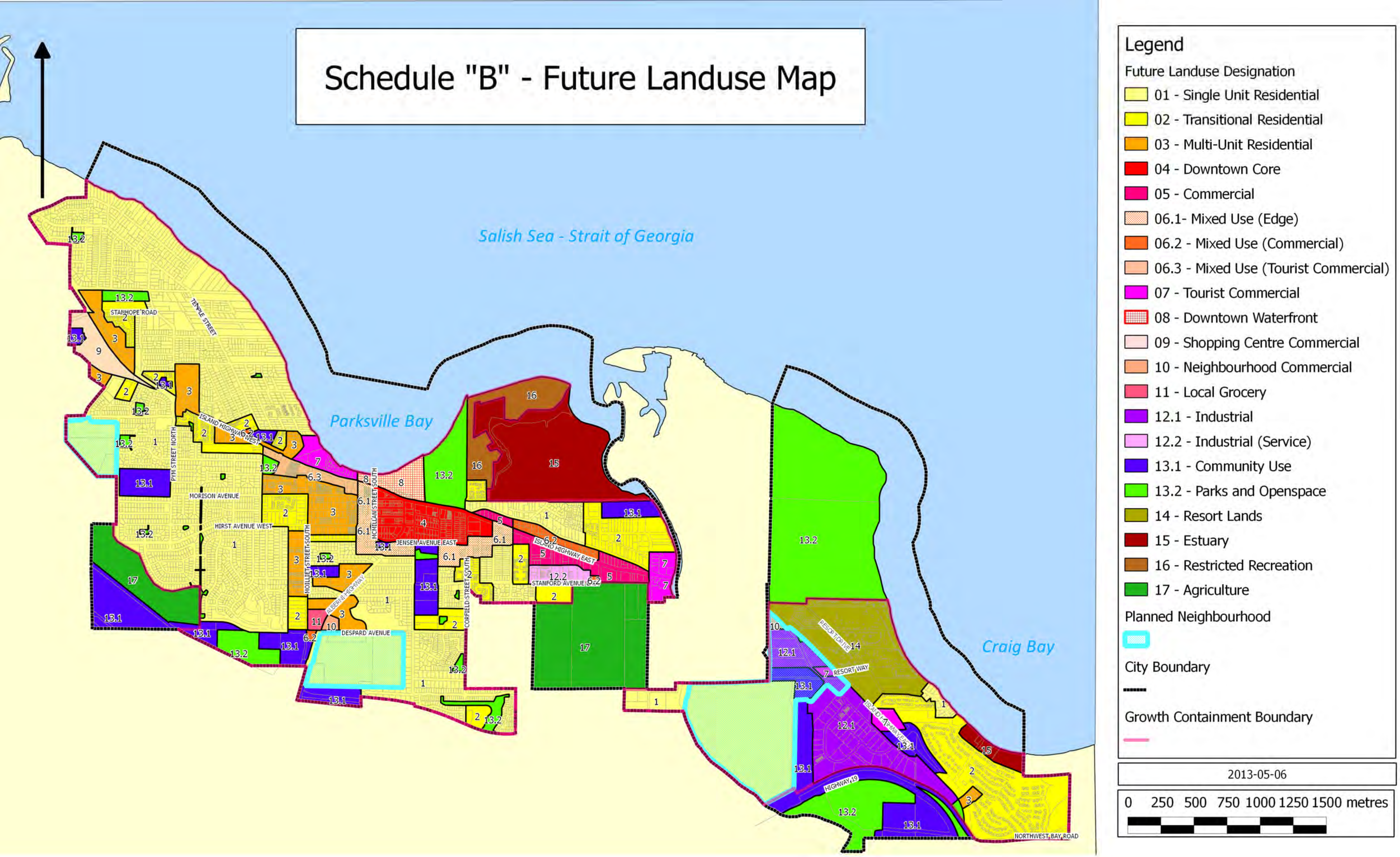
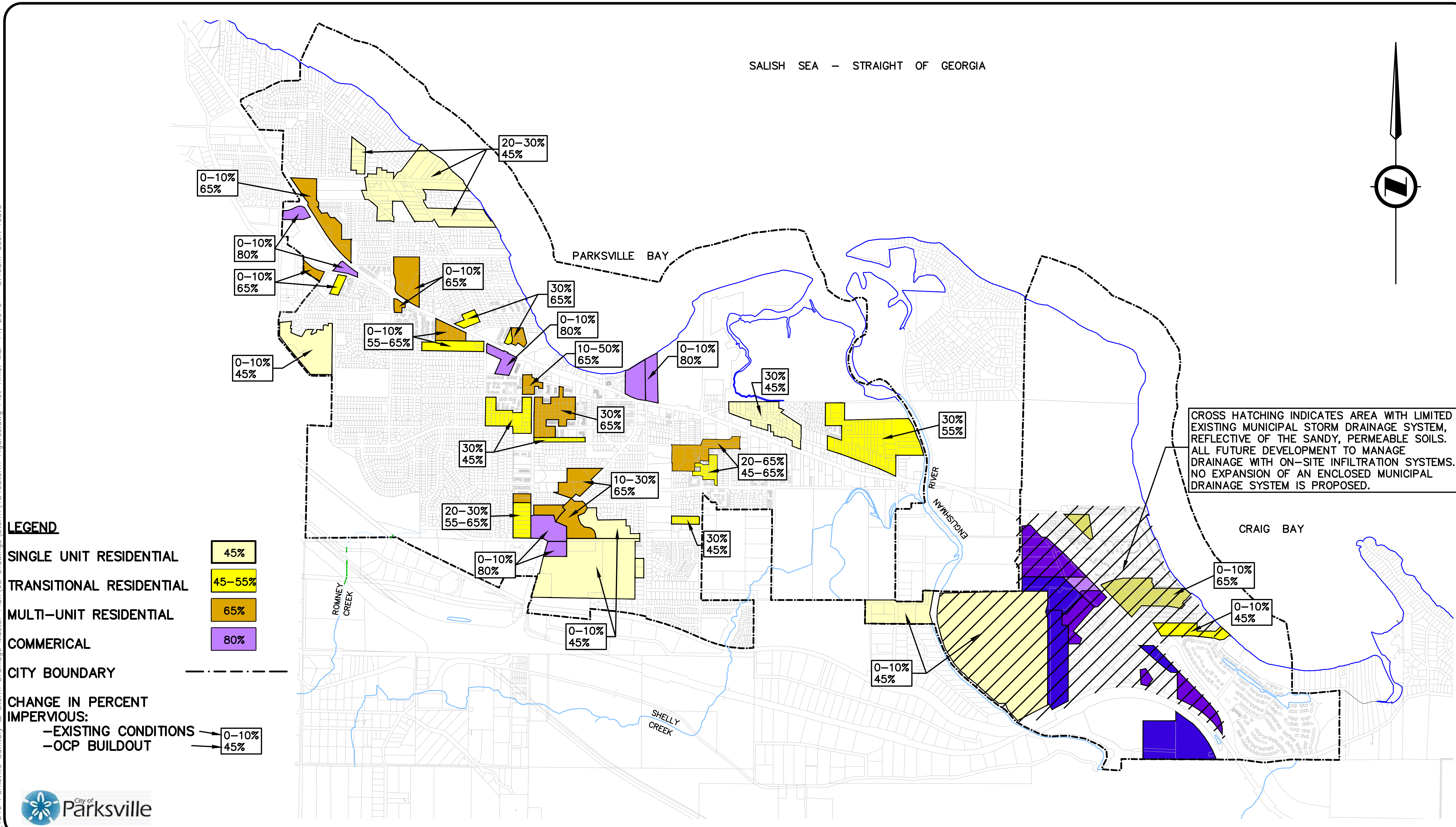


FIGURE 12



KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

CLIENT	CITY OF PARKSVILLE
PROJECT	STORM DRAINAGE MASTER PLAN
TITLE	PROJECTED AREAS OF GROWTH
APPROVED	
DATE	MAR 2015
PROJECT No.	1346
SCALE	1:25,000
DWG No.	FIGURE 13

9 MODELLING RESULTS

9.1 General

The XPSWMM computer model was run using synthetic storms derived from the proposed updated IDF curves for the City of Parksville for the 10 year and 100 year return period. A total of five storms with durations of 30 minute, 1 hour, 2 hours, 6 hours and 12 hours were run to determine the worst case peak flow for each section of pipe, ditch, culvert or drainage course. The 1 hour storm was found to govern, which is expected given the size of the catchments. The exceptions to this were the Romney Creek and Shelly Creek catchments, where the 6 hour duration produced the highest peak flows. In addition, the September 2, 2013 rainstorm event, a short duration high intensity rainstorm, with a return period greater than 100 year for durations up to 2 hours, was modelled.

In general, unit runoff rates for the 10 year events were as follows:

- Undeveloped Area
  - Soil Group A, CN of 70 = 5 - 12 L/s per ha
  - Soil Group C, CN of 83 = 12 - 40 L/s per ha
- Developed Areas
  - 30% impervious area = 27 - 35 L/s per ha
  - 45% impervious area = 34 - 48 L/s per ha
  - 65% impervious area = 46 - 58 L/s per ha
  - 80% impervious area = 59 - 72 L/s per ha

In general, the 100 year flow is approximately 60 % - 140 % larger than the 10 year flow.

The results of the modelling are summarized in three categories based on how full the pipe is flowing (by percentage) under the peak 10 year design flow.

- > 80% and ≤ 100% full,
- > 100% and ≤ 115% full, or
- > 115% full.

The percent full is calculated by dividing the design flow by the just full capacity of existing pipe.

NOTE: The computer modelling results provide a starting point for identifying probable problem areas and can serve as the initial basis for future drainage work improvements. The modelling results must be considered only as a planning model suitable for the calculation of design flows. Final selection of the pipe size(s) should be carried out during detailed design and based on the design flow and the hydraulic grade line (based on proposed pipe grade and Manning's 'n') required to ensure adequate freeboard between the highest water level in the storm drain main and the lowest floor elevation of connected properties. The existing condition of each pipe (diameter, material, slope, and physical condition) should be confirmed before undertaking detailed design work.

9.2 Undersized Pipes and Pipes to be Monitored

Computer modelling results indicate potentially undersized pipes and pipes to be monitored within 15 of the City's 24 outfall catchment areas. A brief discussion of the results for existing and at OCP-Build-out conditions is presented below.

9.2.1 Existing Conditions

A total of 10 sections of pipe with a combined length of 594 m are identified for flow monitoring. These pipes flow greater than100% but less than 115% full under peak design flow (10 year).

A total of 44 sections of pipe with a combined length of 2,482 m are shown by computer modelling to be flowing greater than 115% full under peak design flow (10 year), based on known, available data.

The combined length of the 54 sections identified by computer modelling to be flowing greater than 100% full totals 3,076m, which equates to 4.1% of the City's 75 kms of storm pipe.

The location and characteristics of each pipe is presented in **Table 13**.

9.2.2 Existing System at OCP Build-Out

At OCP Build-Out, a total of 65 sections of pipe are calculated to flowing greater than 100% based on the known available data. Their combined length is 4,137 m, or 5.5% of the City's 75 kms of storm pipe. The performance of these pipes should be monitored.

A total of 4 sections with a combined length of 276 m, are shown by computer modeling to be flowing greater than 100% full and less than 115% full under peak design flow (10 year).

A total of 61 sections with a combined length of 3,861 m, are shown by computer modeling to be flowing greater than 115% full under peak design flow (10year).

The location and characteristics of these pipes are also presented in **Table 13,** and the location of each pipe is shown on drawings **1346–1 to 1346–7** located in the pocket at the end of this report. The area covered by each drawing is shown on **Figure 14**.

Manholes where the hydraulic grade line is expected to surcharge above the road surface during the 1 in 100 year event are also noted on these drawings.

9.2.3 Proposed System at OCP Build-Out

For the 65 sections of pipe that are calculated to be flowing greater than 100% during the 10 year event, the proposed diameters needed to convey the design flow at OCP build-out are shown in **Table 13,** and on drawings **1346-1 to 1346-7.** Some of the pipes have very short lengths and some are culverts. If the existing pipe is in reasonable condition, and of a reasonable size, then twinning with a suitability sized diameter conduit to provide additional capacity to accommodate the design flow may be an option, subject to further investigation and preliminary design at each location.

In some areas it may be prudent to install a larger pipe to prevent the HGL from surcharging above the road surface. However, in some cases, and depending on local constraints and site grading issues, introduction of additional, higher capacity inlets at non-surcharged sections of storm sewer may enable greater capture of overland flow. Specific investigation on a case-by-case basis would be required to evaluate the effectiveness of this approach.

The proposed improvements listed in **Table 13** were modelled, and the noted impacts of these improvements and their reduction in the level of calculated surcharging is shown on drawings **1346-8 to 1346-14**. The improvements are graphically evident when comparing the level of surcharging before and after the improvements are implemented. The amount

KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

Table 13 - Pipes to be Upsized or Performance Monitored

Pipes calculated to flow > 80% and ≤ 100% full | Pipes calculated to flow > 100% and < 115% full. | Pipes calculated to be flowing ≥ 115% full. | Pipes calculated to be flowing ≥ 200% full.

Project Label (op dwgs)	Location Description	Pipe Number	Diameter (m)	Capacity (L/s)	10 Year Flow: Existing Condtions (L/s)	10 Year Flow: OCP Build-Out (L/s)	Percent Full: Existing Conditions (%)	Percent Full: OCP Build-Out (%)	Required Pipe: Dia. (mm)	Required Pipe: Capacity (L/s)	Resulting Condition: 10 year flow Percent Full (%)	Resulting Condition: 100 year flow Surcharge Condition	Priority Ranking 5 = Highest 1 = Lowest	General Comments
	Sunray Road Outfall													
	HWY 19a TRUNK SYSTEM													
	Hwy 19a													
B	- Sanderson Rd to 50 m northwest	DP-0806	300	70	171	172	253%	254%	675 Conc	500 L/s	80%	Above Road	5	Future Storage needed for Developments F010 &2430
C	- 70 m northwest of Sanderson Rd	DP-1436	600	330	504	506	152%	152%	600 Conc	725 L/s	80%	Above Road	3	Improve Inlet Capacity at Kasba Way
	Sanderson Rd													
D	- 110 m east of Hwy 19a to Hwy 19a	DP-0807	250	60	47	86	80%	148%	375 PVC	170 L/s	64%	Above Road	3	Future Storage needed for Developments F004
	Aberdeen Dr Park													
E	- Northeast across the park	DP-0810	300	90	102	123	111%	134%	375 PVC	170 L/s	75%	Above Road	3	
	Phillips Rd													
F	- 140 m to 225 m north of Sanderson Rd	DP-0789	250	70	84	84	119%	119%	300 PVC	120 L/s	81%	Above Road	4	Improve Inlet Capacity on Downstream Section
	- 225 m to 335 m north	DP-0788	300	80	133	133	159%	159%	450 PVC	250 L/s	56%	Below Road	4	-
	- 335 m north to 798 Phillips Rd	DP-0787	350	80	159	159	205%	205%	525 PVC	270 L/s	69%	Below Road	4	-
	- 798 Phillips Rd, south sideyard SRW	DP-0786	350	90	159	159	171%	171%	525 PVC	320 L/s	57%	Below Road	4	-
	PYM STREET TRUNK SYSTEM													Around Temple and improve downstream inlet capacity
	Wembley Rd													
A	- 95 m west of Constantine Pl to 80 m west	DP-1364	450	180	149	248	83%	137%	525 PVC	270 L/s	91%	Above Road	1	Future Storage needed for Developments F006 & F007
G	- 65 m south of Doehle Ave, Sideyard SRW	DP-1102	350	280	303	375	107%	133%	450 PVC	550 L/s	70%	Below Road	3	Future Storage needed for Developments F006 & F007
	- 65 m south of Doehle Ave to Doehle Ave	DP-0865	450	400	397	491	99%	122%	Dup 375 PVC	650 L/s	78%	Below Road	3	Future Storage needed for Developments F006 & F007
	- Doehle Ave to Bradbury Ave	DP-0864	450	470	474	541	102%	116%	Dup 375 PVC	810 L/s	72%	Below Road	3	Future Storage needed for Developments F006 & F007
H	- Stanhope intersection	DP-0840	750	610	836	914	138%	151%	Dup 675 PVC	1,070 L/s	90%	Above Road	4	Future Storage needed for Developments F006 & F007
	Doehle Ave Outfall													
														Improve Inlet Capacity on Downstream Sections
	Soriel Road													
I	- Zengel Way to Temple St	DP-1089	525	220	267	269	120%	121%	Dup 375 PVC	330 L/s	83%	Above Road	3	Future Storage needed for Developments 1971
	- ROW 60 m west of Soriel Rd to Soriel	DP-0914	300	80	91	91	115%	115%	375 PVC			Above Road	3	Improve Inlet Capacity on Downstream Sections
	Temple St													
J	- Soriel Rd to 32 m east	DP-0906	750	390	415	460	106%	117%	Dup 525 PVC	570 L/s	81%	Below Road	2	Future Storage needed for Developments 639 & 640
														Improve Inlet Capacity on Downstream Sections
	Panorama Pl													
K	- Willow St to 45 m east of Temple St	DP-0919	375	90	111	111	121%	121%	Dup 300 PVC	150 L/s	75%	Below Road	3	-
	Doehle Ave													
L	- Willow St to Wisteria St	DP-0902	750	930	821	1,066	89%	115%	Dup 525 PVC	1,350 L/s	76%	Below Road	4	Future Storage needed for Developments 2421 & 1947

KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 13 - Pipes to be Upsized or Performance Monitored

Pipes calculated to flow > 80% and ≤ 100% full

Pipes calculated to flow > 100% and < 115% full.

Pipes calculated to be flowing ≥ 115% full.

Pipes calculated to be flowing ≥ 200% full.

Project Label (op dwgs)	Location Description	Pipe Number	Diameter (m)	Capacity (L/s)	10 Year Flow: Existing Condtions (L/s)	10 Year Flow: OCP Build-Out (L/s)	10 Year Flow: Percent Full: Existing Conditions (%)	10 Year Flow: Percent Full: OCP Build-Out (%)	Required Pipe: Dia. (mm)	Required Pipe: Capacity (L/s)	Resulting Condition: 10 year flow: Percent Full (%)	Resulting Condition: 100 year flow: Surcharge Condition	Priority Ranking 5 = Highest 1 = Lowest	General Comments
	Rushton Ave Outfall													
	Rushton Ave													
M	- Temple St to Willow St	DP-1341	600	400	436	471	109%	118%	Dup 450 PVC	620 L/s	86%	Below Road	see text	Future Storage needed for Developments F 014
	Bay Ave Outfall													
	Bay Ave													
N	- Dogwood St intersection	DP-1022	300	140	238	227	174%	167%	450 PVC	400 L/s	57%	Above Road	4	Improve Inlet Capacity on Downstream Sections
	Bayside Inn Resort Outfall (Carey & Romney Creek)													
	HIRST/FINHOLM TRUNK SYSTEM													
	E&N Railway Tracks													
	- Carey Creek Culvert	DP-1510	600	220	57	57	26%	26%						
	- Romney Creek Culvert	DP-1509	600	410	1,107	1,107	267%	267%	*current size required to mitigate peak flows*					
	Hirst Ave													
O	- 30 m west of James St	DP-0624	375	210	475	475	221%	221%	600 PVC	750 L/s	92%	Below Road	5	-
	- At James St	DP-0623	375	240	464	464	195%	195%	600 PVC	830 L/s	85%	Below Road	5	-
	Despard Ave													
P	- East of Hack Berry Pl	DP-0550	300	150	183	183	121%	121%	375 PVC	325 L/s	56%	Below Road	4	-
	Kingsley St													
Q	- At Wheeler Ave	DP-0616	400	70	107	107	157%	157%	525 PVC	170 L/s	65%	Below Road	3	-
	PYM/HWY 19a TRUNK SYSTEM													
	Pym St													
R	- 50 m south of Gerald Pl to Gerald Pl	DP-0452	525	270	313	313	115%	115%	Dup 375 PVC	400 L/s	78%	Above Road	4	Improve Inlet Capacity on Downstream Sections
	Renz Rd													
T	- 70 m south of Daffodil Dr into Renz Park	DP-1377	525	300	198	347	65%	115%	Dup 375 PVC	450 L/s	82%	Below Road	2	Future Storage needed for Developments in Cedar Ridge system
	Clarkson Pl													
U	- 378 Clarkson Pl to 390 Clarkson Pl	DP-0487	300	70	93	93	131%	131%	375 PVC	130 L/s	72%	Below Road	3	-
	Quality Resort (Bayside) Parking Lot													
N2	- Dogwood St east through SRW	DP-0645	250	60	99	89	159%	144%	375 PVC	180 L/s	49%	Below Road	3	-
	Sutherland Pl Outfall													
	Moilliet St													
V	- 100 m north of Despard to 165 m north	DP-1330	300	110	86	171	80%	159%	450 PVC	315 L/s	56%	Below Road	2	Future Storage needed for Developments F 032
	Harnish Ave													
W	- 85 m west of Moilliet St to Moilliet St	DP-0065	150	20	23	23	117%	117%	250 PVC	80 L/s	39%	Below Road	1	-
X	- Sideyard SRW east of Harnish Ave	DP-1068	250	40	54	54	122%	122%	300 PVC	70 L/s	76%	Below Road	1	-

KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 13 - Pipes to be Upsized or Performance Monitored

Pipes calculated to flow > 80% and ≤ 100% full	Pipes calculated to flow > 100% and < 115% full.	Pipes calculated to be flowing ≥ 115% full.	Pipes calculated to be flowing ≥ 200% full.

Project Label (op dwgs)	Location Description	Pipe Number	Diameter (m)	Capacity (L/s)	10 Year Flow: Existing Condtions (L/s)	10 Year Flow: OCP Build-Out (L/s)	10 Year Flow: Percent Full: Existing Conditions (%)	10 Year Flow: Percent Full: OCP Build-Out (%)	Required Pipe: Dia. (mm)	Required Pipe: Capacity (L/s)	Resulting Condition: 10 year flow: Percent Full (%)	Resulting Condition: 100 year flow: Surcharge Condition	Priority Ranking 5 = Highest 1 = Lowest	General Comments
	McMillan St Outfall													
	Birch Ave													
Y	- 200 m west of Lombardy St to 100 m west	DP-0059	250	50	83	75	137%	138%	375 PVC	160 L/s	48%	Below Road	3	-
	- 100 m west of Lombardy St to 70 m west	DP-0061	300	50	83	84	147%	156%	450 PVC	160 L/s	54%	Below Road	3	-
	Alberni Hwy													
Z	- Lee Ave to Jensen Ave	DP-0004	300	110	114	157	107%	147%	375 PVC	195 L/s	81%	Above Road	4	Future Storage needed for Developments
	- Jensen Ave intersection	DP-1162	375	120	200	245	170%	209%	525 PVC	340 L/s	72%	Above Road	4	Improve Inlet Capacity on Downstream Sections
	Harrison Ave													
AA	- McMillan St - 100 m east	DP-0152	150	20	44	44	231%	231%	300 PVC	140 L/s	37%	Above Road	5	Improve Inlet Capacity on Downstream Sections
	- Alberni Hwy - 30 m west	DP-1286,87	300	80	89	89	113%	113%	375 PVC			Above Road	5	Improve Inlet Capacity on Downstream Sections
	Morison Ave													
BB	- McMillan St to 90 m east	DP-0062	200	20	23	23	115%	115%	300 PVC	50 L/s	46%	Below Road	4	-
	- 90m east of McMillan St to Hwy 19A	DP-0063	200	30	46	46	153%	153%	300 PVC	100 L/s	54%	Below Road	4	-
	Hwy 19a													
BB	- Alberni Hwy intersection	DP-1285	525	430	841	883	196%	206%	750 Conc	1,110 L/s	81%	Above Road	5	Future Storage needed for Developments
	Beachside Drive													Improve Inlet Capacity on Downstream Sections
CC	- 70 m east of McMillan St to 12 m east	DP-1456	300	80	99	99	120%	120%	375 PVC	150 L/s	61%	Below Road	1	-
	- 12 m east to McMillan St	DP-1457	300	85	99	99	115%	115%	375 PVC	155 L/s	65%	Below Road	1	-
	Bagshaw St Outfall													
	Corfield St													
CC2	- 80 m south of Stanford to 160 m south	DP-0132	300	120	168	168	135%	135%	375 PVC	225 L/s	80%	Below Road	3	-
	Hwy 19a													
DD	- Corfield St - 90 m east	DP-0195	900	990	1,262	1,358	128%	138%	Dup 900 Conc	1,980 L/s	74%	Below Road	4	Future Storage needed for Developments
	- 90 m east of Corfield to 140 m east	DP-0304	900	990	1,269	1,364	129%	138%	Dup 900 Conc	1,980 L/s	74%	Below Road	4	Future Storage needed for Developments
	Bagshaw St													
EE	- Hwy 19a to 50 m north	DP-0170	1000	1,540	1,590	1,768	103%	115%	1050 Conc	3250 L/s	59%	Below Road	5	Future Storage needed for Developments
	- 50 m north to 100 m north	DP-0168	1000	1,880	1,594	1,922	85%	102%	1050 Conc	3960 L/s	49%	Below Road	5	Future Storage needed for Developments
	Mills St Outfall													
	Skylark Ave													
FF	- Corfield St to Peacock St	DP-0085	250	50	49	66	98%	138%	375 PVC	140 L/s	47%	Below Road	2	Future Storage needed for Developments
	Hwy 19a													
GG	- McVickers St intersection	DP-0175	375	150	189	215	125%	142%	450 PVC	290 L/s	77%	Above Road	3	Future Storage needed for Developments
	Mills St													Improve Inlet Capacity on Downstream Sections
HH	- Hwy 19a to Pioneer Cres	DP-0075	1650	2,960	3,584	4,190	121%	142%	Dup 1350 Conc	4690 L/s	89%	Below Road	5	Future Storage needed for Developments
	Pioneer Cres													
II	- Shelly Rd to 130 m west	DP-1082	375	80	92	158	123%	211%	Dup 450 PVC	220 L/s	74%	Below Road	2	Future Storage needed for Developments F002 and OCP
	- 130 m west to 235 m west	DP-1083	450	120	130	198	105%	165%	Dup 450 PVC	270 L/s	73%	Below Road	2	Future Storage needed for Developments F002 and OCP
		DP-1084	525	280	159	200	79%	109%	Dup 375 PVC	300 L/s	75%	Below Road	2	Future Storage needed for Developments F002 and OCP
		DP-1079	525	220	166	237	75%	116%	Dup 375 PVC	330 L/s	72%	Below Road	2	Future Storage needed for Developments F002 and OCP
	- 180 m east of Mills St to 85 m east	DP-1080	525	190	216	283	114%	150%	Dup 450 PVC	340 L/s	88%	Below Road	2	Future Storage needed for Developments F002 and OCP
		DP-1081	525	280	228	313	82%	108%	Dup 375 PVC	410 L/s	76%	Below Road	2	Future Storage needed for Developments F002 and OCP
	Mills St													
JJ	- Outfall	DP-0001	1050	3,330	3,881	4,563	116%	137%	Dup 1050 Conc	6660 L/s	69%	Below Road	4	

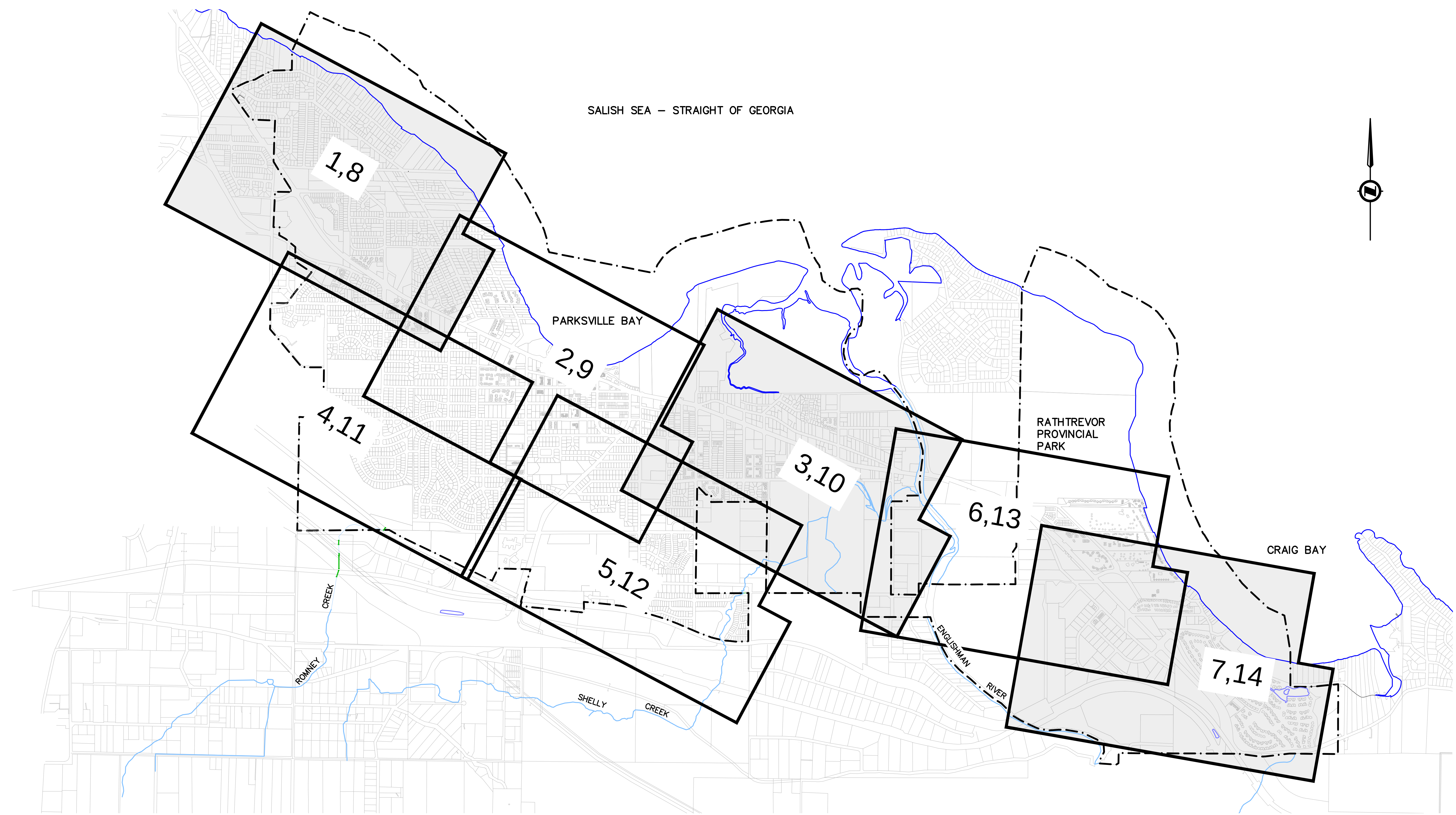
KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 13 - Pipes to be Upsized or Performance Monitored

Pipes calculated to flow > 80% and ≤ 100% full	Pipes calculated to flow > 100% and < 115% full.	Pipes calculated to be flowing ≥ 115% full.	Pipes calculated to be flowing ≥ 200% full.

Project Label (op dwgs)	Location Description	Pipe Number	Diameter (m)	Capacity (L/s)	10 Year Flow: Existing Condtions (L/s)	10 Year Flow: OCP Build-Out (L/s)	Percent Full: Existing Conditions (%)	Percent Full: OCP Build-Out (%)	Required Pipe: Dia. (mm)	Required Pipe: Capacity (L/s)	Resulting Condition: 10 year flow Percent Full (%)	Resulting Condition: 100 year flow Surcharge Condition	Priority Ranking 5 = Highest 1 = Lowest	General Comments
	Shelly Rd Outfall													
KK	- 36 m south of Tulip Ave to Tulip Ave	DP-1054	300	50	96	111	187%	217%	450 PVC	180 L/s	62%	Below Road	3	Future Storage Needed in adjacent developments
	Turner Rd Outfall													
	Hwy 19a													
LL	- Shelly Rd to 105 m east	DP-1003	300	60	98	98	173%	173%	375 PVC	120 L/s	63%	Below Road	2	-
	- 105 m east to 264 m west of Martindale Rd	DP-1002	300	90	145	145	168%	168%	375 PVC	185 L/s	82%	Below Road	2	-
	Shelly Creek													
	Butler Road													
MM	Butler Road Culvert	DP-1506	1070	1,970	3,200	3,200	162%	162%	1.5x2.4 Conc	7,000 L/s	46%		4	Inspect Condition, plan improvements when warranted
	E&N Rail Tracks													
NN	Shelly Creek Culvert	DP-1208a,b	2x760	1,240	3,104	3,104	250%	250%	2x1200 Conc	4,000 L/s	78%		4	Culvert Belongs to ICF, ensure consultation occurs if replacement is pl
	Martindale Road													
OO	Martindale Road Culvert	1503a,b	2x1200	3,140	3,993	3,993	127%	127%	1.5x2.4 Conc	7,000 L/s	59%		4	Inspect Condition, plan improvements when warranted

		Existing	OCP
	Total Length (m):	***186***	***379***
	Total Length (m):	***2,296***	***3,482***
	Total Length (m):	***594***	***276***
	Total Length (m):	***556***	
	Combined Total (m):	***3,633***	***4,137***



File: H:\6012 Parksville City\1346 Parksville Sanitary & Storm Drainage Master Plan\03 Drawings\Storm\1346StormMap Large.dwg Plot Time: Mar 11, 2016 – 8:50am User: rcave

KOERS & ASSOCIATES ENGINEERING LTD.
Consulting Engineers

CLIENT	CITY OF PARKSVILLE	TITLE	DRAWING SHEET LOCATION KEY PLAN
PROJECT	STORM DRAINAGE MASTER PLAN	APPROVED	
		DATE	AUG 2015
		PROJECT No.	1346
		SCALE	1:25,000
		DWG No.	FIGURE 14

of manholes that are predicted to surcharge during the 1 in 100 year event is also significantly reduced.

The need for a given improvement will be partially driven by upstream development, and whether the future impacts could be mitigated by requiring future developments to provide onsite detention to limit post-development flows to pre-development levels.

Drawings **1346-08 to 1346-14** also show the potential storm mains in areas that currently do not have an enclosed storm sewer system. The new mains were modelled as part of the OCP build-out and should be considered if road improvements or other utility upgrades are planned for these areas.

9.3 Groundwater Infiltration

There are several areas of higher permeability soils. These are noted in **Table 11** as soil group A and to a lesser degree by soil group B. The most notable areas where development has occurred with higher permeable soils are:

- Craig Bay Area,
- In and around Pioneer Crescent, and
- In and around the Herring Gull Industrial Park.

A brief discussion of each is presented below.

Craig Bay Area
The existing municipal storm drainage system is very limited, reflective of the soils' conditions. Most developments manage stormwater on-site. For existing conditions and at OCP Build-out, it was assumed that there would be no further expansion of the municipal collection system, and all future development in the area will be required to manage stormwater runoff on-site.

Pioneer Crescent Area
There is an enclosed municipal storm drainage system in parts of the area. As new development or redevelopment is proposed, each should be assessed on a case-by-case basis with the requirement to infiltrate stormwater as much as possible. The ability to eliminate the upgrading of existing mains and the construction of new mains with the use of on-site stormwater management including infiltration to ground should be explored.

Herring Gull Industrial Park Area
The majority of this site has been developed without an enclosed drainage system and relies heavily on the road side ditches which have no defined points of discharge. This ditch and culvert system currently seems to provide effective stormwater management through infiltration to ground. Future development should be encouraged to maintain this infiltration approach.

The application of on-site stormwater management techniques is consistent with the City's OCP guidelines for stormwater management and groundwater protection. The OCP also includes guidelines for diverting stormwater away from the Englishman River Estuary, as well as avoiding discharges to watercourses.

9.4 Stormwater Runoff Treatment

The point of discharge of the City's 24 stormwater outfalls listed in **Table 2** and shown in **Figure 2** can be divided into six categories:

- Fourteen discharge onto the foreshore either into Craig Bay (3), into Parksville Bay (7), or west of Parksville Bay (4).

- Three discharge into Shelly Creek, which discharges into the Englishman River
- One discharges into the Englishman River.
- Three discharge into a watercourse that flows into the Englishman River Estuary,
- Two discharge to ground within the Englishman River estuary, and
- One discharges onto land within the RDN (Renz Outfall).

Englishman River Estuary & Watercourses

The City's OCP includes guidelines for diverting stormwater away from the Englishman River Estuary, as well as avoiding discharges to watercourses. The re-routing of the existing outlets would be very difficult, and is considered impractical. Instead, the use of site specific Stormwater Best Management Practises (BMPs) to mitigate negative effects of stormwater discharge could be explored. This would require identification of the issue(s) requiring mitigation and the BMPs most suitable to be implemented.

On-Site Stormwater Management

The OCP includes guidelines for incorporating on-site stormwater management techniques and providing stormwater treatment for groundwater protection. The management of stormwater as close to the source as possible is preferred, from the perspective of the general taxpayer, as the construction, operation and maintenance of the system is allocated to the property owner/users and not the City of Parksville. However, in some instances a system that is constructed on municipal property and is owned and maintained by the City may be a more appropriate approach.

The treatment of on-site runoff prior to discharge to the municipal storm drain system is becoming more common. For example, the City of Courtenay's Storm Sewer Bylaw No 2182 requires a development with more than 10 vehicle parking stalls (open air) to install an oil and grit separator.

Foreshore, Wildlife & Public Use

The Parksville-Qualicum Beach Wildlife Management Area (P-QBWMA) was created in 1993 by the provincial government. Encompassing 1,024 ha of coastal foreshore, estuary and river habitat between Craig Bay and the Little Qualicum River, the area contains a diversity of ecosystems including the Englishman River Estuary and the Parksville Bay. These areas as important habitats for numerous animals, the most notably of which is the large flock of Pacific Brant geese that stop to rest and feed each spring as they migrate north to their breeding grounds in Alaska and Russia.

The Parksville Bay is an active tourism location that reaches its peak use in the months of July and August. The boardwalk and expansive sandy beach in Parksville Bay, along with walking trails in the estuary, and Rathtrevor Beach Provincial Park invite and draw the public to explore the area.

The importance of the ecosystem along with the public's contact with the foreshore brings a heightened awareness of water quality in the area. This is expected to continue to increase over time as the population continues to grow and development in the City increases.

10 CAPITAL PLANNING

As part of the development of the Master Plan, a risk assessment of the failure of various components of the storm sewer system was carried out. This can be a very useful tool in prioritizing the proposed capital works as the risk assessment compares two areas of each project:

i) Likelihood of Failure, and
ii) Consequence of Failure.

These two assessment tools were applied in assessing the **condition** and the **capacity** of each project. The two assessments were then used to develop a capital planning score to rank the priority of the projects.

10.1 Condition Risk Analysis

The condition criticality analysis assesses the "Likelihood of Failure" based on the infrastructure's age against cost to restore service. The rating used for each is presented below followed by the Condition Risk scoring matrix:

Likelihood of Failure

Likelihood of Failure Rating	Age of Pipe
5	> 60 years
4	> 40 years to ≤ 60 years
3	> 25 years to ≤ 40 years
2	> 15 years to ≤ 25 years
1	< 15 years

Consequence of Failure
(Cost to Restore)

Consequence of Failure Rating	Severity	Cost to Restore
5	Severe	>$500,000
4	Major	$250,000 to $500,000
3	Moderate	$100,000 to $250,000
2	Minor	$50,000 to $100,000
1	Minimal	<$50,000

Once the 'Likelihood of Failure' and the 'Consequence of Failure' scores are determined, the chart below is used to determine the Condition risk score. The higher the score rating, the higher risk is estimated to be.

Condition Risk Score

Consequence \ Likelihood of Failure	1	2	3	4	5
5	3	3	4	5	5
4	2	3	4	4	5
3	2	2	3	4	4
2	1	2	2	3	3
1	1	1	2	2	3

10.2 Capacity Risk Analysis

The capacity criticality analysis assesses the "Likelihood of Failure" based on exceeding the capacity of the infrastructure. The "Consequence of Failure" is ranked according to the number of people affected which is approximated based on the pipe diameter. The rating used for each is presented below followed by the Capacity Risk scoring matrix.

.

Likelihood of Failure

Likelihood of Failure Rating	Hydraulic Capacity (depth/Diameter)
5	> 200% full
4	> 115% to ≤ 200% full
3	> 100% to ≤ 115% full
2	> 80% to ≤ 100% full
1	≤ 80% full

Consequence of Failure

Consequence of Failure Rating	Severity	Pipe Diameter
5	Severe	> 750 mm
4	Major	525 mm to 750 mm
3	Moderate	375 mm to 450 mm
2	Minor	200 mm to 300 mm
1	Minimal	< 200 mm

Once the 'Likelihood of Failure' and the 'Consequence of Failure' scores are determined, the chart below is used to determine the Capacity risk score. The higher the score rating, the higher risk is estimated to be.

Capacity Risk Score

Consequence \ Likelihood of Failure	1	2	3	4	5
5	3	3	4	5	5
4	2	3	4	4	5
3	2	2	3	4	4
2	1	2	2	3	3
1	1	1	2	2	3

10.3 Capital Planning Priority

A Capital planning score is determined by entering the Condition and Capacity scores in the chart below. The higher the score, the higher the priority placed on that project.

Capital Planning Priority Score

Capacity					
5	3	3	4	5	5
4	2	3	4	4	5
3	2	2	3	4	4
2	1	2	2	3	3
1	1	1	2	2	3
	1	**2**	**3**	**4**	**5**

Condition

Where:

1 = Low Priority (low Risk Score for Capacity & Condition)
5 = High Priority (high Risk Score for Capacity & Condition)

The calculated priority score for the projects identified in this study is shown in **Table 13**.

10.4 Infrastructure Condition Verification

The computer model was developed using the City's GIS database and augmented with record drawing information if no data was in the GIS database, such as for recent subdivisions. The conveyance capacity of the mains (% full) was assessed based on this information. No inspections were carried out to confirm the pipe diameters, material slope or condition of the mains.

A video inspection of all mains in the sewer system would document the current condition of each pipe, service connection and manholes. This information would serve several purposes including:

- Prioritizing identified maintenance works, and
- Coordinating the findings with road construction works to ensure the underground infrastructure is dealt with before the road is repaved.

This video inspection of 3 km to 5 km of pipe per year would result in the video inspection of the entire 75 km of pipe in 15 to 25 years.

11 RECOMMENDED IMPROVEMENTS & COST ESTIMATES

Table 14 lists the recommended upgrading works, as discussed in section **9 Modeling Results**. They are grouped by catchment area, with the project locations shown on **drawings 1346-8** to **1346-14**. The proposed pipe diameters are based on the conveyance of the 10 year flow at OCP Build-Out at no more than 80% full and maintain the 1 in 100 year HGL below the road surface where feasible. The projects in the cost estimate reference the Capital Planning Priority Score discussed in section **10 Capital Planning** and noted in **Table 13.** Cost estimates for ditch infill work, or other CSP replacement projects associated with road improvements have not been included.

The cost estimates are based on Class 'D' (feasibility study) estimates, and made without preliminary design input. The estimates include a 30% general contingency allowance and a 30% allowance for legal, construction, financial, administration and engineering costs. The estimates are exclusive of GST, and are assumed to be taken as stand-alone projects, with allowances made for various project related overhead items. Per-unit costs could be less if storm projects are grouped together, or combined with other related utility and road improvement projects to obtain a better economy of scale.

Cost estimates are derived from our in-house construction cost data base for infrastructure construction projects in the mid-Vancouver Island. All costs are as of July 2015 when the ENR Construction Cost Index was 10037.

The computer modelling results provide a starting point for identifying probable problem areas and can serve as the initial basis for future drainage work improvements. The modelling results must be considered only as a planning model suitable for calculation of design flows. Final selection of the pipe size(s) should be carried out during detailed design and based on the design flow and the hydraulic grade line **(based on the available pipe grade** and the Mannings 'n' factor) required to ensure adequate freeboard between the hydraulic grade line and the lowest floor elevation of connected properties. In some cases the assumed pipe grade may not be possible due to the presence of other conflicting utilities, and a larger pipe size could be required. The existing condition of each pipe (diameter, material, slope, and physical condition) should be confirmed before undertaking detailed design work.

Additional investigative items are noted below.

1. <u>Infrastructure Condition Verification -</u> Undertaking a systematic video camera inspection of the storm sewer system would enable the City to confirm record information and verify the condition of the existing pipe network.

2. <u>Ongoing Monitoring and Model Calibration –</u> Utilizing the City's flow monitor to record data at critical spots and cross check the results against recorded rainfall data and the expected model results.

3. <u>Model updating –</u> Continuously updating the storm drainage model on a regular basis to ensure that new works constructed by developers and by the City are incorporated in the model.

4. <u>Study updates</u> – Updating the Master Plan every 5 to 7 years, as new projects are implemented and flow data is collected.

5. <u>Increase Inlet Capacity –</u>Investigating areas flagged as surcharging during the 1 in 100 year event and confirm if the installation of higher capacity inlet structures could mitigate potential overland flow risks.

KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 14 Cost Estimates

Project Label (on dwgs)	Location Description	Existing Pipe Number	Existing Diameter (m)	Existing Length (m)	Existing Age (years)	Existing Material	Required Pipe Dia. (mm)	Construction Cost Estimate ($)	Comments: Potentially DCC Elgible	Comments: Require Future Onsite Storage	Comments: Combine with Road Upgrades	Comments: Priority Ranking 5 = Highest 1 = Lowest
	Sunray Road Outfall											
	HWY 19a TRUNK SYSTEM											
	Hwy 19a											
B	- Sanderson Rd to 50 m northwest	DP-0806	300	49	23	PVC	675 Conc	$101,000	-	-	Yes	5
C	- 70 m northwest of Sanderson Rd	DP-1436	600	24	23+	***CSP***	600 Conc	$48,000	-	-	Yes	3
	Sanderson Rd											
D	- 110 m east of Hwy 19a to Hwy 19a	DP-0807	250	112	23	PVC	375 PVC	$190,000	Yes	Yes	Yes	3
	Aberdeen Dr Park											
E	- Northeast across the park	DP-0810	300	88	24	PVC	375 PVC	$149,000	Yes	Yes	Yes	3
	Phillips Rd											
F	- 140 m to 225 m north of Sanderson Rd	DP-0789	250	84	36	PVC	300 PVC	$137,000	-	-	Yes	4
	- 225 m to 335 m north	DP-0788	300	108	36	PVC	450 PVC	$193,000	-	-	Yes	4
	- 335 m north to 798 Phillips Rd	DP-0787	350	97	36	***AC***	525 PVC	$183,000	-	-	Yes	4
	- 798 Phillips Rd, south sideyard SRW	DP-0786	350	49	36	***AC***	525 PVC	$93,000	-	-	Yes	4
	PYM STREET TRUNK SYSTEM											
	Wembley Rd											
A	- 95 m west of Constantine Pl to 80 m west	DP-1364	450	15	9	Conc	525 PVC	$28,000	Yes	Yes	Yes	1
G	- 65 m south of Doehle Ave, Sideyard SRW	DP-1102	350	8	19	PVC	450 PVC	$14,000	Yes	Yes	Yes	3
	- 65 m south of Doehle Ave to Doehle Ave	DP-0865	450	63	29	PVC	Dup 375 PVC	$106,000	Yes	Yes	Yes	3
	- Doehle Ave to Bradbury Ave	DP-0864	450	70	34	***AC***	Dup 375 PVC	$119,000	Yes	Yes	Yes	3
H	- Stanhope intersection	DP-0840	750	17	13	Conc	Dup 675 PVC	$35,000	Yes	Yes	Yes	4
	Doehle Ave Outfall											
	Soriel Road											
I	- Zengel Way to Temple St	DP-1089	525	97	17	Conc	Dup 375 PVC	$164,000	-	-	Yes	3
	- 60 m west of Soriel Rd	DP-0914	300	57		PVC	375 PVC	$96,000	-	-	Yes	3
	Temple St											
J	- Soriel Rd to 32 m east	DP-0906	750	32	17	Conc	Dup 525 PVC	$61,000	Yes	Yes	Yes	2
	Panorama Pl											
K	- Willow St to 45 m east of Temple St	DP-0919	375	110	34	Conc	Dup 300 PVC	$175,000	-	-	Yes	3
	Doehle Ave											
L	- Willow St to Wisteria St	DP-0902	750	127	33	Conc-R	Dup 525 PVC	$241,000	-	-	Yes	4

Table 14 Cost Estimates

Project Label (on dwgs)	Location Description	Existing					Required Pipe Dia. (mm)	Construction Cost Estimate ($)	Comments			
		Pipe Number	Diameter (m)	Length (m)	Age (years)	Material			Potentially DCC Elgible	Require Future Onsite Storage	Combine with Road Upgrades	Priority Ranking 5 = Highest 1 = Lowest
	Rushton Ave Outfall											
	Rushton Ave											
M	- Temple St to Willow St	DP-1341	600	73	6	HDPE	Dup 450 PVC	$132,000	-	-		
	Bay Ave Outfall											
	Bay Ave											
N	- Dogwood St intersection	DP-1022	300	27	25	PVC	450 PVC	$47,000	-	-		4
	Bayside Inn Resort Outfall (Carey & Romney Creek)											
	HIRST/FINHOLM TRUNK SYSTEM											
	E&N Railway Tracks											
	- Carey Creek Culvert	DP-1510	600	16		***CSP***						
	- Romney Creek Culvert	DP-1509	600	20	?	***CSP***	*current size mitigating peak flows - see text in report*					
	Hirst Ave											
O	- 30 m west of James St	DP-0624	375	6	11	PVC	600 PVC	$11,000	-	-	Yes	5
	- At James St	DP-0623	375	27	11	PVC	600 PVC	$53,000	-	-	Yes	5
	Despard Ave											
P	- East of Hack Berry Pl	DP-0550	300	54	33	Conc	375 PVC	$91,000	-	-	Yes	4
	Kingsley St											
Q	- At Wheeler Ave	DP-0616	400	28	39	***AC***	525 PVC	$53,000	-	-	Yes	3
	PYM/HWY 19a TRUNK SYSTEM											
	Pym St											
R	- 50 m south of Gerald Pl to Gerald Pl	DP-0452	525	50	17	Conc	Dup 375 PVC	$85,000	-	-	Yes	4
	Renz Rd											
T	- 70 m south of Daffodil Dr into Renz Park	DP-1377	525	26	2	Conc	Dup 375 PVC	$44,000	Yes	Yes	Yes	2
	Clarkson Pl											
U	- 378 Clarkson Pl to 390 Clarkson Pl	DP-0487	300	31	17	PVC	375 PVC	$52,000	-	-	Yes	3
	(Bayside Inn) Parking Lot											
N2	- Dogwood St east through SRW	DP-0645	250	59	30	PVC	375 PVC	$100,000	-	-	Yes	3
	Blenkin Avenue Drainage Course Improvements		Open Ditch	250	-	-	900 conc	*$500,000*	Yes	Yes	N/A	TBD
											(see text in report)	

KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 14 Cost Estimates

Project Label (on dwgs)	Location Description	Existing Pipe Number	Existing Diameter (m)	Existing Length (m)	Existing Age (years)	Existing Material	Required Pipe Dia. (mm)	Construction Cost Estimate ($)	Comments: Potentially DCC Elgible	Comments: Require Future Onsite Storage	Comments: Combine with Road Upgrades	Comments: Priority Ranking 5 = Highest 1 = Lowest
	Sutherland Pl Outfall											
	Moilliet St											
V	- 100 m north of Despard to 165 m north	DP-1330	300	65	6	PVC	450 PVC	$116,000	Yes	Yes	Yes	2
	Harnish Ave											
W	- 85 m west of Moilliet St to Moilliet St	DP-0065	150	83	25	PVC	250 PVC	$125,000	-	-	Yes	1
X	- Sideyard SRW east of Harnish Ave	DP-1068	250	28	9	PVC	300 PVC	$44,000	-	-	Yes	1
	McMillan St Outfall											
	Birch Ave											
Y	- 200 m west of Lombardy St to 100 m west	DP-0059	250	100	34	PVC	375 PVC	$170,000	Yes	Yes	Yes	3
	- 100 m west of Lombardy St to 70 m west	DP-0061	300	100	34	PVC	450 PVC	$180,000	Yes	Yes	Yes	3
	Alberni Hwy											
Z	- Lee Ave to Jensen Ave	DP-0004	300	40	15	PVC	375 PVC	$68,000	Yes	Yes	Yes	4
	- Jensen Ave intersection	DP-1162	375	13	10	Conc	525 PVC	$25,000	Yes	Yes	Yes	4
	Harrison Ave											
AA	- McMillan St - 100 m east	DP-0152	150	100	44	***AC***	300 PVC	$159,000	-	-	Yes	5
	- Alberni Hwy - 30 m west	DP-1286,87	300	31		PVC	375 PVC	$52,000	-	-	Yes	5
	Morison Ave											
BB	- McMillan St to 90 m east	DP-0062	200	91	62	***Clay Tile***	300 PVC	$155,000	-	-	Yes	4
	- 90m east of McMillan St to Hwy 19A	DP-0063	200	107	62	***Clay Tile***	300 PVC	$181,000	-	-	Yes	4
	Hwy 19a											
BB	- Alberni Hwy intersection	DP-1285	525	14	26	Conc	750 Conc	$32,000	Yes	Yes	Yes	5
	Beachside Drive											
CC	- 70 m east of McMillan St to 12 m east	DP-1456	300	60	6	PVC	375 PVC	$102,000	-	-	Yes	1
	- 12 m east to McMillan St	DP-1457	300	12	6	PVC	375 PVC	$21,000	-	-	Yes	1
	Bagshaw St Outfall											
	Corfield St											
CC2	- 80 m south of Stanford to 160 m south	DP-0132	300	79	22	PVC	375 PVC	$134,000	-	-	Yes	3
	Hwy 19a											
DD	- Corfield St - 90 m east	DP-0195	900	91	26	Conc	Dup 900 Conc	$245,000	-	-	Yes	4
	- 90 m east of Corfield to 140 m east	DP-0304	900	91	26	Conc	Dup 900 Conc	$245,000	-	-	Yes	4
	Bagshaw St											
EE	- Hwy 19a to 50 m north	DP-0170	1000	52	37	***CSP***	1050 Conc	$167,000	Yes	Yes	Yes	5
	- 50 m north to 100 m north	DP-0168	1000	58	37	***CSP***	1050 Conc	$185,000	Yes	Yes	Yes	5

KOERS & ASSOCIATES ENGINEERING LTD. Consulting Engineers

Table 14 Cost Estimates

Project Label (on dwgs)	Location Description	Existing Pipe Number	Existing Diameter (m)	Existing Length (m)	Existing Age (years)	Existing Material	Required Pipe Dia. (mm)	Construction Cost Estimate ($)	Comments: Potentially DCC Elgible	Comments: Require Future Onsite Storage	Comments: Combine with Road Upgrades	Priority Ranking 5 = Highest 1 = Lowest
	Mills St Outfall											
	Skylark Ave											
FF	- Corfield St to Peacock St	DP-0085	250	117	25	PVC	375 PVC	$198,000	Yes	Yes	Yes	2
	Hwy 19a											
GG	- McVickers St intersection	DP-0175	375	95	22	Conc -R	450 PVC	$171,000	Yes	Yes	Yes	3
	Mills St											
HH	- Hwy 19a to Pioneer Cres	DP-0075	1650	152	26	Conc	Dup 1350 Conc	$546,000	Yes	Yes	Yes	5
	Pioneer Cres											
II	- Shelly Rd to 130 m west	DP-1082	375	130	9	Conc	Dup 450 PVC	$234,000	Yes	Yes	Yes	2
	- 130 m west to 235 m west	DP-1083	450	105	9	Conc	Dup 450 PVC	$189,000	Yes	Yes	Yes	2
		DP-1084	525	104	9	Conc	Dup 375 PVC	$176,000	Yes	Yes	Yes	2
		DP-1079	525	79	9	Conc	Dup 375 PVC	$134,000	Yes	Yes	Yes	2
	- 180 m east of Mills St to 85 m east	DP-1080	525	93	9	Conc	Dup 450 PVC	$167,000	Yes	Yes	Yes	2
		DP-1081	525	83	9	Conc	Dup 375 PVC	$141,000	Yes	Yes	Yes	2
	Mills St											
JJ	- Outfall	DP-0001	1050	60	26	Conc	Dup 1050 Conc	$193,000	Yes	Yes	Yes	4
	Shelly Rd Outfall											
KK	- 36 m south of Tulip Ave to Tulip Ave	DP-1054	300	36	8	Conc-R	450 PVC	$64,000	Yes	Yes	Yes	3
	Turner Rd Outfall											
	Hwy 19a											
LL	- Shelly Rd to 105 m east	DP-1003	300	104	26	Conc	375 PVC	$177,000	-	-	Yes	2
	- 105 m east to 264 m west of Martindale Rd	DP-1002	300	108	26	Conc	375 PVC	$183,000	-	-	Yes	2
	Shelly Creek											
	Butler Road											
MM	Butler Road Culvert	DP-1506	1070	18	33?	*CSP*	1.5x2.4 Conc	$99,000	-	-	Yes	4
	E&N Rail Tracks											
NN	Shelly Creek Culvert	DP-1208a,b	2x760	12		*CSP*	2x1200 Conc		see text			4
	Martindale Road											
OO	Martindale Road Culvert	1503a,b	2x1200	18	33?	*CSP*	1.5x2.4 Conc	$99,000	-	-	Yes	4

12 CONCLUSIONS

The following conclusions are made from the findings of this study:

Collection System

1. The City's storm drainage system is in reasonable shape given the works installed over the last 15 years.

2. Many projects recommended in the 1998 Storm Drainage Study have been undertaken through a combination of developer required offsite projects, DCC projects, or various City Capital improvement projects. Some projects recommended in that study have not yet been completed, but are still required.

3. The Storm drainage collection system consists of more than 75 kilometres of pipe main ranging from as small as 150 mm diameter to as large as the 3050 mm diameter. Approximately 64% of the system is less than 30 years old, 23% are between 30 years and 60 old, and the age of approximately 13% of the system is not known.

4. There are 24 stormwater outfalls that discharge either directly to:

    - the foreshore (14),
    - a creek, stream or estuary (7), or
    - directly to ground (3)

5. There are three drainage courses within the City that receive flow from lands outside of the City:

    - Englishman River,
    - Romney Creek / Carey Creek, and
    - Shelly Creek.

    The lands within each of these catchments that are outside of the City are within in the RDN, which has established land-use and zoning bylaws, but subdivision and drainage works are controlled by the Ministry of Transportation and Infrastructure.

6. There are stormwater detention structures and stormwater detention/treatment systems within the City as noted in **Table 3**. Three are reported to be owned by the City, two by the province and the remaining three by private development.

7. Generally, soil conditions through most of the City do not facilitate ground water infiltration.

Flow Monitoring

8. The use of three temporary flow monitoring stations provided data that was used to successfully calibrate the computer model.

9. The catchments selected for monitoring varied in size from 21 ha to 456 ha. They varied from: mostly developed with a range of land-uses; developed with a mixture of uses including a large area of forested land yet to be developed; and mostly rural developed lands outside of the City which drain into the City's enclosed storm drainage system. The catchment area for each is shown in **Figure 3**.

10 While it was relatively dry during the 3½ month monitoring period (December 2013 to mid-March 2014), two more noticeable rainstorm events were recorded; January 10 - 11, 2014 and February 15 - 16, 2014. Both plotted below a 5 year return period on the City's existing IDF curves.

Climate Change

11 Climate change is influencing rainfall patterns and is expected to increase loading on municipal storm drainage systems in the future as winters on the east coast of Vancouver Island become wetter and summers become dryer.

12 Climate change model forecasts vary significantly based on the input data and assumptions made in developing the models.

13 There are known weather cycles that can span from a few years (El Nino/El Nina) to several decades (the Pacific Decadal Oscillation). It is thought that we may have recently shifted from a warm to a cool PDO phase which could result in more extreme precipitation for the Parksville region. Recent analyses completed by the Pacific Climate Impacts Consortium forecast that by the 2050s, extreme precipitation for the area will increase by:

- 5% to 15% for daily maximum, and
- 5% to 50% for hourly.

The governing Time of Concentration (Tc) for the City's storm drainage network is generally 1 hour, excluding Romney and Shelly creeks, which has an estimated Tc of approximately 6 hours.

Sea Level Rise

14 The Provincial Government has recommended that a sea level rise of 1 m by Year 2100 and 2 m by Year 2200 be used for coastal flood planning.

Design Criteria

15 The City's Engineering Specifications divides the storm drain system into two components:

- The **minor system** which consists of pipes and ditches which are sized to convey flows with a **10 year return period**, and
- The **major system** which consists of surface flow paths, roadways, and watercourses which convey flows with a **100 year return period**.

16 Given the uncertainty in the change of future extreme rainfall intensities, drainage system resilience could be improved through the use of a percentage full limit for pipe design. If the design flow results in the pipe flowing more than the design limit, such as the 80% full used in this study, the pipe diameter would be increased to the next available size.

17 The City's design standards currently state that mains are to be a minimum of 300 mm diameter, except in residential areas where 250 mm diameter will be accepted in the final section provided there is not more than one catchbasin connected to it. The City's GIS database indicates that there are 3.6 kms of 150 mm to 200 mm diameter pipe in the City's collection system

18 Given the various options for catchbasin inlet styles, methods to accommodate overland flow and mitigate system blockages can be improved if the City adopts the use of side-inlet catch basin grates.

19 Future development areas that are defined in the OCP will increase impervious surfaces within the City and increase future post-development storm drainage flows, which will have an impact on existing downstream drainage facilities. The requirement to store storm drainage flows onsite to match pre-development flows will mitigate impacts on the downstream drainage infrastructure and delay or reduce the potential need to upgrade existing plant in some areas.

IDF Curve Update

20 The City's current IDF curves were created in 1998 and have not been updated since that time. Updated IDF curves were developed using the same process by which the existing curves were created; by factoring rainfall data from other nearby Environment Canada weather stations with longer periods of rainfall data recorded in 5 minute increments. The updated curves are based on 25 years of data (1980 to 2005) from the Nanaimo City Yard weather station.

21 The updated curves have a slightly steeper slope, pivoting around the 40 to 50 minute duration. As a result, rainfall intensities less than one hour have increased by up to 34%, while intensities for durations of longer than 1 hour have decreased by up to 31%.

22 The majority of the City's storm drainage basins are calculated as having a Time of Concentration of approximately 1 hour. The updated IDF curves at this duration are nearly identical as the existing ones; being lower by 1% to 5%.

23 For the Romney Creek catchment, for which the majority of the storm drainage system is enclosed within the City, it has an estimated 6 hour Time of Concentration. The updated IDF curves at this duration are 17% to 20% lower than the current ones.

Further Investigation

24 The pipe network used to model the existing system has been based on the grades in the City's GIS database or as shown on the provided record drawings, and has not been confirmed in the field. Improvement options that are noted in the study are based on these published grades, but actual final pipe sizes are subject to detailed design and should be based on the design flows and grades available in the field. Once final locations and grades have been determined, a larger pipe size may be required to convey the design flow.

25 Video inspection of all mains in the storm drainage system would confirm the current condition of each pipe, service connection and manhole. The information could be checked against the computer model and the City's database, and both updated as needed to reflect actual conditions. The computer model should be re-run as new information becomes available to check the findings of this report and assess if other works are required.

This video library would serve several purposes including:

- Verifying the pipe information the computer model uses
- Assessing condition of older mains or ones with incomplete information
- Prioritizing identified maintenance works
- Coordinating the findings with road construction works to ensure the underground infrastructure is dealt with before the road is repaved
- Capital Planning

This video inspection of 3 to 5 kms of pipe per year would result in the inspection of the entire 75 kms of pipe over a 15 to 25 year period.

26 The model should be kept current as future storm drainage projects are designed and constructed. Subsequent upgrades proposed in this study could be refined as the impact on future facilities that have not yet been upgraded would be identified.

27 The results listed in the model assume that inlets and service connections are intact and fully functioning. Inspecting and clearing pipes on a regular basis would improve system performance.

28 Improved inlet capture where inadequate catch basin spacing exists at low points within the road or boulevard or at the end of downhill dead-end roads would improve system performance.

29 The older areas of the City that are without an enclosed storm drainage system or curb and gutter, are not likely graded to properly accommodate the 100 year overland flow. In areas where it is not practical to maintain the 100 year levels below road surface, more detailed investigation to review potential impacts on existing basement elevations is needed.

30 Some older areas of the City lack enclosed storm sewers, which can be included with future road re-construction or other utility improvement projects.

31 The newer areas of the City that have curb and gutter with enclosed storm drains may need retro-fits with improved curb inlet catchbasins to better capture the 100 year surface flow.

Modelling Results at OCP Build-Out

32 Using the updated IDF curve and assuming full build-out under OCP conditions, the modelling results suggest that most of the existing storm drainage system can take the 10 year flow within the pipe. However, 65 sections of pipe totaling 4,137m are expected to surcharge under the 10 year flow. They are listed in **Table 13**, and shown on **Drawings 1346-1 through 1346-7.**

33 The potential improvements needed to convey the 10 year flow within the pipe are shown in **Table 13.** These works generally consist of various replacements, or twinning where a larger (450 mm dia. or larger) main already exists and is believed to be in reasonable condition (less than 40 years old). The conditions following the installation of these upgrades are shown in Table 13 and on drawings **1346-8 through 1346-14.**

34 Using the updated IDF curve and assuming full build-out under OCP conditions, most of the existing system can accommodate the 100 year flow without surcharging above the surface. However, areas are expected to surcharge during the 100 year event, and they are listed in **Table 13**, and shown on drawings **1346-1 through 1346-7.**

35 The areas where the modelling results suggest that the 100 year flows are surcharging above the surface after implementing the improvements listed in **Table 13** are shown on drawings **1346-8 through 1346-14.**

36 Strategies to mitigate surcharging could include adjusting the proposed 10 year improvement where feasible to accommodate additional flow, or increasing the ability of downstream capture in areas of the storm drain system that have capacity, typically by installing additional or improved inlet systems if site grading allows.

Governmental Agency Coordination

37 There are lands outside the City's municipal boundaries that discharge runoff into the City. The decisions regarding land use and development patterns in these areas are controlled by the RDN and MoTi and can have a strong impact on the potential increase in storm drainage flows that enter the City.

38 A private foundation, the ICF, owns and operates the existing railroad that borders the southern boundary of the City. There are numerous drainage appurtenances that direct flow underneath the railroad, and which at present, are undersized and serve to mitigate the flows entering the City system. Upgrading these culverts in the future, would increase the flows entering the City, and could have potential serious consequences on the downstream drainage system.

39 Areas where City development directly abuts the railroad can be susceptible to seepage and flooding caused by the berming effect of the elevated railroad and poor lateral ditching or drainage along the tracks. Improved ditching on the upstream tracks, and formalized culvert crossing points could mitigate these issues.

Significant Storm Drainage Issues (By Basin)

40 The Sunray Drainage Basin (Hwy 19A trunk). With the additional development of Wembly Mall into this basin, sections of the downstream piping system will be undersized for the 10 year event. An increase in capacity, onsite storage within future development, or a combination of both will be required The existing piping on Phillips Road, as noted in the 1998 study, is undersized for the 10 year event and requires replacement.

41 The Sunray Drainage Basin (Pym Street trunk). Future development will increase loading on portions of the Pym Street system and will require onsite storage to mitigate the impacts. Additional measures to capture overland flow during the 1 in 100 year event should be reviewed. Upgrading portions of the Pym Street system or twining may be required if peak flows from future developments are not mitigated.

42 The Rushton / Shoreline Basin. The storm drain runs through the Shoreline townhouse complex, is in poor condition, and there is no defined overland flow path through this area. Previous projects have identified the need to construct a new storm outlet at the north end of Rushton Avenue, and some measures have already been implemented. Additional diversions and re-routing to Rushton on Temple, Willow, Wisteria and Dogwood would enable facilitate this solution.

43 The Carey / Romney Creek Basin. Significant trunk works have been installed on McKinnon and Wheeler, which has improved the capacity within the catchment area. However, this system is heavily influenced by flows originating beyond the City's boundaries, specifically the existing 600 mm dia culvert under the E&N railroad tracks which is currently attenuating peak flows. Upgrading this culvert in the future could create significant downstream problems, and further study should be undertaken to determine the consequences. Areas of CSP piping on Finholm Street are in poor condition and should be replaced with more hydraulically efficient enclosed concrete storm sewer. The 900 mm dia storm sewer on Blenkin Avenue recommended in the 1998 study has not been finished and should be completed.

44 The Sutherland Basin. Recent works on Molliet that were recommended in the 1998 study have substantially improved capacity in this basin. However,

improved inlet capacities (more catch basins, or installing side inlet grates) would help intercept potential surcharge areas along this route.

45 The McMillan Basin. The older and undersized portions of this system within the downtown core should be upgraded, likely in conjunction with road improvement or revitalization projects in the future. Several of the deficient areas were noted in the 1998 Study.

46 The Bagshaw and Mills Basins. Both have existing highway crossings that create bottlenecks, with the potential to surcharge on Highway 19A. Improved inlet capacity, combined with the twinning of the two highway crossings, and replacement of old CSP infrastructure in the area would mitigate potential surcharging along Highway 19A. Future onsite storage in the undeveloped lands near Despard Avenue will be needed to mitigate the impact on the downstream infrastructure.

47 The Shelly Creek Basin. The sections of Shelly within the City boundaries are an open channel with culverts at road crossings. Subject to a condition assessment, the inlet capacity at the Butler and Martindale Road culverts should be improved to eliminate potential surcharging.

13 RECOMMENDATIONS

Based on the conclusions reached in this study, the following recommendations are made:

1. That the City should updated their standards to target the 80% full design criteria for the 10 year storm event in an effort to address the potential impacts of climate change.

2. That the City upgrade sections of storm drain that are less than the 300 mm dia minimum standard, as warranted, and as opportunities arise.

3. That the City adopt the use of side inlet catch basins in areas with curb and gutter at low points, to mitigate the potential for clogging, and more readily accommodate overland flow during surcharged events.

4. That where feasible, the City requires future developments install measures to limit the post-development flows to pre-development levels to mitigate or reduce the need for downstream improvements to the City's infrastructure.

5. That the City formally adopt the new IDF curve created for this study.

6. That future improvement projects listed in this report, whether undertaken by developers, or by the City, base the design on the design flow and available grade of the proposed pipe, and not strictly based on the suggested pipe diameter noted in this report.

7. That the City continue to upgrade existing roadways to curb and gutter standard, improve the potential overland flow paths in areas of expected surcharged conditions, and improve inlet capture with additional and more efficient catch basin inlets.

8. That the City develop an up to date library of all existing video inspection reports to confirm which mains have been video inspected, which videos are still considered to be current, and which mains that have not been inspected.

9. That the City undertake a systematic video inspection program of the City's storm drainage system and develop an inventory of the system to confirm the information used in the model, and assess the condition of the existing system. The information checked against the City's GIS database and the City's records should be updated accordingly.

10. That the City have the computer model developed for this study continually updated as new storm works are installed, and as the annual video camera investigations are completed, so that the model can continue to be an effective planning and design resource.

11. That the City continues to maintain and monitor the rainfall gauges, and collect rainfall data for ongoing calibration purposes, assessing rainfall patterns and verifying the intensities noted in the IDF curve.

12. That the City continue its maintenance and inspection of the storm drainage system, and formally document any areas that are deficient, problematic, or differ from the information used in the storm drainage model.

13. That the City confirm that the diversion chambers located in the Carey/ Romney Creek Basin are functioning adequately during heavy rainfall events.

14. That the City formally update this Master Plan every seven years to ten years, based on the amount of upgrades and improvements that are implemented.

15. That the City consult and collaborate with the RDN and MoTi regarding land-use policies in the upstream portions of the catchment areas that are tributary to the City's drainage system, but not located within the City. If not already in place, the City should request the right to be consulted and to have input on any development applications that have the potential to increase storm runoff that flows into the City.

16. That the City consult with MoTi and formally request that the City be consulted before any upgrades or improvements are undertaken to MoTi owned drainage infrastructure located upstream of the City's boundaries.

17. That the City consult with both the ICF and their operator SVI, and formally request that the City be consulted before any potential upgrades are undertaken to the drainage infrastructure that is within the E&N rail corridor and located upstream of the City.

18. That the City consult with both the ICF and their operator SVI, and discuss possible improvements in areas where City development directly abuts the existing rail road and is susceptible to seepage and flooding caused by the berming effect of the rail road ballast, and /or poor lateral ditching or drainage along the tracks. The City should review opportunities where improved ditching on the upstream side of the tracks, and where formalized culvert crossing points could mitigate drainage problems in downstream areas of the City.

19. That the drainage improvements listed in Table 13 and shown on drawings 1346-1 to 1346-7 be included in the City's capital projects list, and implemented in order of priority, as development patterns facilitate, and in conjunction with other improvement projects as warranted.

20. That the current Storm Sewer DCC projects list be reviewed and updated to reflect the applicable projects noted in this study.