City of Parksville

Sanitary Sewer Master Plan

OCTOBER 2015

KOERS
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Consulting Engineers

PARKSVILLE, BC

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October 13, 2015
1346 – SSMP

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

Attention: **Ms. Rosa Telegus, PEng**

Dear Sirs:

Re: City of Parksville
SANITARY 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 Sanitary Sewer Master Plan, October 2015".

This study examines the adequacy of the sanitary sewer system under two conditions:

- Current (Year 2014) conditions with an estimated service population of 22,000 consisting of 12,000 permanent residents and 8,000 tourists, and
- Full build-out of the Official Community Plan, which is projected to occur in Year 2072. The service population at OCP build-out is projected to be 33,200 consisting of 22,000 permanent residents and 11,200 tourists.

The sanitary sewer system was analysed using the computer program XP-SWMM; a program that interfaces with AutoCAD and the City's GIS program MapGuide. A calibrated model was developed using flow monitoring data from the three temporary stations installed as part of this study as well as data from the two permanent flow monitoring stations. The calibrated model was used to create the Year 2014 and OCP build-out models.

The current (Year 2014) conditions computer model indicate that, based on record drawing data, all City pipes flow less than 70% full. At OCP build-out, six City pipes are calculated to flow more than 70% full, based on the record drawing data.

The current (Year 2014) computer model results show one section of RDN main to flow above its 50% full design standard for mains up to 250 mm in diameter. At OCP buildout, several sections of the RDN owned interceptor upstream and downstream of the Bay Ave pump station are calculated to flow above the RDN's 100% full design standard for mains 450 mm in diameter or larger.

Updating of the City's design standards is recommended based on the findings of the flow monitoring program and a review of pipe sizing design criteria.

…/2

October 13, 2015
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City of Parksville
Ms. Rosa Telegus, PEng

The existing conditions computer model should be updated regularly as developments occur, as additional flow monitoring data comes available, and when site investigations (such as video inspection), reveal differences between the model input data (pipe diameter, material type, pipe slope) and actual conditions.

The computer model can and should be used to assess the impact proposed development or redevelopment will have on the infrastructure downstream and if upgrading works are required.

The conclusions and recommendations of this study should be reviewed, especially if there are major departures for the assumed flows, changes to development patterns, or adjustments to the City's boundaries.

We thank you for the opportunity to have worked on this assignment. We will be pleased to assist the City with working with the model as development proposals are being considered and will implementing the study recommendations.

Please call if you have any questions.

Yours truly,

KOERS & ASSOCIATES ENGINEERING LTD.

Oct. 16 2015

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

Enclosure

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EXECUTIVE SUMMARY

1996 Study, Growth Projections

The current Sanitary Sewer Study, that Staff reference for development applications and for prioritizing capital work projects, was developed in 1996; almost 20 years ago. Many of the lands that were undeveloped in 1996 are now fully developed and a number of properties have been redeveloped. The City's population has seen significant growth, increasing by more than 20% from 10,000 in 1996 to more than 12,000 today. Continued growth is anticipated and planned for in the City's Official Community Plan.

Commercial business in the City has also seen a large increase, notably in the resort accommodation sector where there are now approximately 2,000 accommodation units in the Parksville Area, exclusive of bed & breakfast units. During the summer, the service population increases significantly as people come to vacation in Parksville and Vancouver Island. During the July/August long weekends, the resort/tourism population is estimated to peak at approximately 8,000 people; resulting a combined population of around 20,000 (12,000+8,000).

Since 1996, land-use bylaws have been updated and a new Official Community Plan was adopted in 2013 (Bylaw 2013, No. 1492). The projected build-out service population is 33,200, including a permanent population of just under 22,000. This is lower than the 41,600 service population projected in the 1996 study. Based on available growth projections, it is estimated that the OCP build-out would occur around Year 2072 (57 years).

Collection System (Gravity Mains & Pump Stations)

Parksville Owned & Operated

The City's sewer collection system consists of 85 kilometres of gravity sewer main; the majority of which (70%) are 200 mm diameter or less. Approximately 50% of the system is less than 30 years old and the remaining 50% is between 30 and 60 years old. The City owns and operates two sewage pump stations and their associated forcemains. They are the Craig Bay and Martindale pump stations which service the lands east of the Englishman River.

Regional District of Nanaimo Owned & Operated

Within the City, the Regional District of Nanaimo (RDN) owns and operates approximately 3.3 kms of gravity trunk leading to and away from the Bay Ave pump station and its forcemain which are also owned and operated by the RDN. Approximately 1.4 kms of gravity main (600 mm diameter) leads to the pump station. The remaining 1.9 kms of gravity sewer main (675 mm diameter) comes after the pump station. The station's forcemain (450 mm diameter) is approximately 1 km long. This system was constructed in 1978 (37 years ago). Over the years, the pump station has been upgraded including increasing its pumping capacity.

Some lands within the RDN are serviced by sanitary sewer mains that discharge into the City of Parksville's collection system. East of the Englishman River, the Pacific Shores Resort discharges into the Craig Bay foreshore trunk main that conveys sewage to the Craig Bay pump station. This 1,200 m long section of the foreshore trunk main is owned and operated by the RDN. West of the Englishman River, flow from approximately 250 properties in the Neden Way/Riley Road area enters the City near the intersection of Hwy 19A and Aberdeen Drive and flows north through approximately 700 m of RDN owned gravity sewer main.

Flow Monitoring

As part of the study, sewage flows were recorded in the field at five monitoring sites (manholes) during a 3 ½ month period between December 2013 and February 2014 to assess the extent of inflow & infiltration (I&I) in the system. Inflow occurs when stormwater runoff enters the collection system by a directly connected pipe, e.g. a storm drain service, or through non-watertight manhole lids. Infiltration occurs when groundwater enters into the collection system at cracks or non-watertight joints.

Two rainfall events occurred during the monitoring period that resulted in a large increase in flows at each site. Neither were unusual storms. The larger of the two had a return period of slightly more than 2 years. The calculated peak I&I rate at two monitoring sites exceeded 12,000 L/day per ha. This was higher than the generally acceptable allowance of 5,600 to 11,200 L/day per for the age of the collection system upstream of the monitoring sites. At two of the other sites, the calculated I&I ranged from 7,500 to 10,700 L/day per ha. At the remaining site, the data was not usable as the meter was determined to have under-recorded the flows. This was at the City's permanent flow monitoring site which was installed a number of years ago in the foreshore Community Park. The meter was recalibrated in September 2014 and is now recording correctly.

<u>Inflow &Infiltration (I&I) Management</u>

While I&I will always be a part of a sewer collection system, large I&I flows can result in the unnecessary oversizing of infrastructure (mains, pump stations, forcemains, and sewage treatment plants).

The findings of the flow monitoring program supports the development and implementation of an I&I management plan.

<u>Design Flows</u>

A detailed analysis of the daily flow data recorded at the RDN's Ocean Place flow meter, found the City's current design standard of 410 L/day per person is high and design flow of 300 L/day per person is more appropriate.

The City's I&I design standard of 8,460 L/day per ha was found to be too low and a value of 12,500 L/day per ha is recommended.

It is recommended that design flows for hotels and motels be added to the City's design standards with values of 300 L/day per guest and 500 L/day per guest; respectively.

It is recommended that the City adopt an equation for determining peaking factor. The equation ($6.75P^{-0.11}$), more closely reflects the flow monitor data. The current design standard range of 4 to 5 was found to be excessively high.

<u>Design Criteria</u>

It is recommended that the City's design standard for the conveyance capacity of gravity mains be based on 70% full. The current standards do not include conveyance capacity criteria.

<u>Computer Model & Calibration</u>

The program XP-SWMM was chosen for analyzing the sewage collection system. XP-SWMM 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.

The model was calibrated using the flow measurements recorded in the field at the five flow monitoring sites, with the computer model calculating slightly higher flows.

<u>Modelling Results</u>

Existing Conditions – City Owned Infrastructure

The computer modelling results did not identify any capacity concerns. This appears to be consistent with information provided by operational Staff of the couple of system maintenance issues regarding grease and debris build-up and tree root infiltration. These would not be identified by modelling. It should be noted that the compiled computer model is based on the City's GIS database and information from record drawings. The accuracy of this data is has not been confirmed. There may be capacity issues that have not been identified by the model because the installed pipe diameter or its slope and material type is different.

Existing Conditions –RDN Owned Infrastructure
Three sections of the gravity trunk main downstream of the Bay Ave pump station are calculated to be just over the RDN design requirement of 100% full.

The trunk main in the sideyard Statutory Right-of-Way (SRW) between Aberdeen and Temple is calculated to be operating at 100% capacity. The RDN design standard for this 200 mm diameter pipe is 50%.

OCP Build-Out – City Owned Infrastructure
A 147 m long section of the Craig Bay foreshore main west of the Craig Bay pump station is calculated to be at 94% full during peak loading

The 380 m section of Corfield Rd north of Stanford Ave is calculated to flow more than 100% full during peak flow. The main should be upsized to 250 mm diameter subject to confirmation of the available grade.

OCP Build-Out – RDN Owned Infrastructure
The gravity main leading to the Bay Ave pump station, the pump station, and the gravity main downstream of the station are calculated to be undersized to convey the peak flow by OCP Build-Out.

The trunk main in the sideyard SRW between Aberdeen and Temple is calculated to be operating at 100% capacity. The RDN design standard for this diameter of pipe is 50%.

<u>Video Inspection & Capital Planning</u>
Video inspection of all mains in the sewer 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
- Development of an I&I management program
- 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 4 to 8 kms of pipe per year would result in the inspection of the 85 kms of pipe over a 10 to 20 year period.

1 INTRODUCTION

1.1 Authorization

In December 2013, the City of Parksville authorized Koers & Associates Engineering Ltd. to develop a Master Plan for the City's sanitary sewer system. The study was to update and analyze the City's sanitary sewer computer model for current conditions and for the future development based on build-out in accordance with the City's 2013 Official Community Plan – A Vision for the Future (OCP). The study was to be carried out in accordance with Koers' proposal dated November 29, 2013.

1.2 Background & Previous Studies

The two most recent studies of the City's entire sewage collection system were:

- Sanitary Sewer Study Update, September 1996, and
- Sanitary Sewer Study, November 1990.

Both were prepared by Koers & Associates Engineering Ltd. The objectives of the 1996 study included assessing the City's sewer system, using the computer model SANSYS, to support existing and projected development and identify improvements necessary to support full build-out in accordance with the OCP. The ultimate service population was estimated at over 41,000 in the 1996 study including an allowance for 1,274 units in the Rathtrevor Resort area.

Since 1996, the City's permanent and seasonal population has increased but has not reached the ultimate service population projection of the 1996 study. The City's current service population is estimated at approximately 20,000 consisting of a permanent population of 12,227 (as of July 1, 2014), as reported by BCStats, and a seasonal resort tourism population peak of approximately 8,000 during the summer July/August long weekends. Ongoing growth is expected and is being planned for within the City's Official Community Plan.

Over the past 18 years, significant advances in computer modelling have occurred, the City's OCP has undergone several changes, and new and re-development of properties has occurred. The public awareness of water conservation along with the manufacturing and sale of low flow fixtures and appliances and changes to the BC plumbing code, have resulted in a lowering of indoor and outdoor per capita water usage. 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, including sanitary sewer system 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 1996 sanitary sewer 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 Purpose

The City desires to update their sanitary 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 Sanitary Sewer 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. Reviewing the potential impact of climate change.

4. Assessing impact of the failure of key infrastructure components (trunk mains and pump stations).

1.4 Scope of Work

To meet the study purpose, 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 in 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. Identify 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. Review and analyze flow data in conjunction with rainfall and catchment area characteristics. Estimate Inflow & Infiltration (I&I) rates and compare with City design standard. Summarize findings in a Technical Memorandum complete with data tables, graphs, charts, photographs, drawings, and conclusions.

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

4. Carry out an evaluation of several sanitary sewer computer programs including; compatibility with 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 including unit flow rates per land use category, I&I rates, peaking factors, pipe sizing criteria. Recommend design criteria to be used for existing conditions and for OCP build-out. Present findings in a Technical Memorandum *(Technical Memorandum Nos. SSFW & SS-2 submitted August 29, 2014)*.

6. Develop a computer model of the existing sanitary sewer collection system using digital information for the City's Hydra 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 identify 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 sewer collection system. The findings are to be used to develop a priority list of the infrastructure for capital projects.

8. Present findings in a Master Sanitary Sewer Plan report complete with; tables; colour figures; maps; tables; priority list of projects with construction cost estimates suitable for budgetary purposes; identification if project is a capital works, DCC, or development project; identification if 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, PEng. – Director of Engineering
- Ms. Rosa Telegus, PEng – Development Engineer
- Mr. Blaine Russell, MCIP – Director of Community Planning
- Mr. Fred Pakkala – Engineering Technologist
- Mr. Randy Hall – GIS Technician
- Ms. Barbara Silenieks – Engineering Technologist
- Mr. Connor Bankes, GradTech – Engineering Technologist

We also wish to acknowledge with thanks the assistance provided by the following Regional District of Nanaimo Staff in the supply of flow record data from the Ocean Place flow meter:

- Mr. Maurice Mauch, AScT – Project Engineer
- Ms. Jessica Dorzinsky – Special Projects Assistant
- Mr. Adrian Limpus – Engineering Technologist

2 EXISTING SYSTEM

2.1 General

The municipal sewage collection, treatment and disposal system was installed in 1963, servicing the village core area. Treatment was by an Imhoff tank (essentially a large septic tank providing primary treatment) located at the foot of Corfield Street. The treated effluent was disposed of by an outfall to the Strait of Georgia. In 1976 the collection system was significantly expanded to the northwest to serve the area between the Island Highway and the shoreline.

In 1980 the Regional District of Nanaimo (RDN) completed the construction of the French Creek Water Pollution Control Centre, the Parksville Interceptor trunk sewer, and the Bay Street Pump Station, forcemain and gravity trunk main. The Imhoff tank and ocean outfall were then abandoned.

In 1991 the City incorporated the Parksville East Improvement District and extended the sanitary sewer system to service the area.

In 1994 the City incorporated the Wembley Mall area. This area already had a municipal collection system which was owned and operated by the RDN and flowed into the City's collection system.

In 1995 the City incorporated the Craig Bay area and adjacent Rathtrevor resort areas. A sewage collection system was constructed to service the area along with two pumping stations (Craig Bay and Martindale), their forcemains, and trunk mains to convey under the Englishman River and into the City's sewage collection system. This eliminated two small pump stations along the boundary of the Parksville Flats.

In 2004, the problematic pump station on Despard Ave near Meridian Way was removed when gravity sewers were extended south along Corfield St and west along Despard Ave.

All developed areas within the City boundary are provided with sanitary sewer service with the exception of portions of the City's Industrial Park on the east side of the Englishman River, which is still mostly on individual septic tanks. The Industrial Park can be serviced by gravity to the Craig Bay trunk sewer by extending lateral sewers into the subdivision.

2.2 Collection System

The City's GIS database, along with record drawings of recent subdivision projects not yet entered into the database show that the City's collection system consists of 85 kilometres of gravity sewer main. Almost 70% of the collection system is made up of gravity mains that are 200 mm diameter or less. Approximately 50% of the gravity mains are reported to be less than 30 years of age and the remaining 50% more than 30 years and less than 60 years of age.

There are six trunk mains in the City, of which three are owned by the City and three are owned by the RDN. The three owned by the City are described below. The RDN owned mains are described further on in this report in section 2.4.1 Trunk Mains.

Craig Bay Pump Station Trunk Sewer
This 2.5 kilometer long, 300 mm to 400 mm diameter gravity trunk sewer runs along the Craig Bay foreshore servicing the waterfront resorts including Pacific Shores Resort located in the RDN. The Craig Bay pump station is located at the west end of Saltspring Place within the Craig Bay Estates, a residential strata development.

Martindale Pump Station Trunk Sewer
This 1.6 kilometer long, 375 mm to 750 mm diameter gravity sewer receives sewage from the Craig Bay pump station and conveys it west to the Martindale pump station located on the west bank of the Englishman River at the east end of Despard Ave. Sewage is conveyed under the river by a 750 mm diameter main before flowing into the pump station.

Parksville Bay Trunk Sewer
This 2.1 kilometer long, 525 mm to 600 mm diameter gravity sewer receives sewage from the Martindale pump station as well as the majority of the City east of Molliet St. The trunk main begins at the intersection of the Old Island Highway and Martindale Rd and runs along Martindale Rd, Turner Rd, Shelly Road, Mills St, the estuary park, Nebrus Lane, and Corfield Road where it connects to the RDN trunk main that begins next to the curling rink in the foreshore Community Pak and runs west to discharge into the Bay Ave pump station.

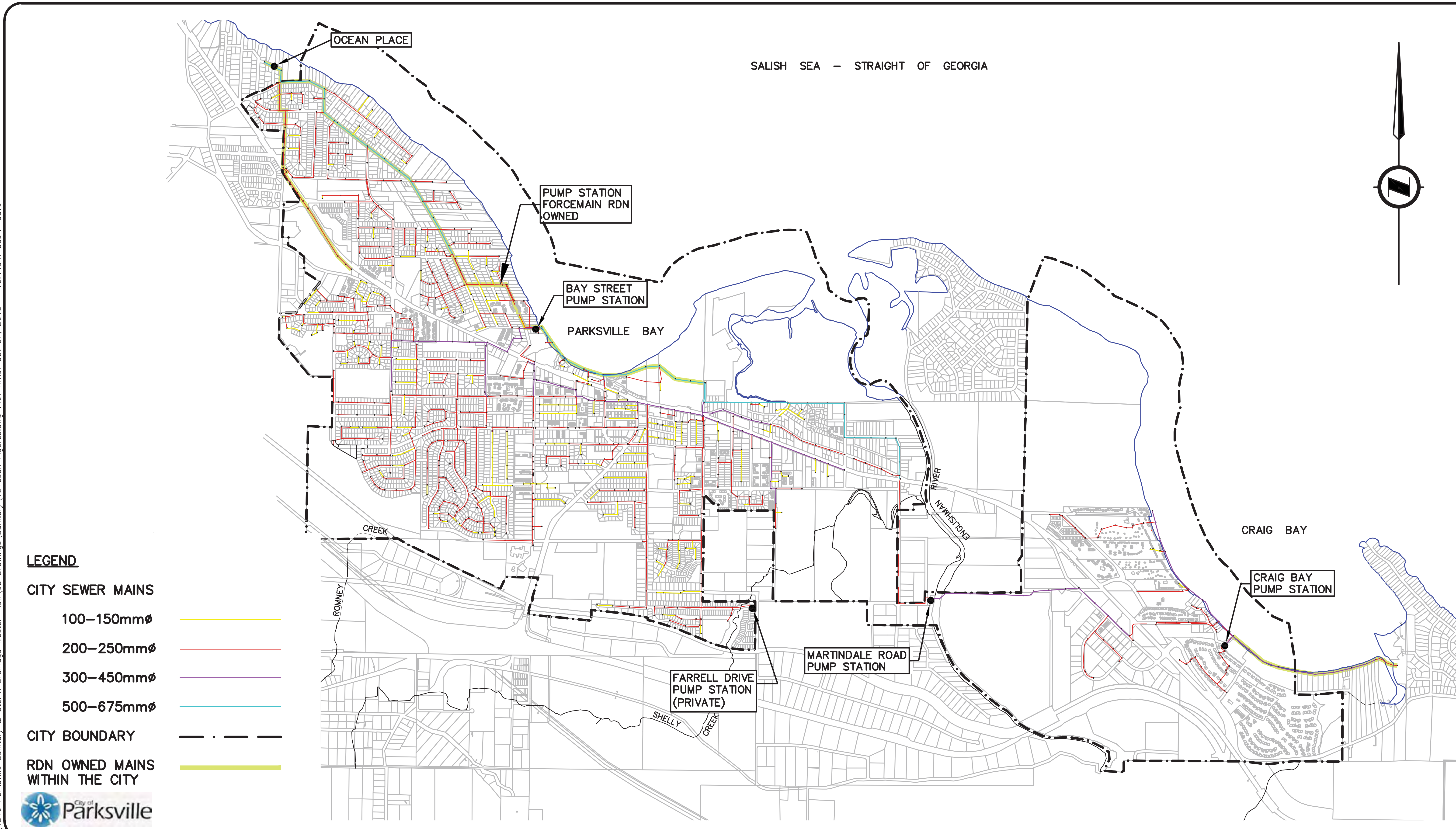
Table 1 presents the total length and percentage of gravity sewer pipe in the City by pipe diameter and age. The collection system is shown on **Figure 1**.

Table 1 – Gravity Pipe Lengths and Age by Diameter

Age (years)	Pipe Diameter (mm)													
	100	**150**	**200**	**250**	**300**	**350**	**375**	**400-450**	**500-525**	**600**	**675**	**700**	**750**	**?**	**Total**
0 - 10		823	3,962	1,220	593		296								6,893
11 - 30	123	3,317	20,009	2,802	3,748	104	593	1,017	908	1,141			66		33,829
31 - 60		9,214	19,394	2,807	2,161	434		2,613	137	1,360	2,077	27			40,222
?		388	1,460	206	65		22							1,762	3,904
Total	123	13,742	44,825	7,035	6,566	538	911	3,631	1,044	2,502	2,077	27	66	1,762	84,848
Length as % of Total by Diameter															
0 - 10		1	5	1	1										8%
11 - 30		4	24	3.5	4		1	1	1	1					**40%**
31 - 60		11	23	3.5	3	1		3		2	2				**47%**
?			2											2	5%
Total		16%	**53%**	8%	8%	1%	1%	4%	1%	3%	2%			2%	100%

There are just over 5 kilometres of forcemain in the City servicing three pump stations. More than 80% of it is under 300 mm diameter and less than 20 years old; consisting of the forcemains servicing the Craig Bay and Martindale pump stations which were constructed in 1995/96. The remaining 20% (0.94 kms) is the 450 mm diameter forcemain that services the Bay Ave pump station) and is 37 years old. The Bay Ave forcemain is owned by the RDN.

Table 2 presents the total length and percentage of forcemain installed in the City by pipe diameter and age.



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CLIENT	CITY OF PARKSVILLE		
PROJECT	SANITARY MASTER PLAN		
TITLE	COLLECTION SYSTEM		
APPROVED		SCALE	1:25,000
DATE	FEB 2015	DWG No.	FIGURE 1
PROJECT No.	1346		

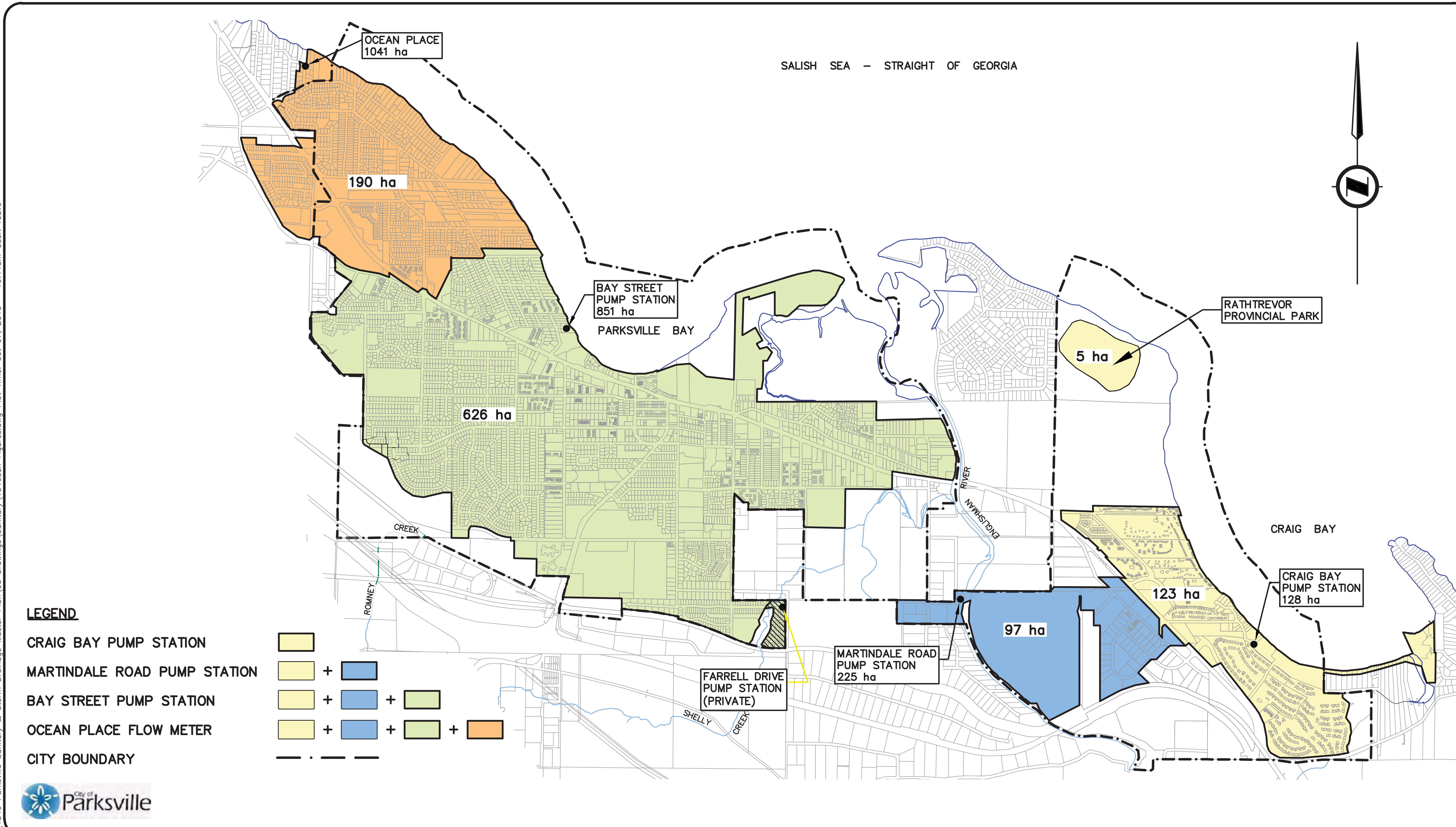
Table 2 – Forcemain Pipe Lengths and Age by Diameter

Age (years)	Length (m) by Diameter (mm)			
	200 mm	300 mm	450 mm	Total
19 to 20	3,000 m	1,140 m	-	4,140 m
37	-	-	940 m	940 m
Total	3,000 m	1,140 m	940 m	5,080 m
	Length as % of Total by Diameter (mm)			
19 to 20	59%	22%	-	81%
37	-	-	19%	19%
Total	59%	22%	19%	100%

2.3 Pump Stations

There are three municipal pumping stations within the City of Parksville. Two are owned and operated by the City of Parksville, the other and the largest of the three, is owned and operated by the RDN. There is at least one privately owned pump station within the City. It services the 49 lot strata single family subdivision on Farrell Drive. There may be other privately owned pump stations in the City. A brief summary of the characteristics of each station is presented in **Table 3**. It is based on record drawings and information provided by City of Parksville and Regional District of Nanaimo operational Staff, unless noted otherwise.

The service area of each station and of the flow meter at Ocean Place, which meters all flow from the City of Parksville, is shown on **Figure 2**.



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CLIENT	CITY OF PARKSVILLE
PROJECT	SANITARY MASTER PLAN

TITLE	PUMP STATIONS & OCEAN PLACE FLOW METER CATCHMENT AREAS		
APPROVED		SCALE	1:25,000
DATE	FEB 2015	DWG No.	FIGURE 2
PROJECT No.	1346		

Table 3 – Pump Station Characteristics

Description	Pump Station Name			
	Craig Bay	Martindale	Farrell Drive	Bay Avenue
Owner	Parksville	Parksville	Private	RDN
Year Built	1996	1995	2000?	1978
Location	Saltspring Place (west end in Craig Bay Estates)	Despard Ave (east end, next to Englishman River)	Hamilton Ave @ Farrell Dr	Bay Ave (east end, on foreshore)
Service Area (ha):				
- Existing	123	159	3.4	725
- Ultimate	123	343	3.4	846
Pump Data				
No of Pumps:				VFDs *
- Existing	2	2	2	4
- Ultimate	3	3	2	4
Manufacturer	Flygt	Flygt	Flygt **	Flygt
Model	C 3201 HT	C 3201 HT	CP 3085 **	CP 3201 HT
Impeller	457	454	n/a	452
Pumping Rate (L/s):				
- One Pump	38 - 42 #	51 - 57 ##	5 ***	73 (max)
- Two Pumps	48 - 54 #	60 - 69 ##	6 ***	137 (max)
Motor Data				
Power	47 Hp / 35 kW	30 Hp / 22 kW	n/a	47 Hp / 35 kW
Phase	3 phase	3 phase	1	3 phase
Emergency Power Generator				
Automatic Start	Yes	Yes	n/a	Yes
Manufacturer	Cummins/Onan	Cummins/Onan	n/a	n/a
Model	125 DGEA	125 DGEA	n/a	n/a
Fuel:				
- Type	Diesel	Diesel	n/a	Diesel
- Tank Capacity (L)	n/a	n/a	n/a	n/a
Wet Well Data				
Diameter	3.1 m x 4 m	3.1 m x 4 m	1.8 m	3 m x 4.9 m
Depth	7.1 m	8.1 m	4.6 m	7.3 m
Storage Volume Between (m^3):				VFDs *
- Pump Off & On	5.0 m^3	13.3 m^3	3.3 m^3	n/a
- Pump On & High Level Alarm	5.0 m^3	21.7 m^3	1.4 m^3	n/a
Forcemain				
Number of Forcemains	2	2	1	1
Diameter, (mm)	200 mm	200 & 300 mm	100 mm	450 mm
Length, (m)	930	1,140	113	940 m
Material	HDPE, DR 26	PVC	PVC	DI
Static Head, (m)	21.1	7.7	2.75	26.4 +/-
Point of Discharge	Industrial Way @ Huntley Rd	Martindale @ Hwy 19A	Hamilton Ave	Temple St @ Doehle Ave

Notes:

* Variable Frequency Drive.

** As noted on pump station design drawings. Pumps presently in use unknown.

*** Based on flow monitoring data. See discussion under **Section 4.4.2 Inflow/Infiltration Estimates**, 171 Corfield (SMH 783).

\# Based on a Hazen Williams friction coefficient (C) of 110 and 130 for HDPE pipe and using only one of the two forcemains as shown in **Appendix A** in the Composite Pump Curves figure from the pump station O&M Manual.

\## Based on a Hazen Williams friction coefficient (C) of 110 and 130 for HDPE pipe and using only the 200 mm diameter forcemain as shown in **Appendix A** in the Composite Pump Curves figure from the pump station O&M Manual.

n/a Information Not Available.

With the exception of the Bay Avenue station operated by the RDN, all pump stations have vehicle access. At Bay Avenue, operators must descend three sets of concrete stairs to access the station. Removal of the pumps requires the use of a crane. A photograph and summary of each station is presented in **Appendix A**.

2.4 RDN Infrastructure

The RDN operates: regional trunk sewers; pump stations; and the sewage treatment plant in French Creek (French Creek Water Pollution Control Centre), all of which service the City of Parksville and other local communities in the area.

A brief description of the infrastructure (trunk mains and pump stations) located within the City of Parksville is presented below.

2.4.1 Trunk Mains

The RDN owns and operates three trunk mains in the City of Parksville, brief descriptions of which are as follows:

<u>Parksville Bay Foreshore Trunk Sewer</u>
This 1.4 kilometer long, 600 mm diameter gravity trunk sewer runs along the foreshore from the foot of Corfield Street, west into the Bay Ave pump station located on the edge of foreshore at the end of Bay Avenue.

<u>Bay Avenue Forcemain &Temple Street Trunk Sewer</u>
The Bay Avenue pump station has a 940 m long, 450 mm diameter forcemain that runs along Bay Avenue, Dogwood Street, Rushton Ave, and Temple Street where it discharges into a gravity sewer main at the intersection of Temple Street and Doehle Ave.

The Temple Street gravity trunk sewer is a 1.9 kilometer long, 675 mm diameter concrete pipe within the City's boundary. It runs west along Temple Street, then north along Sunray Road, west along Wright Road and exits the City's boundary in an approximately 75 m long Statutory Right-of-Way as it heads north to the east end of Cavin Road within the RDN.

<u>Hwy 19A Trunk Sewer</u>
This 1.1 kilometer long, 200 mm diameter gravity trunk sewer starts at the Oceanside Place complex and runs west along Hwy 19A, north along Aberdeen Drive, through a sideyard Statutory Right-of-Way, north along Field Crescent, through a sideyard Statutory Right-of-Way, and crosses Wright Road where it connects to the RDN trunk main that services the Bay Ave pump station.

2.4.2 Bay Avenue Pump Station

The Bay Avenue pump station is located adjacent to the foreshore of Parksville Bay at the east end of Bay Avenue. The characteristics of the station are noted in **Table 3**. As noted previously, operators must descend three sets of concrete stairs to access the station. There is no vehicle access to the pump station. Pump removal can only be accomplished with the use of a crane.

The foreshore of the site is armoured by large diameter rip-rap overtop of filter cloth. Some of the rip-rap is missing, exposing portions of filter cloth and allowing the washing away (erosion) of soil around the outlet of a CSP storm drain.

2.4.3 French Creek Pollution Control Centre

The French Creek Pollution Control Centre (FCPCC) became operational in 1980. Sewage is conveyed to the treatment plant by way of gravity mains and three main pumping stations along the foreshore:

- Bay Avenue Pump Station (in the City of Parksville);
- Hall Road Pump Station (in the Town of Qualicum Beach); and
- Lee Road (in French Creek).

The treatment plant provides secondary treatment to a service population of approximately 27,000 people, including the approximately 12,000 people in the City of Parksville. The treatment plant has a permitted capacity of 16,000 m^3/day and effluent BOD/TSS discharge limits of 45/45 mg/L.

After treatment, the effluent is discharged by gravity into the ocean by way of a 2.4 kilometre long outfall terminating at a depth of 61 m. Prior to construction of the treatment plant, raw sewage was discharged into the Strait of Georgia (Salish Sea).

More information on the FCPCC can be found on the RDN's website www.rdn.bc.ca

3 POPULATION

3.1 Historic

Since completion of the previous sanitary sewer study (1996) carried out for the City, the permanent population has grown by 25% from its then population of just under 9,800. The City's permanent population, as of July 1, 2014, is estimated at 12,227 as published by BC Stats which provided annual updates as of July 1 each year. During the past 17 years, this growth equates to a population growth of 135 people per year, or 1.24% compounded annually.

During the summer, the service population increases significantly as people come to vacation in Parksville and Vancouver Island. During the July/August long weekends, the resort/tourism population is estimated by the City planning department Staff to peak at approximately 8,000 people; resulting a combined population of around 20,000 (12,000+8,000).

Tourists stay in various locations including approximately 2,000 accommodation units in the Parksville area. These consist of:

- 750 units in the resort area (east side of the Englishman River):
- 840 units in the central area (west of the Englishman River),
- 200 units at Rathtrevor Park (caretaker, 174 camping site & 25 walk-in sites),
- 200± units in the Parksville area not connected to the sanitary sewer system (River Bend RV Park, Big Tent RV Park, and French Creek House Resort), and
- Unknown number of bed & breakfast units in residential homes.

3.2 Projected

According to City planning Staff, the OCP supports a future permanent population of just under 22,000. This growth is anticipated to be accommodated with the construction of 4,980 dwelling units (2,710 single family + 2,270 multi-family). No specific time frame is attributed to when this is expected to be reached. A copy of the Planning Department PowerPoint presentation title "Population & Land Capacity" is presented in **Appendix B**.

BCStats publishes a forecast of future population growth for each regional district for the next 25 to 30 year horizon. The forecasts are updated annually and the most current (as of September 2014) extends to year 2041. The forecast uses the Component/Cohort-Survival method which ages the population while applying births, deaths, and migration forecasts by age. The forecasts are based on past trends which are then modified to account for possible future changes.

For the RDN, of which the City of Parksville is a part, BCStats forecasts the population to increase by 33% between 2014 and 2041.

The City of Parksville's population presently accounts for 8% of the total RDN population, as it has for the past 15 years. Assuming this ratio will continue, the City's permanent population is projected to reach 16,100 by year 2041. Assuming an annual growth rate of 1% beyond 2041, the OCP build-out population of just under 22,000 would be reached in year 2072.

At OCP build-out, the number of accommodation units is expected to increase by 1,175 units; not including bed & breakfasts in residential homes. This growth is expected to occur as follows:

- 430 units in the resort area (east of the Englishman River), and
- 745 units in the central area (west of the Englishman River).

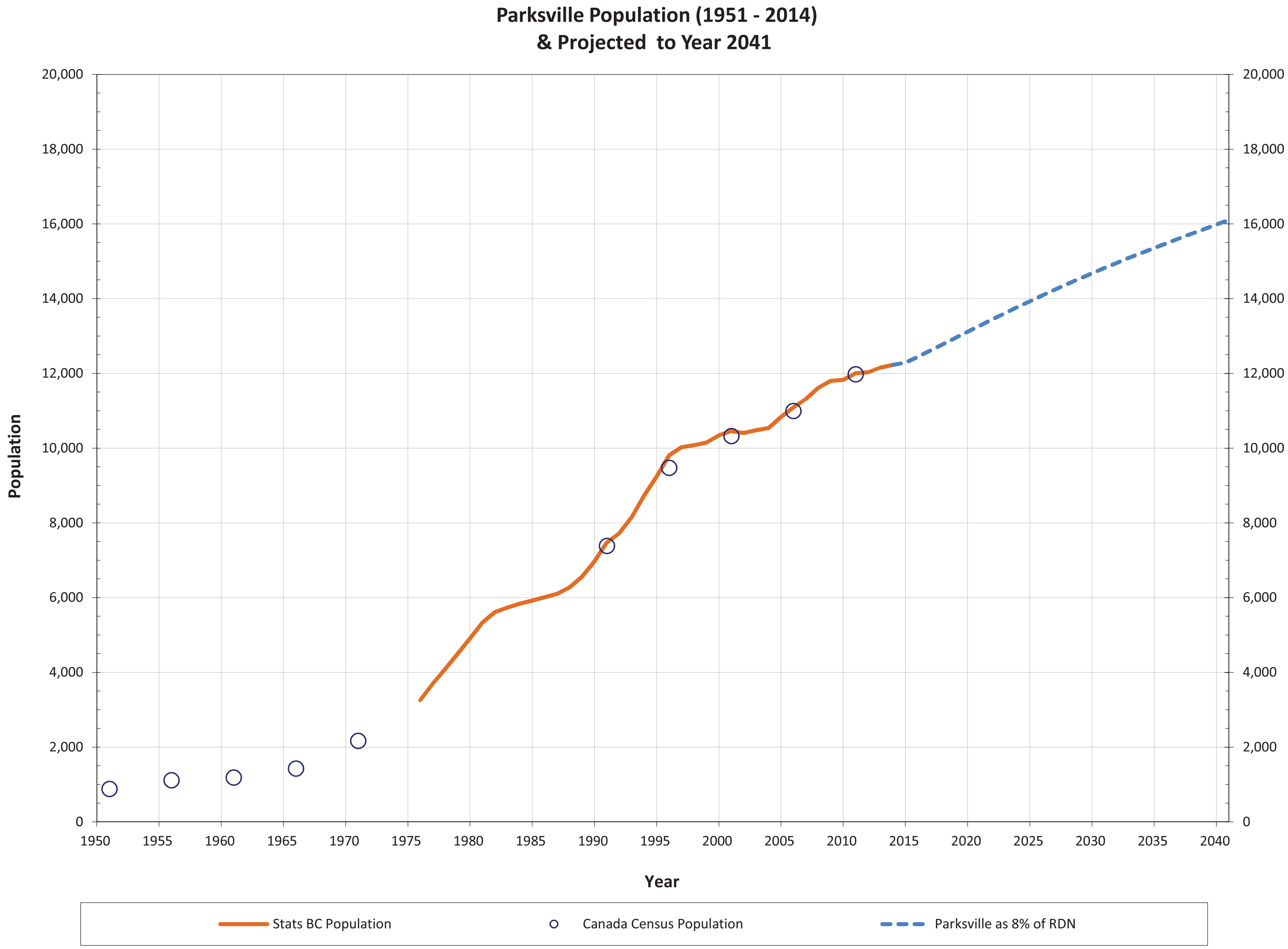
In addition, allowance has been made for all of the Big Bend RV Park, 105 units, to connect to the City's sanitary sewer system.

Figure 3 presents the City's annual population from 1951 to 2014 and projected to 2041. **Table 4** presents current and projected permanent and seasonal population estimates to year 2072 (OCP Build-out).

Table 4 – Projected Population & Dwelling Units

Year	Population			Dwelling Units		
	Permanent	Seasonal Peak	Combined	Permanent	Tourism*	Combined
2014	12,230	8,000	20,230	5,805	1,790	7,595
2020	13,100	8,300	21,400	6,250	1,890	8,140
2030	14,700	8,900	23,600	7,050	2,090	9,140
2041	16,100	9,500	25,600	7,800	2,290	10,090
(1% annual population growth assumed beyond Year 2041)						
2050	17,600	10,000	27,600	8,570	2,450	11,020
2060	19,400	10,500	29,900	9,510	2,620	12,130
OCP Buildout (2072)	21,900	11,200	33,200	10,785	2,965	13,750
Increase, #	9,700	3,200	12,900	4,980	1,175	6,155
%	79%	40%	64%	86%	66%	81%

<u>Build-out Service Population Estimate, 1996 Study</u>
It is noted that this OCP Build-out population (33,200) is less than the 41,600 projected in the 1996 Parksville Sanitary Sewer Study Update (see Table 3.2 on page 9 of that report). The 1996 projection included an allowance for 1,272 tourism accommodation units in the Craig Bay area (see page 7 of that report). When allowance was made for contributing catchments outside of the City to the east and to the west, the projected service population increased to just under 43,400 in the 1996 study (see page 7 of that report).



Parksville Population (1951 - 2014)
& Projected to Year 2041
Population
Year
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
1950
1955
1960
1965
1970
1975
1980
1985
1990
1995
2000
2005
2010
2015
2020
2025
2030
2035
2040
Stats BC Population
Canada Census Population
Parksville as 8% of RDN

FIGURE 3

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 SMH 720 in the parking lot on the west side of curling rink in the Community Park. Further discussion on this installation is presented below in **4.2 Permanent & Temporary Monitoring Sites** under the heading SMH 720 (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 is 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 Permanent & Temporary Monitoring Sites

Sewage flows generated within the City of Parksville are recorded at two ***permanent flow monitoring*** stations as follows:

<u>SMH 720 (Community Park)</u> – This site has been in operation since June 2009 and is maintained by the City of Parksville. It records rainfall and sewage flows in 5 minute increments. The catchment area includes all of the City's lands east of the Englishman River including the resorts along Resort Drive, Craig Bay Estates and Pacific Shores which is located in the RDN.

<u>SHM 36 (Ocean Place)</u> – This site has been in operation since 2007 and is maintained by SFE for the RDN. It records sewage flows in 5 minute increments. Its catchment area consists of all of the City of Parksville, including Rathtrevor Provincial Park, as well as Pacific Shores, to the east of the City, and 250 properties in French Creek, to the west of the City. Originally, it was proposed to review flow data for the RDN Bay Street Pump Station, which services most, but not all, of the City of Parksville. The Ocean Place data was used in its place when the availability of the flow data was made known.

In addition, ***temporary flow monitoring*** was carried out at three additional sites between Dec 2013 and March 2014. These sites were:

<u>SMH 783 (171 Corfield St)</u> – This location was selected as it has a mixture of commercial, older and new residential development, institutional development, and vacant lands.

<u>SMH 547 (102 Acacia St)</u> – This location was selected as it is fully developed with newer single family residential development and would contain gasketed PVC pipe sewer pipe.

<u>SMH 603 (254 Roscow St)</u> – This location was selected as it has a mixture of new and older residential development, public assembly, is mostly developed, and contains the Acacia monitoring catchment area, permitting a comparison of flow data.

Other criteria used in selecting the temporary locations included:

- Manhole compatibility with flow monitoring equipment,
- Downstream side of manholes to capture all upstream flow, and
- Site accessibility and workers safety.

In September 2014, the City installed its portable flow meter at the following location as recommended by Koers:

SMH 560 (631 Blenkin St) – This location was selected as it was a sub-catchment of SMH 603 (254 Roscow St) permitting a comparison of I&I estimates. It has a mixture of new and older residential development as well as a public school.

A summary of each of the monitoring sites is presented in **Table 5**. The location of each is shown in **Figure 4**.

Table 5 – Flow Monitoring Sites

Location	SMH No.	Catchment Area (ha)	No of Lots	Description
Temporary Installations				
171 Corfield St (Dec 2013 – March 2014)	783	55	390+	Mixed use including new and older single family residential and multi-family, commercial, institutional (civic centre). Some un-developed land.
102 Acacia St (Dec 2013 – March 2014)	547	35	400+	Newer single family residential. Fully developed.
631 Blenkin St (Sept 2014 – Dec 2014)	560	72	460+	Mixed use including mostly newer single family residential, multi-family, secondary school and public assembly (two churches). Includes 11 ha of undeveloped land.
254 Roscow St (Dec 2013 – March 2014)	603	171	1,300+	Mixed use including new and older single family residential, multi-family, secondary school and public assembly (two churches) and a few small commercial properties. Majority of area is developed. Includes SMH 547 & 560 catchment areas.
Permanent Installations				
Community Park (Permanent)	720	421	-	Includes SMH 603 catchment area.
Ocean Place - RDN Site (Permanent)	36	930	-	All of City of Parksville including Rathtrevor Provincial Park plus the Pacific Shores development and 250 properties in French Creek.

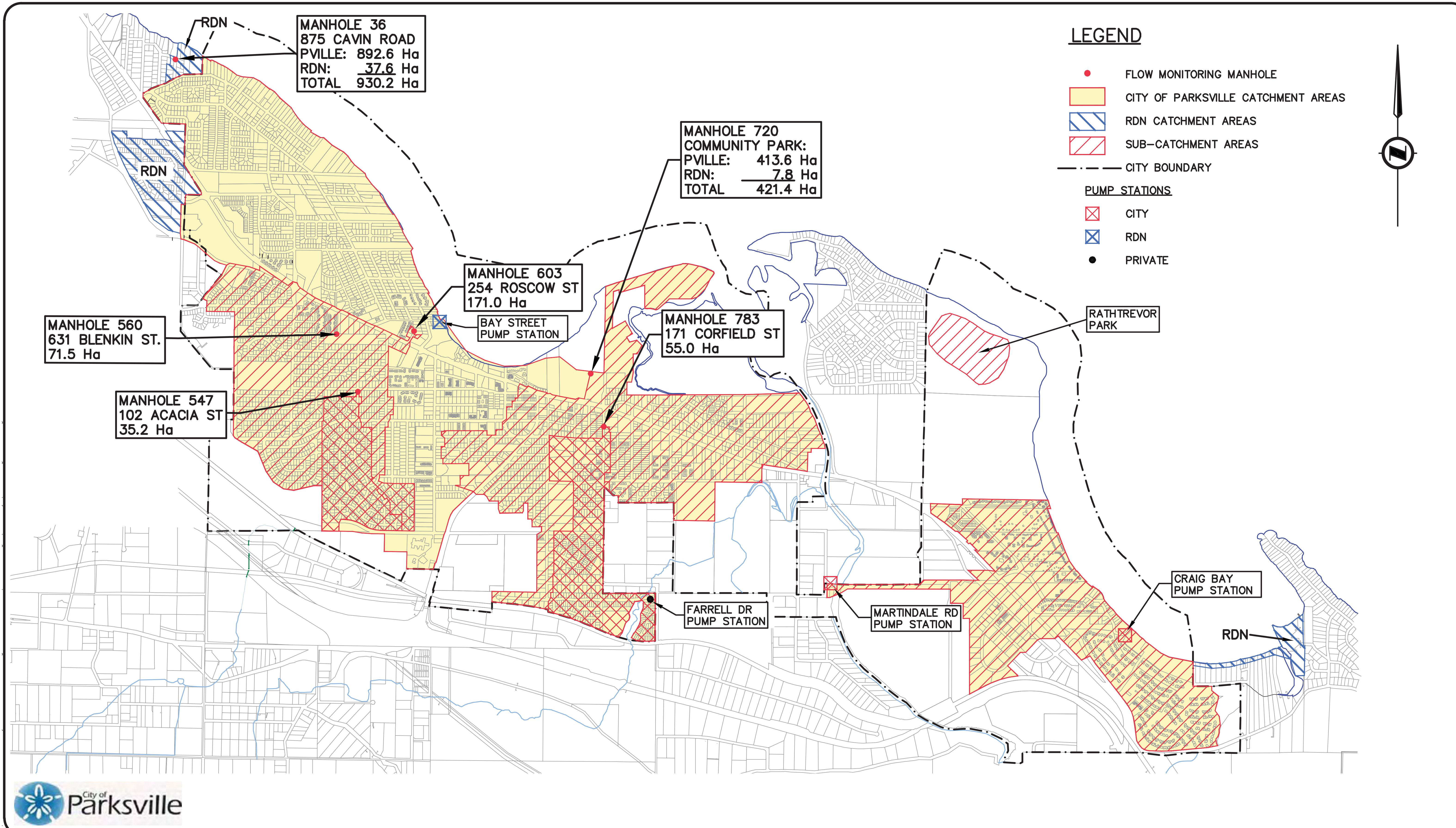
An overview of the findings is presented below. Additional information is presented in **Appendix C - Technical Memorandum No. SSFM (Sanitary Sewer Flow Monitoring)** including the flow monitoring report by SFE Global Ltd. The information in Appendix C is based on the time period ending May 2014. The information presented below is based on the time period ending December 2014.

4.3 Sewage Flows

A review of sewage flows on an annual, monthly, and daily basis was carried out to assess annual, seasonal and daily trends as follows:

4.3.1 Total Annual Flow

A review of total annual flow for the Ocean Place flow meter for the past 18 years (1996 to 2014) showed both flows and population increasing. Since 1996, flows have increased 31%, reaching a peak in the year 2005 of 2,189,000 m^3. For 2014, the total flow was 1,922,000 m^3; an increase of 0.8% from the previous year (1,907,000 m^3) during which time the City's population increased by 0.6%. It is not known how much the service population changed during the past year at Pacific Shores or French Creek. A plot of annual flow vs the City's annual population and published by BCStats is presented in **Figure 5**.



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CLIENT	CITY OF PARKSVILLE
PROJECT	SANITARY MASTER PLAN

TITLE	FLOW MONITORING SITES, TEMPORARY AND PERMANENT		
APPROVED		SCALE	1:25,000
DATE	FEB 2015	DWG No.	FIGURE 4
PROJECT No.	1346		

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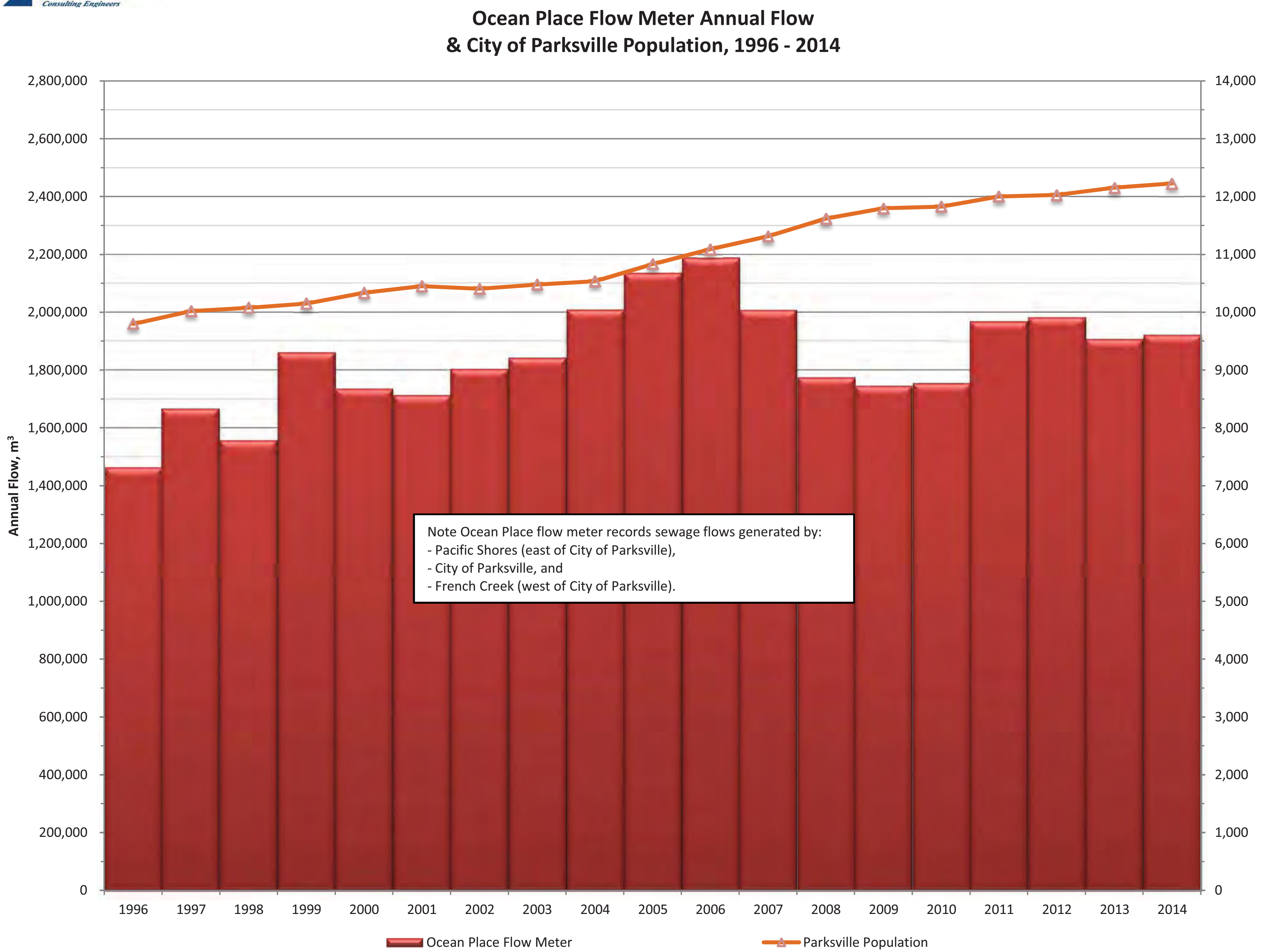

FIGURE 5

4.3.2 Monthly Per Capita Day Flows

For the period 2007 to 2015, monthly average day per capita flows were calculated for the Ocean Place flow meter using the population estimates published by BCStats with allowances for the service population from the contributing catchment areas of Pacific Shores and French Creek, as calculated by the RDN. For Pacific Shores and French Creek, a constant population of 267 and 525 people; respectively, was allowed.

Figure 6 presents the calculated monthly per capita per day flow from 2007 to 2015 along with monthly rainfall totals. Also shown is the calculated annual per capita per day flow compared to the City's design flow standard of 410 lpcd. The 410 lpcd design flow is only attributed to residential land-uses, i.e., does not include flows generated from the other types of land-uses in the City (commercial, industrial, institutional or I&I), while the calculated flow does.

During the past four years (2011 – 2014), the lowest calculated monthly per capita flows per day were nearly identical, ranging from 386 lpcd in year 2011 to 371 in year 2014.

For five of the past eight years, the annual average per capita day flow has been less than 410 lpcd. This indicates the City's actual dry weather residential land-use per capita per day flow is lower than the City's current 410 lpcd design standard. **Table 6** presents the estimated monthly per capita flows for the last eight years along with the average for each year and the City's current design standard.

Table 6 – Estimated Monthly Per Capita Day Flow, 2007 - 2014

Month	Estimated Per Capita Day Flow* (lpcd)							
	2007	2008	2009	2010	2011	2012	2013	2014
January	**534**	**430**	412	**442**	405	446	398	395
February	458	397	**341**	356	423	429	372	408
March	504	387	370	368	**502**	434	408	424
April	468	387	362	380	412	416	**386**	377
May	445	374	358	368	415	**380**	396	**371**
June	460	380	354	354	398	403	413	383
July	420	402	395	382	440	429	418	413
August	469	420	404	394	450	450	**438**	424
September	419	370	363	355	399	399	411	383
October	**394**	**362**	353	**341**	**386**	401	395	404
November	417	385	**452**	382	406	424	405	405
December	448	396	382	435	407	**459**	389	**452**
Annual Average	453	391	379	380	420	422	402	403 *
City Design Standard (For residential population only)	410 **							

Notes:
Bold red text is **highest monthly** value **for the year**.

Bold underlined text is **lowest monthly** value **for the year**.

* For all land-uses (residential, commercial, industrial, institutional) + I&I.

** Only for residential population. Does not include commercial, industrial, institutional, or I&I.

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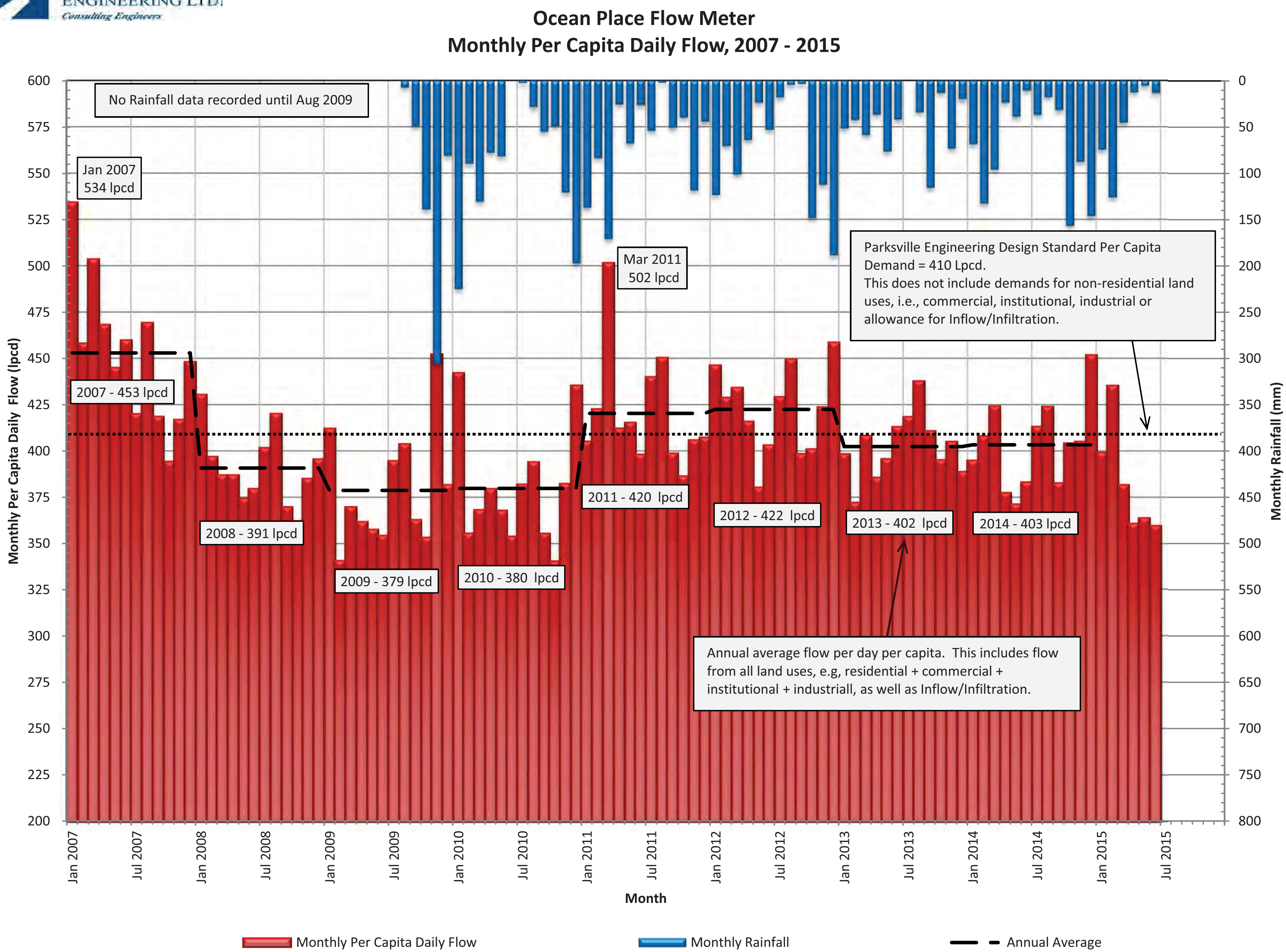

FIGURE 6

4.3.3 Daily Flows

A review of the Ocean Place flow meter daily flow was carried out to assess seasonal and daily changes. Flow data recorded at the City's temporary monitoring stations was overlain with each other as well as with the Ocean Place station providing a good visualization of how flows varied from each other on a daily basis. It also provided insight into how changes in upstream catchments impacted downstream catchments.

Flows to the Ocean Place flow meter peak during the wet winter months followed by a second, but lower, peak in the dry summer months. The winter peak is in response to rainfall which creates I&I in the sewage collection system, while the summer peak is in response to the increase in the local population by tourism. In the spring and the early fall, sewage flows gradually decrease in response to the reduction in rainfall and the decline in the tourism population; respectively.

Increases in flow were recorded at each of the five flow monitoring stations during rainfall events. Conversely, when the rainfall stopped, flows decreased; a few of the stations more quickly than the others, suggesting inflow was more prevalent than infiltration. A graph of daily flows vs rainfall for the past two years (October 2012 to December 2014), is presented in **Figure 7**. The seasonal and daily changes in response to rainfall and summertime tourism are clearly evident. A detailed review of I&I rates for each monitoring site is presented under the next section below.

A comparison of the flows recorded at the Community Park station with those recorded at the upstream and downstream monitoring sites (254 Roscow St and Ocean Place; respectively) suggested the flow meter was not recording correctly as noted on **Figure 7**. This was subsequently confirmed by SFE Global Ltd and the monitoring equipment subsequently corrected in September 2014 as discussed in **Section 4.5**.

4.4 Inflow & Infiltration

4.4.1 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, operating since August 2009. The rainfall data reported in SFE's flow monitoring reports are from the Community Park station, and is the station used in this report. A copy of SFE report is located in **Appendix C - Technical Memorandum No SSFM (Sanitary Sewer Flow Monitoring).**

Two notable rainfall events occurred during the 3 ½ month monitoring period (Dec 2013 – March 2014) while a series of three storm events occurred over three consecutive days in December 2014, after the temporary flow meters had been removed. There was also a short duration high intensity rainfall event at the beginning of September 2013 which caused localized flooding, most notably in the Corfield Rd/Hwy 19A intersection area. The rainfall amounts during these events were as follows:

- September 2, 2013 = 33.2 mm (30 minute storm)
- January 10 & 11, 2014 = 45.2 mm (18.8 + 26.4)
- February 15 & 16, 2014 = 44 mm (16.6 + 27.4)
- Dec 9, 10 & 11, 2014 = 84.8 mm (24.6 + 32.8 + 27.4)

While all of the events had rainfall intensities that plotted well below a five year return period on the City's Intensity-Duration-Frequency (IDF) curves, excluding September

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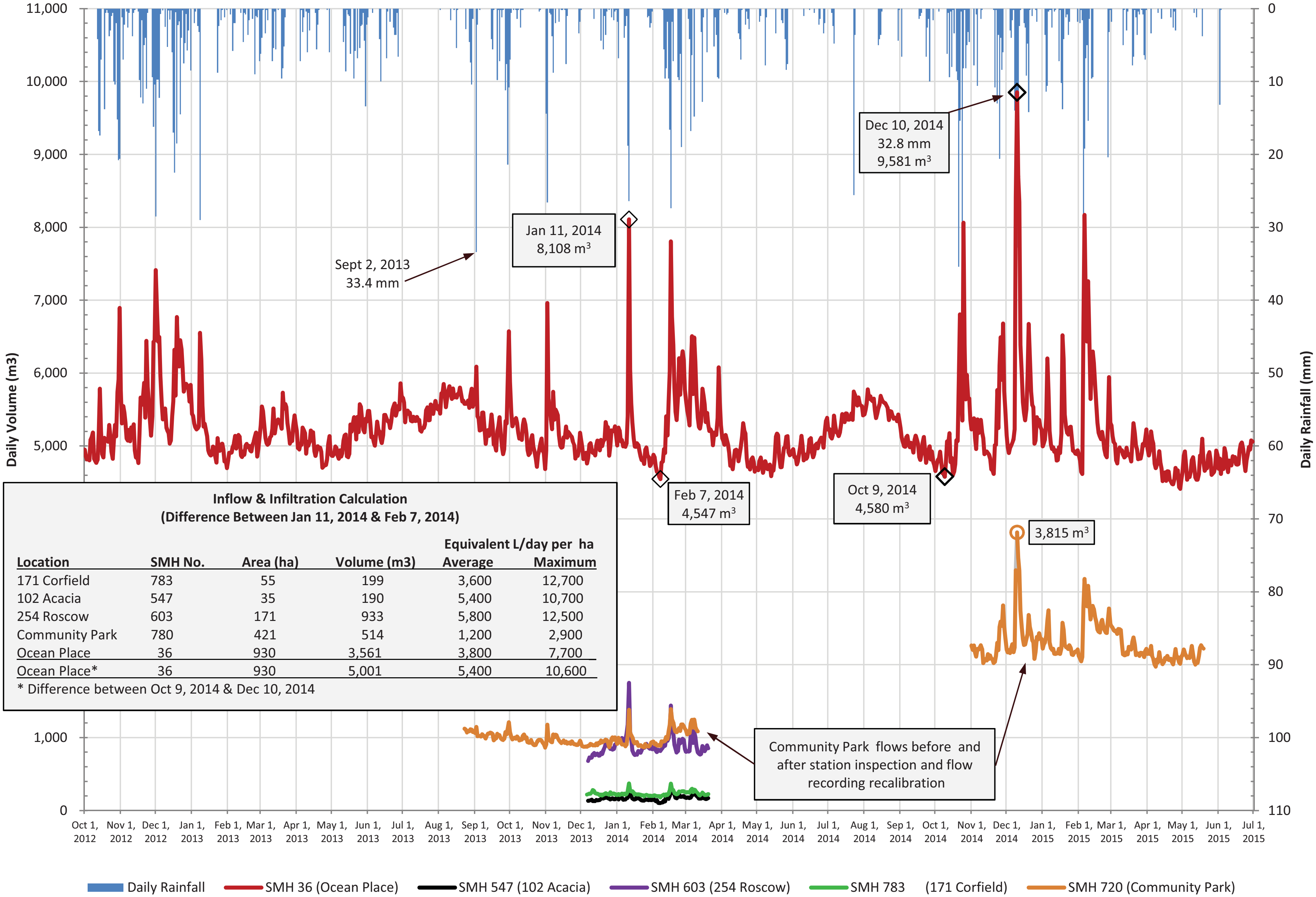
Flow Meters Daily Flow (m^3)
October 2012 - June 2015



Inflow & Infiltration Calculation
(Difference Between Jan 11, 2014 & Feb 7, 2014)

Location	SMH No.	Area (ha)	Volume (m3)	Equivalent L/day per ha Average	Maximum
171 Corfield	783	55	199	3,600	12,700
102 Acacia	547	35	190	5,400	10,700
254 Roscow	603	171	933	5,800	12,500
Community Park	780	421	514	1,200	2,900
Ocean Place	36	930	3,561	3,800	7,700
Ocean Place*	36	930	5,001	5,400	10,600

* Difference between Oct 9, 2014 & Dec 10, 2014

FIGURE 7

2, 2013 which plotted in excess of a 100 year event, they all coincided with observable increases in flow at each of the monitoring sites. It is, therefore, assumed that if a rainfall event with a higher intensity (return period) had occurred, excluding September 2, 2013 which was an excessively high intensity storm, it would result in a larger increase in flow. **Figure 8** presents the rainstorms plotted on the City's current IDF curves. While the December 10, 2014 rainstorm had less than a 2 year return period, it was preceded by a similar storm on December 9 and followed by another on December 11. A review of cumulative rainfall over a three day period for Environment Canada's Comox Airport weather station (ID No. 1021830), revealed the events frequency at once every 5 to 7 years for the cumulative rainfall over three days.

4.4.2 Inflow & Infiltration Estimates

The daily flow versus rainfall data was reviewed to assess the impact of rainfall on the rate and volume of flow at each site. Flow and rainfall data recorded in 5 minute increments was provided by SFE for four of the five sites. The City provided flow recording data in 5 minute increments for the flow recording station in the Community Park (SMH 720). The RDN provided flow data in 5 minute increments for the Ocean Place flow meter (SMH 36). A brief review of the flow monitoring results is presented below along with a summary of estimated average day and maximum instantaneous I&I rates.

<u>171 Corfield (SMH 783)</u>
A review of the daily volume data shows a noticeable increase in flow in response to heavier and ongoing rainfall followed by a quick decrease after heavier rainfall which is followed by a gradual decline in the days following stoppage of rainfall. This is indicative of both Inflow & Infiltration occurring in the system. A graph of daily volume vs rainfall for the 3 ½ month monitoring period is shown in **Appendix C, Figure 2.**

Appendix C, Figure 3 presents a 5 day comparison of daily flow from January 9 to 13, which includes the January 10 to 11 rainfall event, against the period of February 5 to 9, capturing the low flow day of February 7. The typical daily diurnal flow pattern of two peaks and two valleys each day can be seen even during the rainfall events. Large increases in flow spanning one to three recording increments (5 to 15 minutes) several times a day are evident. These spikes do not occur during the early morning hours between 2 am to 5 am. A review of the timing of the events each day vs the day of the week was carried out. The review indicated a general pattern for the weekday and for the weekend. Each day recorded four to five noticeable large increases in flow for a short period. In general, there were two to three spikes between 9 am and 12 noon and two to three spikes for the rest of the day. During weekdays, there was a spike generally between 3 pm and 4 pm, but not on the weekends. In the evenings, there was generally a spike between 6 and 10 pm during the weekday and on weekends. It is understood there is a privately operated sewage pump station at the top end of this catchment area servicing the 49 lot strata residential subdivision on Farrell Drive. This is suspected to be the source of the spikes.

The oscillating frequency of the readings is not unexpected for a relatively small catchment area, where flow fluctuations in response to a pump station(s) or individual activities (e.g., showering, laundry, meals preparation and dish washing) can be seen.

<u>102 Acacia (SMH 547)</u> - Dec 2013 to March 2014
As with the Corfield site, a noticeable increase in flow occurred in response to rainfall, but with more noticeable peaks and a slower decrease in flow after the event. This would suggest a higher rate of both Inflow & Infiltration compared to the Corfield catchment. A graph of daily volume vs rainfall for the 3 ½ month monitoring period is shown on **Appendix C, Figure 4**.

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Parkseville Proposed IDF Curves
with Year 2010, 2013, & 2014 Storms

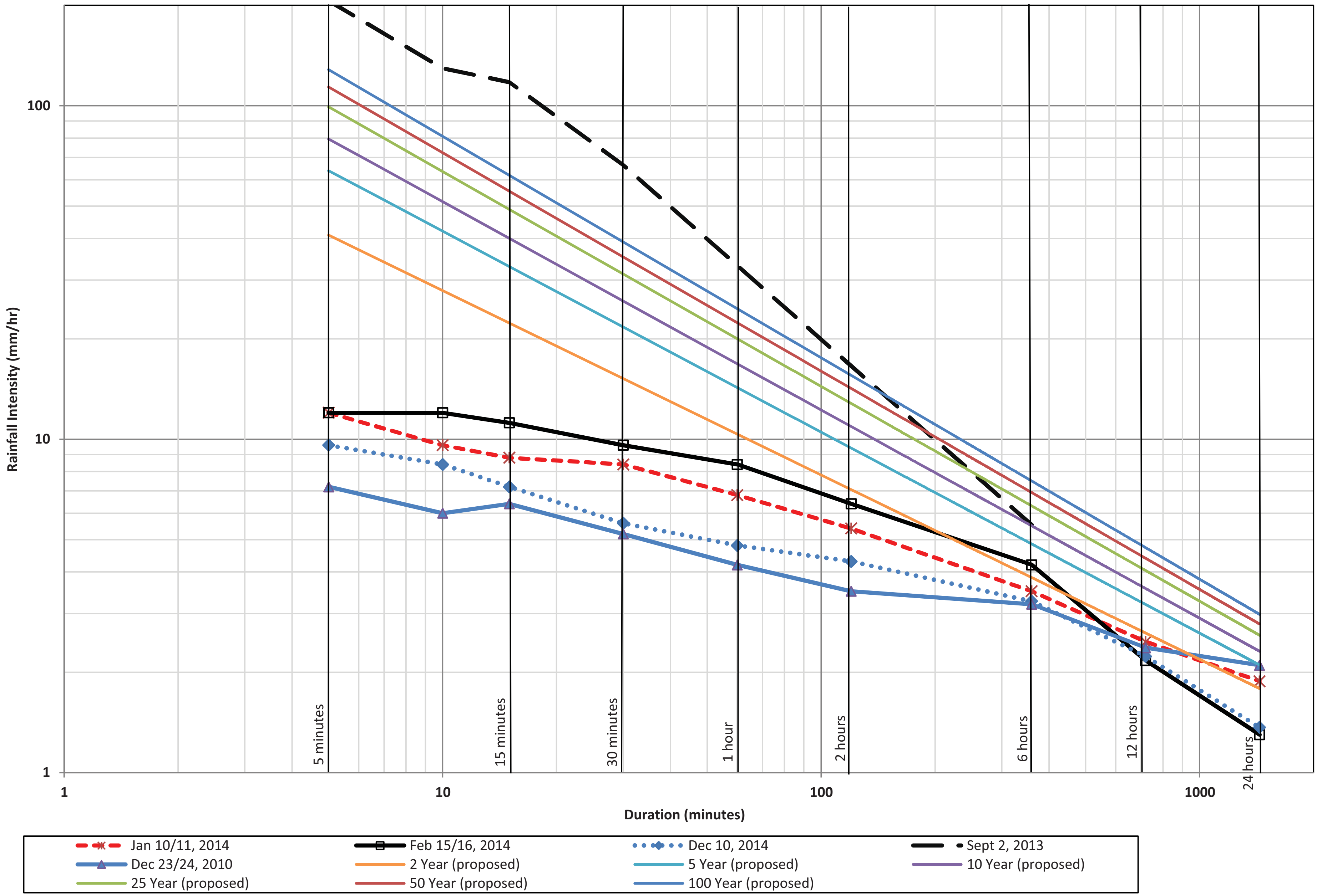

FIGURE 8

Appendix C, Figure 5 presents a 5 day comparison of daily flow from January 9 to 13, capturing the January 10 to 11 rainfall event, against the period of February 5 to 9 capturing the low flow day of February 7. The typical daily diurnal flow pattern of two peaks and two valleys each day occurs but is difficult to see when rainfall is occurring. The oscillating frequency of the readings is not unexpected for a relatively small and homogeneous (mainly single family residential) catchment area, where flow fluctuations in response to individual activities (e.g., showering, laundry, meals preparation and dish washing) can be seen. Flow readings at or below zero can be seen during the February 5 to 9 monitoring period. These are indicative of the very low flow levels experienced during the early morning hours when flow depths of less than 4 mm were recorded in the 250 mm diameter pipe.

631 Blenkin (SMH 560) - Sept 2014 to Dec 2014
The meter started recording flows on Sept 17 and the first data download was carried out on October 15. A review of the data suggested I&I did occur in response to a few typical small rainstorm events. During the second download, performed on Dec 8, City Staff noted the pipe diameter was incorrectly listed at 100 mm and not the actual 300 mm. The cause for the diameter change was not known. A review of the flow and level data showed accurate readings until Nov 2. Between Nov 2 and Nov 8, a noticeable jump in the flow depth and flow rate occurred, after which followed a large increase in the amplitude of the daily highs and lows. The data was determined to be corrupted and not usable from Nov 2 onward.

A consistent correlation between rainfall and increases in flow is not specifically identifiable as can be seen on the daily volume vs rainfall graph shown on **Appendix C, Figure 18** and in the graph of the daily flow vs rainfall as shown on **Appendix C, Figure 19**. Evidence of I&I is visible in **Appendix C, Figure 20**, which presents a 5 day comparison of daily flow between a dryer period of Oct 1 – 5 and the wetter period of Oct 22 – 26. Both durations span the same days of the week (Wednesday to Sunday). The typical daily diurnal flow pattern of two peaks and two valleys each day is evident. The lag in flow increase in response to rainfall suggests Infiltration is more predominant during this event.

254 Roscow (SMH 603) - Dec 2013 to March 2014
The Roscow catchment experienced noticeable increases in flow in response to heavier rainfall events, followed by a quick decrease after rainfall has stopped. This suggests Inflow is more predominant than Infiltration. A graph of daily volume vs rainfall for the 3 ½ month monitoring period is shown on **Appendix C, Figure 6**.

Appendix C, Figure 7 presents a 5 day comparison of daily flow from January 9 to 13, capturing the January 10 to 11 rainfall event, against the period of February 5 to 9 capturing the low flow day of February 7. The typical daily diurnal flow pattern of two peaks and two valleys each day is very evident, as is quick response to rainfall, suggesting Inflow is more predominant than Infiltration during this event.

Community Park (SMH 720) – June 2009 to Present
This station was not part of the flow monitoring program, but it is a continuous rainfall and flow recording site operated by the City of Parksville and is located downstream of the SMH 783 (171 Corfield St) monitoring site. The station has been recording rainfall and flow data in 5 minute increments since June 2009.

Appendix C, Figure 8 presents a 5 day comparison of daily flow from January 9 to 13, capturing the January 10 to 11 rainfall event, against the period of February 5 to 9 capturing the low flow day of February 7. The typical daily diurnal flow pattern of two peaks and two valleys is present, but somewhat difficult to distinguish. Regular spikes in flow are evident throughout the day, even during the rainfall event. This is indicative of a pump station(s) operating upstream of the monitoring site. The quick response to the onset and cessation of rainfall suggests Inflow is more predominant than Infiltration.

A comparison of the flows recorded at SMH 720 (Community Park) with those recorded at the station upstream, SMH 603 (254 Roscow) and downstream SMH 63 (Ocean Place) for the same time periods, suggests the Community Park flow meter is not recording correctly. The Community Park flows appear to be well below those expected for the contributing catchment area. For example, over the five day period of January 9 to 13, 2014, the Community Park total recorded volume was 5,400 m^3, which was less than the 6,000 m^3 recorded at the SMH 603 (254 Roscow) monitoring site located upstream, which has a contributing catchment area of 171 ha compared to Community Park's 416 ha. This was subsequently confirmed by SFE Global Ltd. and the monitoring equipment corrected in September 2014 as discussed in **4.5 Community Park Flow Meter Recalibration**.

Ocean Place (SMH 36) – July 2007 to Present
This is a continuous flow recording site operated by the RDN and is located within the RDN just beyond the northwest boundary of the City of Parksville, at the intersection of Ocean Place and Cavin Road. The station has been recording flow data in 5 minute increments since July 2007 but rainfall data was not included until August 2009.

Daily flows for the 7 years of record were plotted against rainfall to assess changes in flow in response to rainfall. As anticipated, flow increased during the wet fall/winter months, with noticeable spikes during heavy rainfall events. Six storm events were analyzed and assessed to estimate I&I rates. The storm events and the resulting calculated daily and maximum instantaneous I&I rates are presented in **Table 7**.

Table 7 - Ocean Place Maximum I&I Rates, 2007 - 2014

Dates (Wet Day – Dry Date)	Volume Difference (m^3)	Equivalent I&I (L/day per ha) *	
		24 Hour Average	Maximum Instantaneous (based on largest difference between flow rates)
Dec 3 – Oct 31, 2007	4,820	5,200	10,300
Nov 19 – Oct 22, 2009	4,445	4,800	9,000
Dec 24 – Oct 20, 2010	6,040	6,500	11,100
Sept 2 – Sept 1, 2013 **	515	550	11,000
Jan 10/11 & Feb 6/7, 2014	3,560	3,800	7,500
Dec 10 – Oct 9, 2014	5,000	5,400	10,600
City of Parksville Design Standard		**8,460**	**8,460**

Notes:
* Based on a catchment area of 930 ha.
** Rainstorm lasting 30 minutes (7 pm to 7:30 pm).

Appendix C, Figures 10, 11, 12 and **13,** graph flow and rainfall data in 5 minute increments for the first four storm events in **Table 7** (December 3, 2007; November 19, 2009; December 24, 2010; and September 2, 2013). The December 10, 2014 event is presented in **Figure 9**. Each graph shows how the flows responded to the rainfall pattern and when the estimated maximum instantaneous I&I rate occurred (based on flows recorded during the nearest low flow day). **Figure 10** presents the daily flow vs rainfall for the Ocean Place flow meter from 2007 to May 2015. The impact of rainfall is clearly evident and average and maximum instantaneous I&I rates for the large storms are noted.

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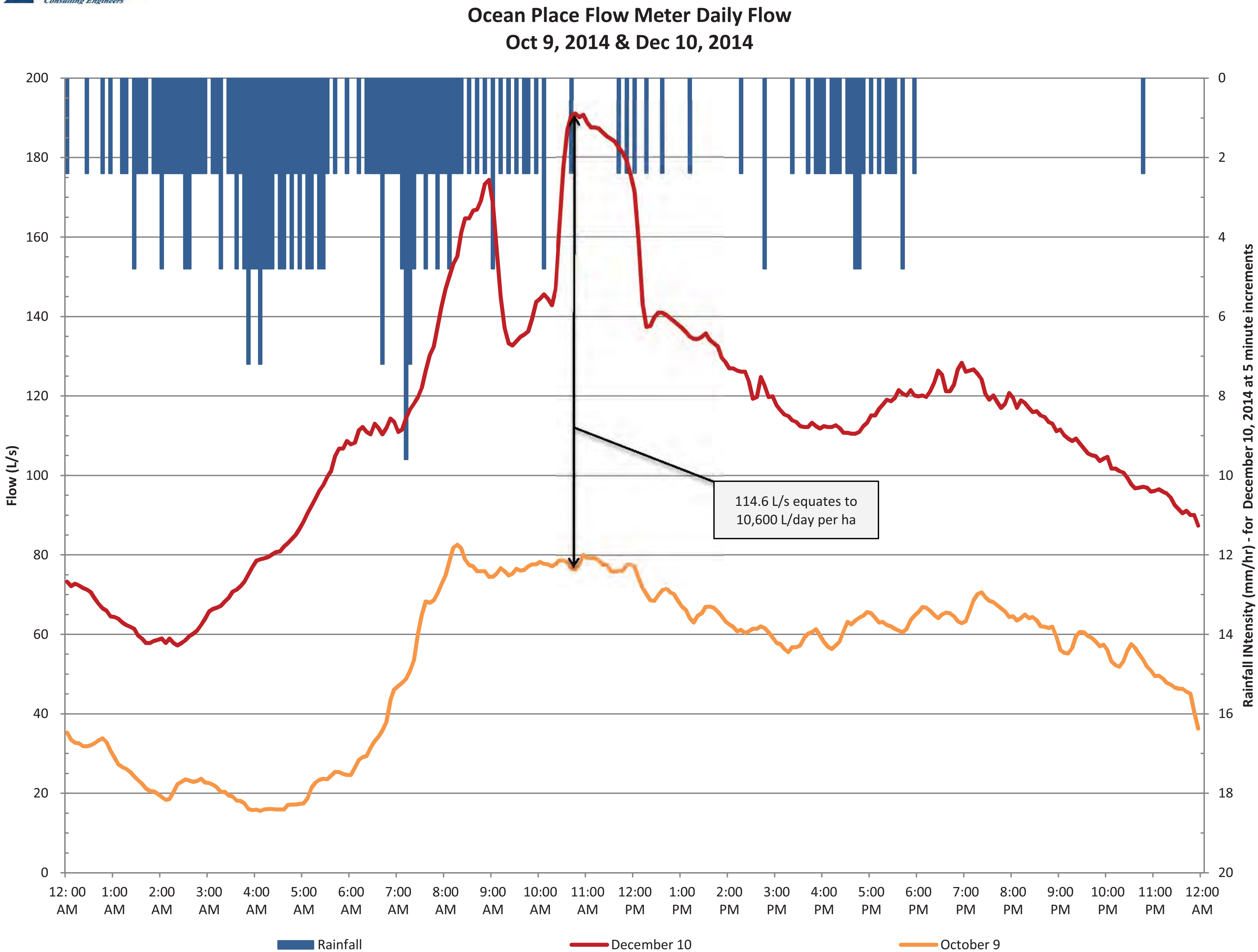

FIGURE 9

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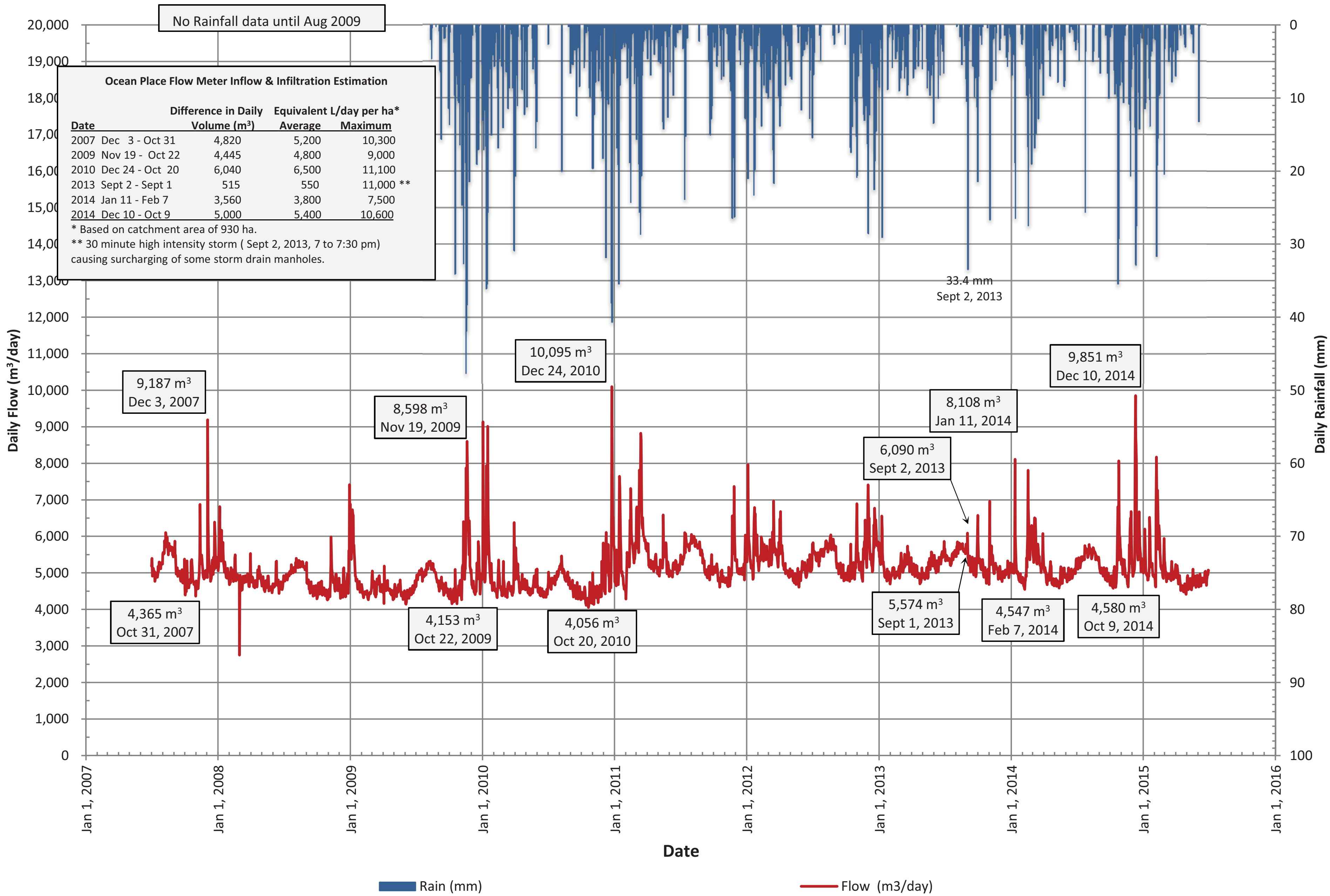
Ocean Place Daily Flow
January 2007 - June 2015



Ocean Place Flow Meter Inflow & Infiltration Estimation

Date	Difference in Daily Volume (m^3)	Equivalent L/day per ha* Average	Equivalent L/day per ha* Maximum
2007 Dec 3 - Oct 31	4,820	5,200	10,300
2009 Nov 19 - Oct 22	4,445	4,800	9,000
2010 Dec 24 - Oct 20	6,040	6,500	11,100
2013 Sept 2 - Sept 1	515	550	11,000 **
2014 Jan 11 - Feb 7	3,560	3,800	7,500
2014 Dec 10 - Oct 9	5,000	5,400	10,600

* Based on catchment area of 930 ha.
** 30 minute high intensity storm (Sept 2, 2013, 7 to 7:30 pm) causing surcharging of some storm drain manholes.

FIGURE 10

As noted previously, the short duration (30 minute) localized high intensity rainstorm event of September 2, 2013, resulted in localized roadway flooding and surcharging of several stormdrain manholes. While a total of 17.8 mm of rainfall was recorded at the Public Works Yard rain gauge, a much higher volume of 33.2 mm was recorded at the Community Park rain gauge during the same 30 minute interval. The rainfall intensities plot at over the 100 year return period for the 5 minute to 20 minute durations at the Public Works Yard. The Community Park's intensities plot much higher and do not drop below the 100 year return period until after 2 hours. A review of the storm event, revealed that while the estimated average daily I&I at Ocean Place was only 600 L/day per ha, the maximum instantaneous I&I was 11,300 L/day per ha, which was only slightly less than the 11,500 L/day per ha calculated for the December 24, 2010 event. **Appendix C, Figure 14** presents the daily flow and rainfall data in 5 minute increments for September 2, 2013 compared to the day before for the Ocean Place flow meter and for the Community Park flow meter (note the Community Park flow meter was not recording correctly, as previously discussed). The impact of this high intensity, short duration storm is clearly evident.

Table 8 presents the rate of I&I for each monitoring site calculated by the difference for the 24 hour period.

Table 8 – Monitoring Sites I&I Rates

Location	SMH No.	Catchment Area (ha)	Volume Difference (m^3)	Equivalent I&I (L/day per ha)	
				24 Hour Average	Maximum Instantaneous (based on largest difference between flow rates)
Temporary Monitoring Sites					
Oct 1 & Oct 22, 2014 (12 am to 12 am)					
631 Blenkin	560	72	147	2,050	4,200
Jan 10/11 & Feb 6/7, 2014 (9 pm to 9 pm)					
171 Corfield St	783	55	199	3,600	12,700
102 Acacia St	547	35	190	5,400	10,700
254 Roscow St	603	171	991	5,800	12,500
Permanent Monitoring Sites					
Jan 10/11 & Feb 6/7, 2014 (9 pm to 9 pm – Community Park, 12 am to 12 am – Ocean Place)					
Community Park	720	421	514	1,200 *	2,900 *
Ocean Place	36	930	3,561	3,800	7,500
City of Parksville Design Standard				**8,460**	**8,460**

Notes:

* Calculations are inaccurate. In September 2014, the Community Park flow meter was inspected, found to be recording incorrectly, and subsequently recalibrated by SFE Global Ltd. as discussed in **4.5 Community Park Flow Meter Recalibration**.

It is noted that the **Table 7** and **Table 8** calculated 24 hour average I&I rate at all sites and for all events is less than the 8,640 lpcd allocated in the City's Engineering Design Standards. However, for the calculated maximum instantaneous rate, the City's design standard was exceeded at all of the sites, excluding Community Park which was not recording correctly. The analysis indicates higher I&I rates should be incorporated into the City's standard and during computer modelling.

4.4.3 Peaking Factors

A review of peaking factors for each of the monitoring sites was carried out. While theoretical peaking factors are to be based on dry weather flow, i.e., when I&I is not occurring, this was not possible, as the flow monitoring data covered the 3 ½ month period of December 5, 2013 to March 21, 2014 for three of the temporary sites (171 Corfield, 102 Acacia, and 254 Roscow) and September 18 to December 8, 2014 for the 631 Blenkin St site. However, for three of the sites, periods of low flows were observed in the five days of February 5 – 9, 2014 in response to minimal rainfall during the preceding four weeks. At the Blenkin site, a low flow period at the beginning of October was noted. A detailed assessment of flows in 5 minute increments for February 7, 2014, revealed peaking factors ranging from 3.4 to 1.6 which, as expected, decreased as the catchment area increased. For Blenkin St, an assessment of October 2, 2014 revealed a lower than expected peaking factor of 1.4, based on its catchment size. A comparison of its average day flow with the other previously monitored sites found the Blenkin St average day flow to be appropriate.

A comparison of the peaking factors revealed that they decrease as the catchment areas increase, with the exception of Blenkin St catchment (SMH 560), as shown in **Table 9**. The peaking factors are not unusual and are in the range of expected values for the size of the catchment areas and the land-uses, with the exception of the Blenkin St being lower than expected. The reason for Blenkin St being lower could not be determined.

Table 9 – Monitoring Site Peaking Factors

Location	Catchment Area (ha)	Ave Flow (L/s)	Peak Flow (L/s & time)		Peaking Factor
October 2, 2014					
631 Blenkin St	72	3.6	5.1	8:35 am	1.4
February 7, 2014					
102 Acacia St	35	1.2	4.1	8:00 am	3.4
171 Corfield St	55	2.1	5.3 *	10:30 am	2.5 *
254 Roscow St	171	9.6	19.5	8:15 am	2.0
Community Park	421	10.8	18.2	11:30 am	1.7
Ocean Place	930	52.6	81.6	12:45 pm	1.6

Notes:
* The peak flow and peaking factor are exclusive of the intermittent "spikes" in the flow shown in **Appendix C, Figure 3** which is suspected to be from the operation of the private pump station servicing the 49 strata lot residential subdivision on Farrell Drive.

The daily flows on February 7, 2014 for each monitoring site are presented graphically in **Appendix C, Figures 3, 5, 7, 8** and **13**; respectively. The October 2, 2014 daily flow for Blenkin Street is presented graphically in **Appendix C, Figure 20**.

4.5 Community Park Flow Meter Recalibration

As noted previously in **4.4.2 Inflow & Infiltration Estimates** under Community Park (SMH 720) – June 2009 to Present, flows recorded at the City's permanent monitoring site in the Community Park were much lower than expected. An inspection of the station in the summer of 2014 by City Staff confirmed the station was under-recording by approximately 50%. The station was subsequently recalibrated in September 2014 by SFE Global Ltd. A copy of their inspection report is presented in **Appendix D**.

An attempt by SFE Global Staff to apply a correction factor to the flow data recorded prior to September 2014 was not successful.

5 DESIGN CRITERIA

A detailed review of the City's current design criteria comparing it against several other municipalities (on Vancouver Island and in the Lower Mainland), as well as with the findings of the flow monitoring program as discussed in section **4 Flow Monitoring**. The review is presented in **Appendix E - Technical Memorandum No. SS-2 (Sanitary Sewer Design Criteria)**. Below is a summary of the design criteria used for the analysis of the Existing Conditions computer model and the OCP Build-Out computer model.

5.1 Existing Conditions

While the findings of the flow monitoring program indicated the per capita flow and peaking factor were less than the City's current design standards, the current design standards were selected for modelling, in part because they are the current standards and they were found to be conservative. The design standards used are as summarized in **Table 10**.

Table 10 – Existing Design Criteria

Item	Unit	Quantity
Daily Flow		
- Residential	Litres/day per capita	410
- Institutional	Litres/day per capita	410
- Commercial	Litres/day per ha	22,500
- Hotel	Litres/day per patron	n/a*
- Motel	Litres/day per patron	n/a*
- Industrial	Litres/day per ha	22,500
- Infiltration	Litres/day per ha	8,640
Peaking Factor		
For Population of:		
< 1,000	(multiplier)	5
> 1,000 and < 3,000	(multiplier)	4
Commercial & Industrial	(multiplier)	(Ave Day ÷ 410)**
Max Depth (gravity mains)		none
Velocity		
Gravity Mains		
- Minimum	m/s	0.6
- Maximum	none	
Force Mains		
- Minimum	m/s	0.9
- Maximum	m/s	3.5 (should not exceed)
Pipe Friction Factor		
Gravity Flow (Manning's, N)		
- concrete pipe		0.013
- PVC pipe		0.011
Pressure Flow (Hazen-Williams, C)		
- all pipe		120
Minimum Pipe Diameter	mm	200+
+ can be reduced to 150 mm for last upstream section that cannot be extended in the future.		

Notes:

* The City's current design standards do not include a unit flow for hotels or motels.

** Peaking factor for Commercial and Industrial lands to be based on the equivalent population calculated by dividing the daily flow by 410 L/day per capita.

5.2 OCP Build-Out

Based on the findings of the flow monitoring program, a lower per capita daily flow for residential and institutional development, a higher I&I allowance, and the peaking factor formula from the Master Municipal Contract Document Design Manual were used. In addition, the capacity (maximum depth) of gravity sewer mains was assessed using the City of Nanaimo's design standard of 70% full for all pipe diameters. The OCP Build-Out design parameters are presented in **Table 11**. Changes from the current standard are in bold.

Table 11 – Proposed Design Criteria

Item	Unit	Quantity
Daily Flow		
- Residential	Litres/day per capita	**300**
- Institutional	Litres/day per capita	**300**
- Commercial	Litres/day per ha	22,500
- Hotel	Litres/day per patron	**300**
- Motel	Litres/day per patron	**500**
- Industrial	Litres/day per ha	22,500
- Infiltration	Litres/day per ha	**12,500**
Peaking Factor		**$6.75P^{-0.11}$**
Max Depth (gravity mains)		**70% of diameter**
Velocity		
Gravity Mains		
- Minimum	m/s	0.6
- Maximum	none	
Force Mains		
- Minimum	m/s	0.9
- Maximum	m/s	3.5 (should not exceed)
Pipe Friction Factor		
Gravity Flow (Manning's, N)		
- concrete pipe		0.013
- PVC pipe		0.011
Pressure Flow (Hazen-Williams, C)		
- all pipe		120
Minimum Pipe Diameter	mm	200^{+}

$^{+}$ can be reduced to 150 mm for last upstream section that cannot be extended in the future.

6 COMPUTER MODEL

6.1 Computer Software Evaluation & Selection

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

The City had two computer models of the sanitary sewer system; a model developed by the City using the computer software program Hydra, and the model developed by Koers using the computer software program Sansys as part of the 1996 Sanitary Sewer 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 sanitary sewer system and the storm drainage systems.

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

6.2 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 sanitary sewer program is used for:

- Development of sewer master plans
- Inflow & infiltration studies
- Wet weather flows scenarios
- Pumping and pressure sewers
- Prediction of overflows

Sanitary sewer flows can be loaded globally or locally with different allowances for both dry and wet weather flows. Flows may be varied using hourly and daily temporal variation factors. Wet weather (I&I) flows can be incorporated into the model both globally or to specific manholes as constant (base) flows, simulated rainfall, simulated groundwater mounting, unit hydrographs or user defined hydrographs.

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 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 G - XPSWMM Technical Literature**.

7 MODEL DATA ENTRY & CALIBRATION

Three unique computer models were developed:

- Existing Conditions – Calibrated
- Existing Conditions – Current Design Standards
- Future Conditions – OCP Build-out with Proposed Design Standards.

A calibrated model was the first to be developed and served as the basis to create the other two models. A discussion of how the models were developed is presented below.

7.1 Data Collection & Entry

The computer model of the sanitary sewer system was developed as follows:

1. The City's sanitary sewer 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.

2. Pump station information was entered manually from record drawing information.

3. The City's cadastral and current zoning information was imported from their GIS database.

4. 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.

5. The current zoning designation for each lot was imported from the City's GIS database.

6. Catchment area boundaries for each pipe were created digitally within the model based on how each property is serviced and on the zoning of contributing properties.

7. Catchment areas of undeveloped properties, that will require installation of new mains and how they will connect to the City's sewer system, was established utilizing digital ground contour information from the City (LIDAR maps) and proposed development plans, if available.

8. Residential population density per land-use category was entered manually.

9. Design flows based on land-use, I&I and peaking factors (see **Table 10 and 11**) were entered manually.

10. 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 pipe. Field observations and record drawings were consulted and the model corrected.

7.2 Calibration Model

The computer model for existing conditions was calibrated utilizing the following information:

- A compiled pipe-node network of the City's and RDN current sewage system,
- land-use in accordance with the City's current zoning plan,
- flow monitoring data from the five flow monitoring stations,
- BCStats 2013 population estimate for the City of Parksville, and
- Estimate of the service population of contributing areas in the RDN.

The process undertaken was as follows:

Step 1 Service Population Calibration

.1 Assigned population densities to each existing residential land-use to the model flow data table. All other land uses (commercial, industrial, and institutional) assigned population densities of zero.

.2 Ran computer model and compared calculated model population (sum of density x area) with BCStats population estimate. Note that the initial calculated population was slightly higher (18%) that BCStats estimate.

.3 Reduced population density globally, re-ran model, compared calculated vs estimated population. Repeated procedure until calculated population approximated estimated population (12,000).

Step 2 Per Capita Flow Calibration

.4 Applied City's current residential per capita flow (410 L/day) and peaking factor (4) to model residential flow data table. All other land uses (commercial, industrial, and institutional) assigned flows of zero.

.5 Ran computer model, generated sewage flow hydrograph at SMH 547 (102 Acacia Street); a fully developed catchment with only one land-use (residential), and compared with flow monitoring hydrograph at the same location. The calculated flows were found to be significantly higher.

.6 Developed a diurnal (daily) flow pattern based on the flow monitor hydrograph recorded during drier periods when I&I was estimated to be minimal, e.g., Feb 6/7, 2014, resulting in a calculated peaking factor of 2.5 and a per capita flow of 300 L per day.

.7 Model re-run with a residential per capita flow of 300 L/day, a peaking factor of 2.5 and the developed diurnal flow pattern. The hydrograph generated at SMH 547 (102 Acacia St) was compared with the flow monitor hydrograph and a good fit was confirmed as shown in **Figure 11**.

Step 3 Non-Residential Flows

.8 Unit flows for commercial, industrial and institutional land-uses were added to the model using the City's current design flows of:

- 22,500 L per day per ha for Commercial zoned land,
- 22,500 L per day per ha for Industrial zoned land,
- 410 L per day per capita for Institutional zoned lands, and
- 8,640 L per day per ha for I&I.

.9 For hotels and motels, the City's design standards do not have a designated unit flow per unit/room or patron. The Master Municipal Contract Document design manual values of 300 and 500 L per day per patron for hotel and motel;

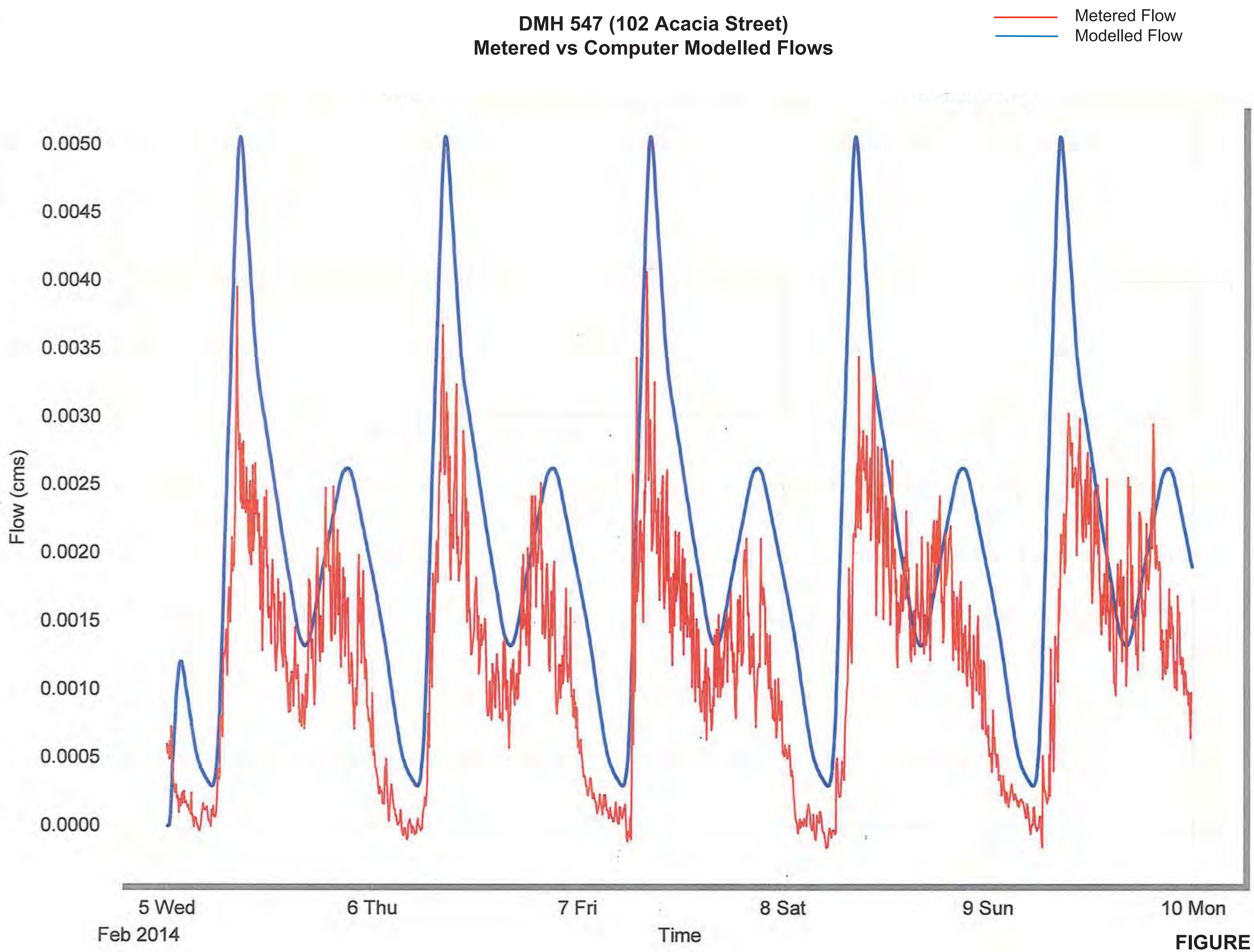

DMH 547 (102 Acacia Street)
Metered vs Computer Modelled Flows
Metered Flow
Modelled Flow
Flow (cms)
0.0050
0.0045
0.0040
0.0035
0.0030
0.0025
0.0020
0.0015
0.0010
0.0005
0.0000
5 Wed
Feb 2014
6 Thu
7 Fri
8 Sat
9 Sun
10 Mon
Time

FIGURE 11

respectively were applied to the estimated 8,000 tourists that are reported to visit the City during the peak of the summer vacation season.

.10 Flows from commercial and industrial land-uses were modelled to occur between 9 am to 5 pm, to reflect typical working hours.

.11 The model was run, hydrographs generated at each of the five flow monitoring sites and compared with the recorded (flow monitoring) hydrographs. The calculated hydrographs were notably higher than the recorded hydrographs at each location.

7.3 Existing Conditions Model

Using the calibrated computer model, the City's current design criteria for the various sewage flows per land-use and for peaking factor were applied to create dry weather flows. A copy of the City's current land-use plan is presented in **Figure 12**.

For the seasonal population (tourists), a design flow of 300 L per day per tourist for hotels and campsites and 500 L per day per tourist for motels was applied to 1,790 tourism accommodation units within the City with an assumed occupancy of 3 tourists per unit. This accounts for just under 5,400 of the estimated 8,000 tourists. An additional 600 tourists were added to the model to account for the estimated 200 accommodation units in the general area but not connected to the City's sewer system people (see discussion in **3.1 Historic**). The remaining allowance of 2,000 tourists (8,000 – 5,400 – 600) were distributed evenly throughout residential areas to allow for bed & breakfast accommodation, out of town visitors staying with friends, and visits during the day by out of town visitors.

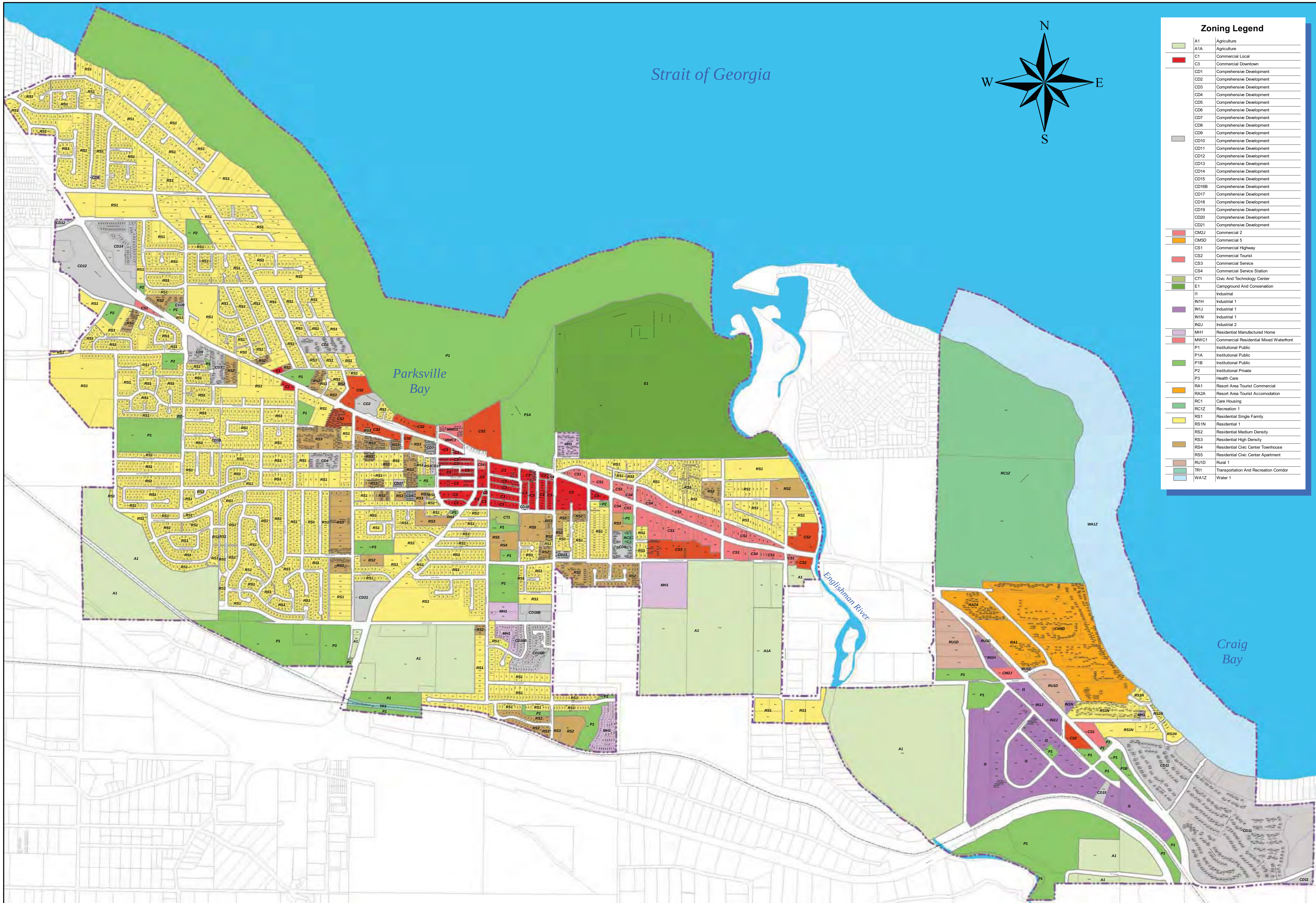
An allowance for I&I was applied in accordance with the City's design standards. While the application of I&I during the peak tourist season (summer), may seem inappropriate, the flow data from the Sept 1, 2013 thunderstorm event indicates high infiltration rates can occur at any time of the year (see discussion in **4.4.2 Inflow/Infiltration Estimates** and **Table 7**).

7.4 OCP Build-Out Model

The OCP Build-Out model was developed from the Existing Conditions model. The design criteria for the various land-uses and peaking factor was changed from the City's current standards, to the proposed standards as presented in **Table 11**. Undeveloped lands identified for future development were assigned the designated land-uses. Adjustments to areas to account for redevelopment/rezoning were also made. The areas where residential growth is anticipated are shown in **Figure 13**. A copy of the OCP is presented in **Figure 14**.

The computer model was run with all densities turned off except for residential. The calculated population was compared with the City's planning department predicted OCP build-out population of just fewer than 22,000. As anticipated, the calculated population was much higher. This was not unexpected, as planning Staff have indicated that the permitted maximum number of units per ha in the multi-family zonings is generally not constructed as market forces (home buyers) do not want/support this way of living. The densities were, therefore, adjusted downwards until the projected population matched that of the OCP at Build-Out within the available undeveloped lands.

An additional 1,175 tourist resort accommodation units were added to the model in the two areas discussed in **3.2 Projected**, resulting in a total tourism population of approximately 11,200 as listed in **Table 4**.



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 12

DISCLAIMER

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

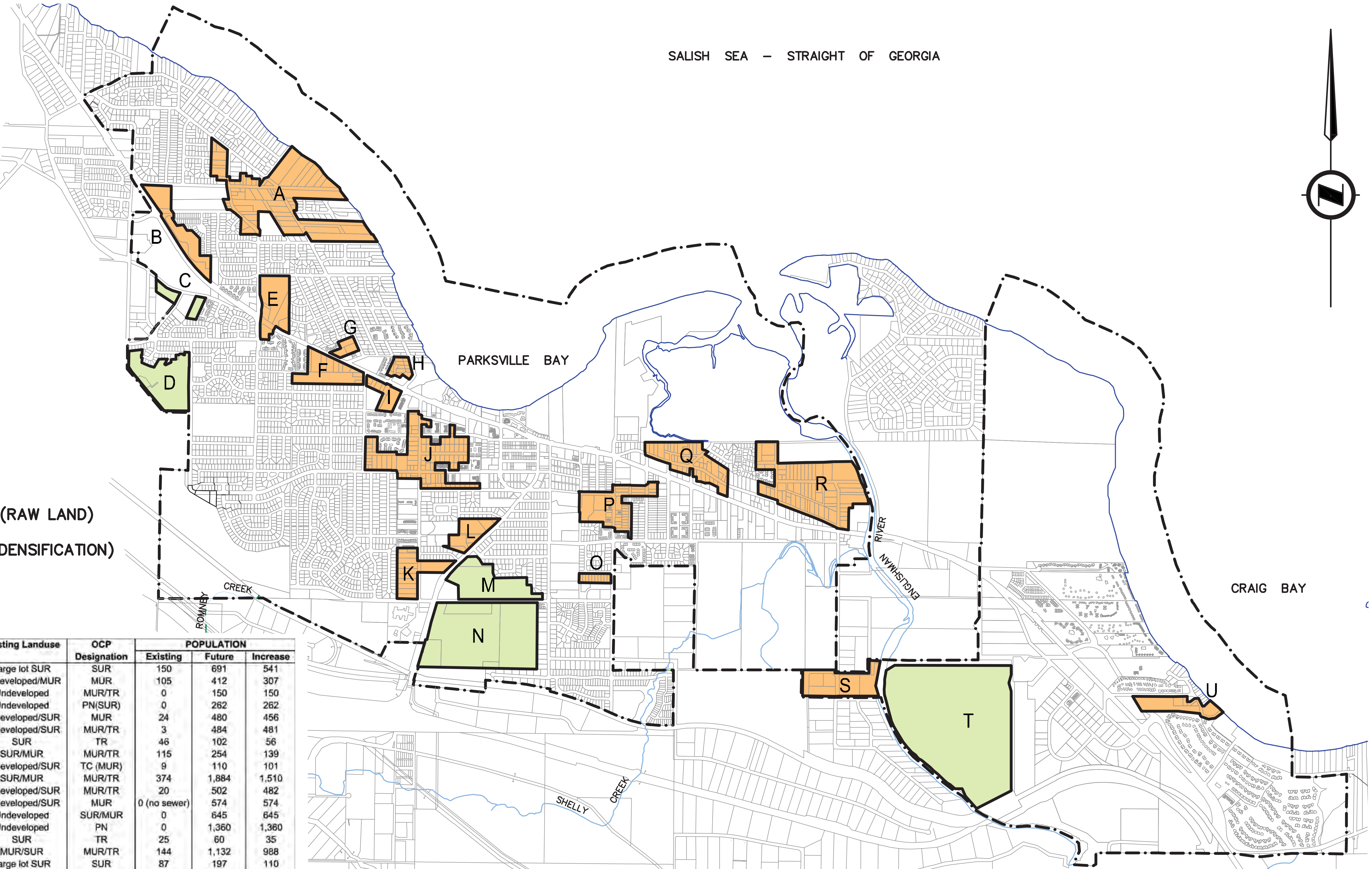
Project Location: I:\USERS\GIS\Map Projects\Zoning.mxd

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



NEW DEVELOPMENTS (RAW LAND)

RE–DEVELOPMENTS (DENSIFICATION)

CITY BOUNDARY

Population Growth Area	Location	Area (Ha)	Existing Landuse	OCP Designation	POPULATION Existing	Future	Increase
A	Pym/Temple/Sanderson	28.9	Large lot SUR	SUR	150	691	541
B	Sanderson/Stanhope	10.3	Undeveloped/MUR	MUR	105	412	307
C	Wembly/Church	1.8	Undeveloped	MUR/TR	0	150	150
D	Cedar Ridge	9.3	Undeveloped	PN(SUR)	0	262	262
E	Hwy19a/Pym/Doehle/Tranfield	6.3	Undeveloped/SUR	MUR	24	480	456
F	Blenkin/Hwy19a	6.6	Undeveloped/SUR	MUR/TR	3	484	481
G	Temple at Bay	2.8	SUR	TR	46	102	56
H	Roscow/Dogwood	2.5	SUR/MUR	MUR/TR	115	254	139
I	Hyw19a at Finholm	3.8	Undeveloped/SUR	TC (MUR)	9	110	101
J	Morrison/Banks/Hirst/Molliet	15.4	SUR/MUR	MUR/TR	374	1,884	1,510
K	Moillet at Despard	5.3	Undeveloped/SUR	MUR/TR	20	502	482
L	Alberni Hyy at Marks St	3.7	Undeveloped/SUR	MUR	0 (no sewer)	574	574
M	Despard/Alberni Hwy/McKillop	9	Undeveloped	SUR/MUR	0	645	645
N	Despard/Alberni Hwy/Craig	27.7	Undeveloped	PN	0	1,360	1,360
O	Skylark Ave	1.2	SUR	TR	25	60	35
P	Corfield/Evergreen/Bagshaw	10.3	MUR/SUR	MUR/TR	144	1,132	988
Q	Pioneer Cres/Bagshaw/Mill	8.8	Large lot SUR	SUR	87	197	110
R	Shelly/Turner/Martindale/Pioneer	21.2	Large lot SUR	TR	159	1,429	1,270
S	Despard at Martindale	8.2	Large lot SUR	SUR	0 (no sewer)	198	198
T	Despard / Tuan east of River	55.3	Undeveloped	PN	0	1,327	1,327
U	Arbust Road	5.5	Undeveloped/SUR	TR	17	340	323
		243.9				Total	11,315

City of Parksville

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Consulting Engineers

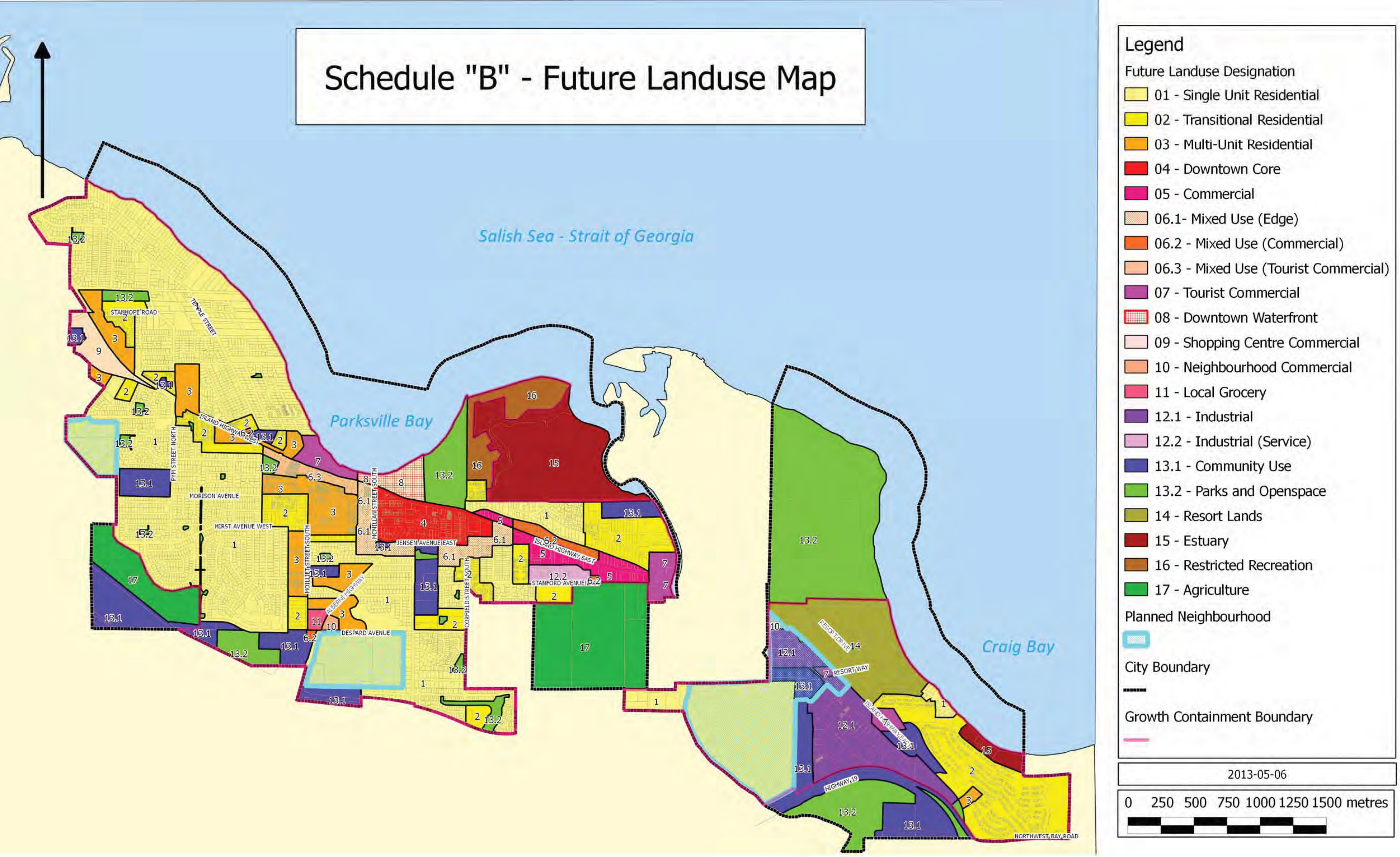
CLIENT: CITY OF PARKSVILLE

PROJECT: SANITARY MASTER PLAN

TITLE: AREAS OF GROWTH

APPROVED		SCALE	1:25,000
DATE	FEB 2015	DWG No.	FIGURE 13
PROJECT No.	1346		

File: H:\6012 Parksville City\1346 Parksville Sanitary & Storm Drainage Master Plan\03 Drawings\Sanitary\1346san Figures.dwg Plot Time: Oct 07, 2015 – 10:45am User: rcave



Schedule "B" - Future Landuse Map
Salish Sea - Strait of Georgia
Parksville Bay
Craig Bay
Legend
Future Landuse Designation
01 - Single Unit Residential
02 - Transitional Residential
03 - Multi-Unit Residential
04 - Downtown Core
05 - Commercial
06.1- Mixed Use (Edge)
06.2 - Mixed Use (Commercial)
06.3 - Mixed Use (Tourist Commercial)
07 - Tourist Commercial
08 - Downtown Waterfront
09 - Shopping Centre Commercial
10 - Neighbourhood Commercial
11 - Local Grocery
12.1 - Industrial
12.2 - Industrial (Service)
13.1 - Community Use
13.2 - Parks and Openspace
14 - Resort Lands
15 - Estuary
16 - Restricted Recreation
17 - Agriculture
Planned Neighbourhood
City Boundary
Growth Containment Boundary
2013-05-06
0 250 500 750 1000 1250 1500 metres
STANHOPE ROAD
TEMPLE STREET
ISLAND HIGHWAY WEST
PYM STREET NORTH
MORISON AVENUE
HIRST AVENUE WEST
MOILLIET STREET SOUTH
MCMILLAN STREET SOUTH
JENSEN AVENUE EAST
ISLAND HIGHWAY EAST
CORFIELD STREET SOUTH
STANFORD AVENUE
ALBERNI HIGHWAY
DESPARD AVENUE
RESORT DRIVE
RESORT WAY
HIGHWAY 19
NORTHWEST BAY ROAD

FIGURE 14

An allowance for I&I was applied through-out the model at a recommended design rate of 12,500 L per day per ha as discussed in **5 Design Criteria**.

7.4.1 Municipal Boundary Expansions

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

7.4.2 RDN Lands

From discussions with RDN Staff and a review of the Area E, Area F, and Area G community sewer service planning areas maps, the OCP Build-out condition model does not include any allowance for:

- increasing flows from lands within the RDN that are serviced by the regional sewer system and flow into the City of Parksville (i.e. no increases in density), or
- servicing lands beyond the current service area (i.e., no extension of the collection system).

Lands within the RDN that are presently serviced and discharge into the City's sewage collection system are located east of the City, in the Craig Bay area and drain to the Craig Bay pump station.

Lands within the RDN to the west of the City, in the Wembley Road area that drain to Ocean Place flow meter, convey flow through the City in RDN owned mains.

8 MODELLING RESULTS

8.1 Recorded Flows & Design Flows

The computer model was used to assess the ability of the sanitary sewer collection system to convey the peak flow under existing conditions and at OCP Build-Out. The peak flow consists of the following components:

Qpeak = [Res + STourist]*PF + ICI*3 + I&I

Where:

Qpeak = Peak Flow
(highest flow expected over 24 hours)

Res = Permanent Residential Population Flow
(average day flow of 300 lpcd for:
12,000 people, current conditions, and
22,000 people, OCP Build-Out)

STourist = Seasonal Tourism
(average day flow for:
8,000 people, Current Conditions, and
11,200 people, OCP Build-Out)

PF = Peaking Factor
(Factor used to convert average day flow to peak day flow based on total equivalent service population. Values of 2.27 were used for Existing Conditions and 2.15 for OCP Build-Out based on the equation $6.75P^{-0.11}$)

ICI = Institutional, Commercial & Industrial
(average day flow allocated to be between 9 am to 5 pm, resulting in the peaking factor of 3 which is included in the equation)

I&I = Peak Inflow/Infiltration
(12,500 L/day per ha for:
930 ha, Current Conditions, and
1,035 ha, OCP Build-Out)

The peak flow from this equation is considered to be a conservative approach to assessing the performance of the collection system for two reasons:

1. The unlikelihood of the peak I&I occurring on the same day as the peak tourism population which it is estimated presently reaches its height twice a year (the July and the August long weekends), and

2. The unlikelihood of the peak I&I occurring at the same time of the peak sewage flow which generally occurs between 8 am and 1 pm.

As winter rainstorm events are more common, as previously discussed in **4.4.2 Inflow/Infiltration Estimates** under Ocean Place (SMH 36) and as shown in **Table 7** and on **Figure 10**, it appears to be more likely for the peak I&I flow to occur during the winter rather than summer months. In addition, during the winter months the service population sewage flow would be lower as the seasonal tourism population is lower compared to the summer months. Therefore, the computer model results are considered to be an assessment of the ability of the City's sewer system to operate during a rare maximum loading condition.

For comparative purposes, **Table 12** presents the highest recorded summer flow in 2013 and in 2014, and the two highest recorded winter flows during the past 8 years along with the summer and winter flows for Existing Conditions and OCP Build-Out at the Ocean Place flow meter based on the proposed design flows listed in **Table 10** and **Table 11**; respectively.

Table 12 – Recorded and Design Flows at Ocean Place Flow Meter

Description	Land-Use Flow			Inflow/Infiltration (L/s)		Cumulative Flow	
	Average Day						
	Residential (L/s)	ICI (L/s)	Tourism (L/s)	Average Day	Instantaneous Peak	Average Day (L/s)	Daily Peak (L/s)
Recorded Flows							
Summer							
(Sept 2, 2013 *)	65 (total for all land-uses)			6	119	71	191
(Aug 4, 2014 **)	< 67 (total for all land-uses)			> 0	> 0	67	116
Winter							
(Dec 24, 2010 ***)	47 (total for all land-uses)			70	120	117	187
(Dec 10, 2014 ****)	53 (total for all land-uses)			61	114	114	191
Existing Conditions, Current Design Flows (Table 10)							
Summer	57	9	-	-	-	66	156
Winter	57	9	-	93	93	159	249
Existing Conditions, Proposed Design Flows (Table 11)							
Summer	42	9	26	-	-	77	181
Winter	42	9	7 #	135	135	193	273
OCP Build-Out, Proposed Design Flows (Table 11)							
Summer	76	9.33	39	-	-	124.33	275
Winter	76	9.33	10 #	150	150	245.33	363

Notes:

* The September 2, 2013 rainstorm was a localized high intensity short duration event. A total of 33.2 mm of rainfall was recorded during the storm's 30 minute duration (7:00 pm to 7:30 pm). Almost 90% (29.4 mm) fell within 15 minutes. This high intensity of rainfall was well above a 100 year return period for up to a two hour duration storm as shown on **Figure 8**.

The resulting increase in flow at the Ocean Place flow meter was calculated at 515 m^3 and the majority passed by during a 2 hour period (7:30 pm to 9:30 pm). At its peak, the flow increased by 119 L/s as shown on **Appendix C – Figure 14**.

** Highest recorded flows for the summer of 2014 (5,777 m^3). The highest flows were recorded in the latter half of the morning. Flows greater than 110 L/s started just after 10 am and end just before noon. The peak flow (116 L/s) started just before 10:30 am and ended just after 10:40 am.

*** The December 24, 2010 rainstorm event resulted in the highest flow at the Ocean Place flow meter (10,095 m^3) during the past 8 years. At the French Creek Pollution Control Centre the total flow was 18,874 m^3, the second highest during the past 8 years. The highest flow recorded at the FCPCC during the past

8 years was only slightly higher at 18,983 m^3 and was recorded on December 10, 2014.

A total of 78.6 mm of rain was recorded over two days (38 mm + 40. 6 mm, Dec 23 and Dec 24; respectively). While the individual daily rainfall totals had less than a 2 year return period, a review of the rainfall data for the Environment Canada Comox Airport weather station, revealed this storm event to have the fourth highest two day total for the 62 years of data (1953 – 2014) for a recurrence interval of once every 15 – 16 years. A similar analysis of the City of Parksville rainfall stations was not possible as there is only 10 years of data (2005 to present) as noted previously in **4.4.1 Rainfall Events**.

The 70 L/s average day I&I flow is based on the difference between the total volume over 24 hours for the rain event day (Dec 24) compared to the closest low flow day (Oct 20). The 47 L/s average day flow for all land-uses is based on the difference between the storm event total 24 hour flow minus the calculated volumes of I&I as discussed in **Appendix C**. The peak I&I flow is based on the maximum difference in instantaneous flow between the two dates and is shown on **Appendix C – Figure 12**.

**** The December 10, 2014 rainstorm had less than a 2 year return period (see **Figure 8**) but was preceded by a similar storm on December 9 and followed by a storm on December 11 (see discussion under **4.4.1 Rainfall Events**). A review of cumulative rainfall over a three day period for Environment Canada's Comox Airport weather station (ID No. 1021830), revealed an occurrence frequency of once every 5 to 7 years. The 61 L/s average day I&I flow is based on the difference between the total volume over 24 hours for December 10 compared to the closest low flow day (October 9) as discussed in **4.4.2 Inflow & Infiltration Estimates** under the heading <u>Ocean Place SMH 36</u>. At its peak, the difference in flow was more than 114 L/s as shown on **Figure 9**.

For winter season tourism, occupancy rate data was not available. For comparative purposes, peak flow based on 25% occupancy is shown.

8.2 Existing Conditions

As noted in section **8.1 Historical & Design Flows**, the design flows modelled are a conservative approach to the system analysis as they are based on full occupancy of the City's tourism accommodation coinciding with a peak I&I allowance. Peak tourism occupancy occurs around the long weekends in the beginning of July, August and September.

8.2.1 Gravity Sewers

There are gravity sewer trunk mains within the City that are owned and operated by the RDN and some of these are shown by the modelling to be operating above design capacity. A brief discussion of the modelling results on City mains as well as the RDN owned mains is presented below.

<u>City Owned Mains</u>

Based on the sewer collection model, created using the information from the City's GIS database and information from record drawings, the computer analysis shows no capacity issues for the City's current design standards (**Table 10**) or the proposed design standards (**Table 11**).

Known Problem Mains

Public Works Staff provided a list of a dozen problem pipes and two manholes. The documented problems can be categorized as being either due to poor installation (settlement along the pipe or at the manhole) or age (broken pipe, numerous repairs, pipe wall deteriorating, root infiltration).

RDN Owned Mains

For the RDN owned trunk mains, three sections of the Temple Street trunk main were calculated to flow at 100% of capacity and two sections of the Parksville Bay trunk main were calculated to flow at above 90% of capacity during peak flow. The RDN's Bylaw No, 500, 1987, 'Schedule 4D' Community Sewer System Standards allow for pipes 500 mm diameter and larger to flow just full (100%). The majority of both trunk mains were calculated to flow above the proposed City of Parksville design standard of 70% full. A brief discussion of each trunk main is presented below.

Parksville Bay Trunk Main (Corfield to Bay Ave Pump Station)

This 1,360 m long 600 mm diameter reinforced concrete trunk main has a calculated just full capacity ranging from a low at 190 L/s (at its start by the curling rink where the pipe slope is reported to be 0.1%) to a high of 330 L/s (as its discharges into the Bay Ave pump station where the pipe slope is reported to be 0.29%).

The computer modelling results show the majority of the trunk main (1,181 m, or 9 of the 12 sections) flows more than 70% full when conveying the daily peak flow during a heavy rainfall event and a total of 278 m (2 sections) of pipe is shown to flow at more than 90% full. The impact of the I&I allowance (61 L/s) can be seen, for when it is removed, the trunk main is shown to flow at less than 50% full when conveying the daily peak flow.

Temple Street Trunk Main (Doehle Ave to 75 m west of Ocean Place flow meter),

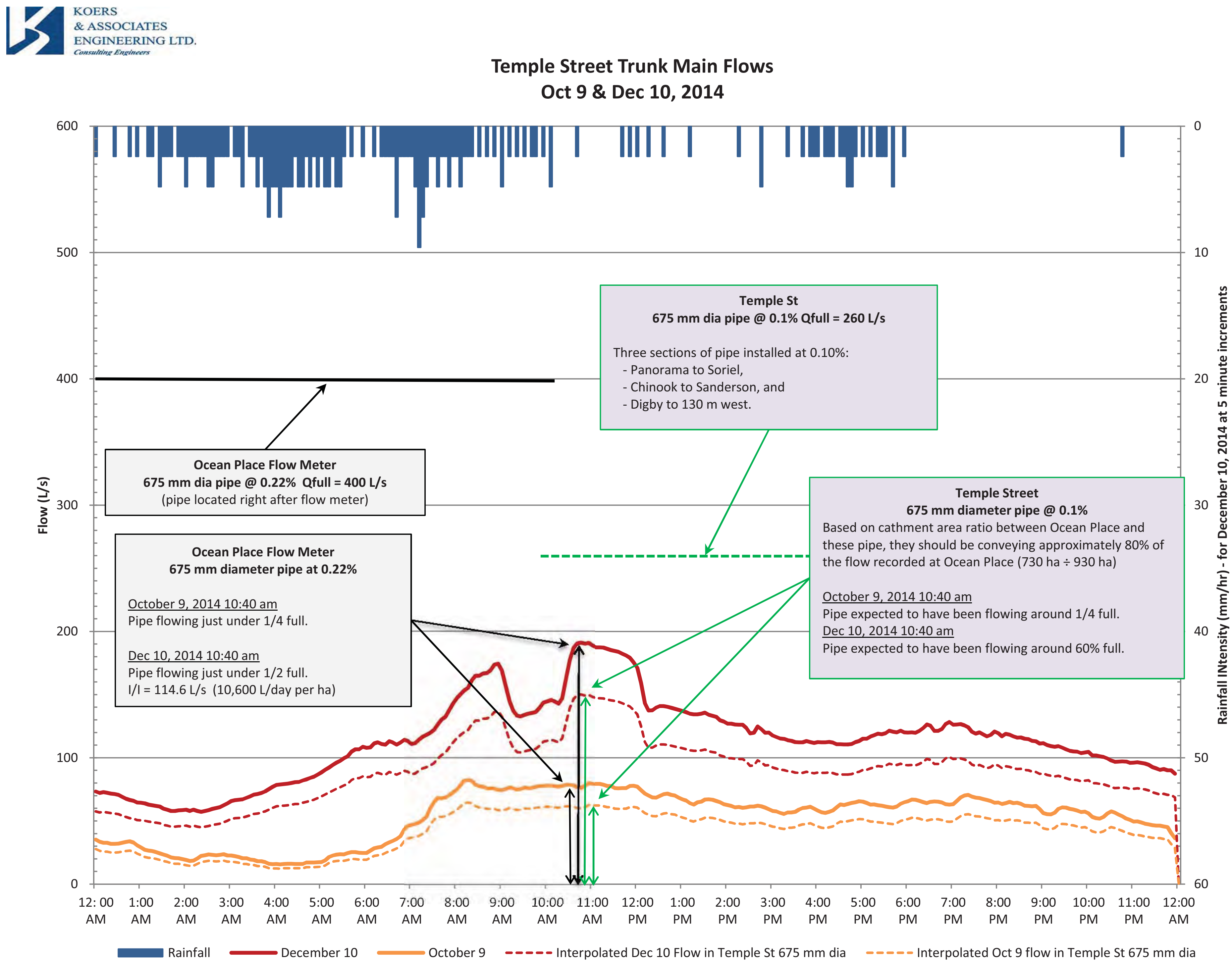
This 2,090 m long 675 mm diameter reinforced concrete trunk main has a calculated just full capacity ranging from 260 L/s to 1,270 L/s based on the reported pipe slopes of 0.1% and 2.29%; respectively.

The computer modelling results show the majority of the trunk main (1,863 m, or 12 of the 19 sections) flow more than 70% full when conveying the daily peak flow during a heavy rainfall event. These are the sections of pipe with slopes between 0.1% and 0.17%. A total of 899 m (6 sections) of the main when conveying the peak daily flow is calculated to be more than 90% full, and 504 m (3 sections) of this is calculated to be flowing at 100% capacity (just full).

The flows recorded at the Ocean Place flow meter during the December 10, 2014 storm event were used to interpolate the depth of flow in the RDN owned 675 mm diameter gravity main along Temple Street. The review suggests the majority of the trunk main is estimated to have been at approximately 40% full for most of the day. During the peak flow, which lasted approximately 1 ¾ hours (10:15 am to 12 noon), the majority of the trunk main is estimated to have been 60% full. This storm recorded the second highest flow (9,851 m^3) at the Ocean Place flow meter during the past 8 years; only being exceeded by the December 24, 2014 storm event (10,095 m^3), as can be seen on **Figure 10**.

A review of depth and duration of flow for a fall day before the onset of fall/winter rains was also carried out for comparative purposes. On October 9, 2014 a total of 4,580 m^3 was recorded at the Ocean Place flow meter. The review suggests the majority of the Temple Street trunk main is estimated to have been approximately 25% full for most of the day, increasing to an estimated 30% during the morning peak flow time.

Figure 15 presents a comparison of the recorded flow at the Ocean Place flow meter for October 9, 2014 and December 10, 2014 along with the estimated



KOERS & ASSOCIATES ENGINEERING LTD.
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Temple Street Trunk Main Flows
Oct 9 & Dec 10, 2014
Temple St
675 mm dia pipe @ 0.1% Qfull = 260 L/s
Three sections of pipe installed at 0.10%:
- Panorama to Soriel,
- Chinook to Sanderson, and
- Digby to 130 m west.
Ocean Place Flow Meter
675 mm dia pipe @ 0.22% Qfull = 400 L/s
(pipe located right after flow meter)
Temple Street
675 mm diameter pipe @ 0.1%
Based on cathment area ratio between Ocean Place and these pipe, they should be conveying approximately 80% of the flow recorded at Ocean Place (730 ha ÷ 930 ha)
October 9, 2014 10:40 am
Pipe expected to have been flowing around 1/4 full.
Dec 10, 2014 10:40 am
Pipe expected to have been flowing around 60% full.
Ocean Place Flow Meter
675 mm diameter pipe at 0.22%
October 9, 2014 10:40 am
Pipe flowing just under 1/4 full.
Dec 10, 2014 10:40 am
Pipe flowing just under 1/2 full.
I/I = 114.6 L/s (10,600 L/day per ha)
Flow (L/s)
Rainfall INtensity (mm/hr) - for December 10, 2014 at 5 minute increments
600
500
400
300
200
100
0
0
10
20
30
40
50
60
12: 00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
4:00 PM
5:00 PM
6:00 PM
7:00 PM
8:00 PM
9:00 PM
10:00 PM
11:00 PM
12:00 AM
Rainfall
December 10
October 9
Interpolated Dec 10 Flow in Temple St 675 mm dia
Interpolated Oct 9 flow in Temple St 675 mm dia

FIGURE 15

flows for the three sections of the Temple St trunk main with the flattest slope (0.1%).

Table 13 lists the pipes calculated to be flowing above the design standard under current conditions and/or at OCP Build-Out. The OCP Build-out review is discussed in **8.3.1 Gravity Sewers**. The majority of the identified mains are owned and operated by the RDN. The percent full values listed in the table have been calculated for the following two different ratios;

- % full = calculated peak flow÷pipe just full capacity
  (commonly referred to as the ratio of q/Q)

- % full = maximum depth of flow÷pipe diameter
  (commonly referred to as the ratio of d/D)

The d/D ratio is the one most appropriate for identifying capacity constraints. The depth of flow is dynamically calculated and is more indicative of actual flow depths compared to the q/Q ratio which is based on Manning's formula and is a static calculation.

8.2.2 Pump Stations

Pump stations were modelled to convey all flows as they enter the station, i.e., the station is able to convey the peak flow. This approach allowed for the checking of the infrastructure downstream of the pump station to see if it had sufficient capacity to handle the peak design flows under existing conditions and at OCP Build-Out. A brief overview of the findings for each of the three pump stations is presented below.

Craig Bay & Martindale Pump Stations (City Owned)

The modelling indicates both the Craig Bay and Martindale pump stations are capable of handling the average day and peak day design flows with a single pump operating under existing conditions for both the City's current design standards (**Table 10**) and the proposed design standards (**Table 11**).

Bay Ave Pump Station (RDN Owned)

The Bay Ave pump station is equipped with four pumps and the RDN's Bylaw No, 500, 1987, 'Schedule 4D' Community Sewer System Standards requires pump stations with more than two pumps to be able to pump the design flow with any one pump out of service.

The computer modelling indicates the pump station is capable of handling the summer average and peak day design flow as well as the winter average day design flow with three or less pumps running. However, the winter peak design flow, calculated at 259 L/s, exceeds the estimated pumping limit of 195 L/s for three pumps operating simultaneously. The RDN design standard requires the pump station be capable of handling the peak flow using only three of the four pumps.

The actual volumes pumped by the Bay Ave station are not known as there is no flow meter in the station. Pump run-hours are recorded on a daily basis, but the pumps operate on a variable speed basis, resulting in their instantaneous pumping rates rising and falling depending on the depth of flow in the wet well. Prior to 2014, pump run-hours for each pump were recorded only on a weekly basis. A review of the 2013 weekly data showed the station averaged 20 run-hours each day. A sustained increase of approximately 4 hours (20%) in July and August was evident. No strong correlation between rainfall and an increase in run-hours was evident. Average daily run-hours vs rainfall for each week of 2013 is presented in **Figure 16**.

Starting in 2014, pump run-hours were recorded on a daily basis. A review of November and December revealed an increase in run-hours in response to

Table 13 – Pipes Characteristics & Percent Full, Existing Conditions & OCP Build-Out

Location Description	Pipe													Comment	Proposed Diameter (mm)
	Number	Dia (mm)	Length (m)	Slope (%)	Material	Age (years)	Capacity (L/s)	Peak Flow, (L/s)		% Full (Flow/Capacity)		% Full (depth/Dia)			
								Existing	Build-Out	Existing	Build-Out	Existing	Build-Out		
Owned by City of Parksville															
Craig Bay Foreshore Trunk Main														**Pipe Capacity Design Standard**	
Nicklin – Seaway	SP-0923	300	147	0.25	RC	21	50	36	47	72%	94%	69%	86%	70%	monitor
Corfield Road														70%	
Stanford – 80 m north	SP-0829	200	87	0.47	PVC	n/a	30	14	31	47%	97%	51%	141%	Replacement Design Underway	250
80 m north – 80 m south of Jensen	SP-0496	200	108	1.56	AC	40	40	15	32	38%	80%	43%	69%	Replacement Design Underway	250
80 m south of Jensen - Jensen	SP-0495	200	106	2.0	AC	40	46	15	34	33%	74%	39%	63%	Replacement Design Underway	250
Jensen – 54 m north	SP-0752	200	54	1.75	AC	52	40	19	40	48%	100%	46%	76%	Replacement Design Underway	250
92 m north of Jensens – 117 m north	SP-0751a	200	25	1.36	AC	n/a	40	19	40	48%	100%	49%	88%	Replacement Design Underway	250
Owned by RDN															
Parksville Bay Trunk Main (Corfield to Bay Ave Pump Station)														**Pipe Capacity Design Standard**	
Community Park	SP-0681	600	100	0.10	RC	37	190	152	209	80%	110%	65%	108%	100%	monitor
Community Park	SP-0682	600	110	0.12	RC	37	210	152	209	72%	100%	63%	108%	100%	monitor
Community Park	SP-0683	600	175	0.12	RC	37	210	152	209	72%	100%	64%	117%	100%	monitor
Parks Sands RV Park	SP-0684	600	92	0.11	RC	37	200	155	212	78%	106%	64%	117%	100%	monitor
Parksville Beach Resort	SP-0685	600	156	0.12	RC	37	210	155	217	74%	103%	66%	118%	100%	monitor
Beach Club Resort	SP-0686	600	154	0.12	RC	37	220	155	218	70%	99%	73%	120%	100%	monitor
Sea Edge Motel (below high tide)	SP-0687	600	147	0.12	RC	37	210	194	265	92%	126%	73%	120%	100%	monitor
Single family lots (below high tide)	SP-0680	600	154	0.12	RC	37	220	195	266	89%	121%	73%	105%	100%	monitor
Paradise RV Park (below high tide)	SP-0679	600	129	0.12	RC	37	210	199	269	95%	128%	72%	93%	100%	monitor
Quality Resort (below high tide)	SP-0677	600	19	0.14	RC	37	230	199	269	87%	117%	63%	78%	100%	monitor
Quality Resort (below high tide)	SP-0678	600	120	0.23	RC	37	290	200	270	69%	93%	61%	78%	100%	monitor
Bay Ave, Pump Station Inlet	SP-0942	600	14	0.29	RC	37	330	200	270	61%	82%	52%	61%	100%	monitor
Temple Street Trunk Main (end of Bay Ave Forcemain to Ocean Pl)														**RDN Pipe Capacity Design Standard**	
Doehle - Panorama	SP-0623	675	62	0.13	RC	37	300	259	336	86%	112%	70%	134%	100%	monitor
Panorama - Soriel	SP-0622	675	137	0.13	RC	37	310	260	336	84%	108%	70%	134%	100%	monitor
Soriel - 100 m west	SP-0676	675	95	0.14	RC	37	310	262	337	85%	109%	69%	123%	100%	monitor
100 m west - Chinook	SP-0601	675	138	0.19	RC	37	360	262	338	73%	94%	79%	128%	100%	monitor
Chinook - Sanderson	SP-0675	675	166	0.10	RC	37	260	263	339	101%	130%	79%	128%	100%	monitor
Sanderson - Digby	SP-0253	675	115	0.11	RC	37	270	264	341	98%	126%	77%	112%	100%	monitor
Digby - 130 m west	SP-0674	675	136	0.10	RC	37	260	264	342	102%	132%	77%	104%	100%	monitor
130 m west - Pyme	SP-0673	675	154	0.12	RC	37	290	265	342	91%	118%	72%	91%	100%	monitor
Pyme - Fairwind	SP-0672	675	131	0.16	RC	37	340	276	357	81%	105%	68%	84%	100%	monitor
Fairwind - 27 m east of Foster	SP-0671	675	138	0.17	RC	37	350	276	357	79%	102%	66%	78%	100%	monitor
27 m east of Foster - Foster	SP-0419	675	27	0.79	RC	37	750	276	357	37%	48%	42%	63%	100%	monitor
Foster - Sunray	SP-0420	675	93	0.79	RC	37	750	278	359	37%	48%	42%	51%	100%	monitor
Sunray Rd, Temple - Sunray Cl	SP-0029	675	101	0.71	RC	37	710	280	361	39%	51%	46%	58%	100%	monitor
Sunray Rd, Sunray Cl - Wright	SP-0030	675	72	0.69	RC	37	700	280	361	40%	52%	72%	102%	100%	monitor
Wright Rd, Sunray - 125 m west	SP-0031	675	123	0.15	RC	37	330	283	364	86%	110%	74%	102%	100%	monitor
Wright Rd, 120 m west - SRW	SP-0005	675	200	0.13	RC	37	300	283	364	94%	121%	74%	95%	100%	monitor
Sideyard SRW, Wright - Cavin	SP-0032	675	87	2.18	RC	37	1,240	305	386	25%	31%	55%	66%	100%	monitor
Cavin Rd, Sideyard SRW - Ocean Pl	SP-0034	675	57	0.48	RC	37	580	305	386	53%	67%	70%	83%	100%	monitor
Cavin Rd, Ocean Pl - Glenhale Cres	SP-0035	675	115	0.16	RC	37	330	305	386	92%	117%	70%	83%	100%	monitor
Hwy 19A Trunk Sewer														**RDN Pipe Capacity Design Standard**	
SRW, Aberdeen - Temple	SP-0006	200	93	0.36	PVC	n/a	20	20	20	100%	102%	74%	74%	50%	monitor

Bay Ave Pump Station
Weekly Average Day Pump Runhours, 2013

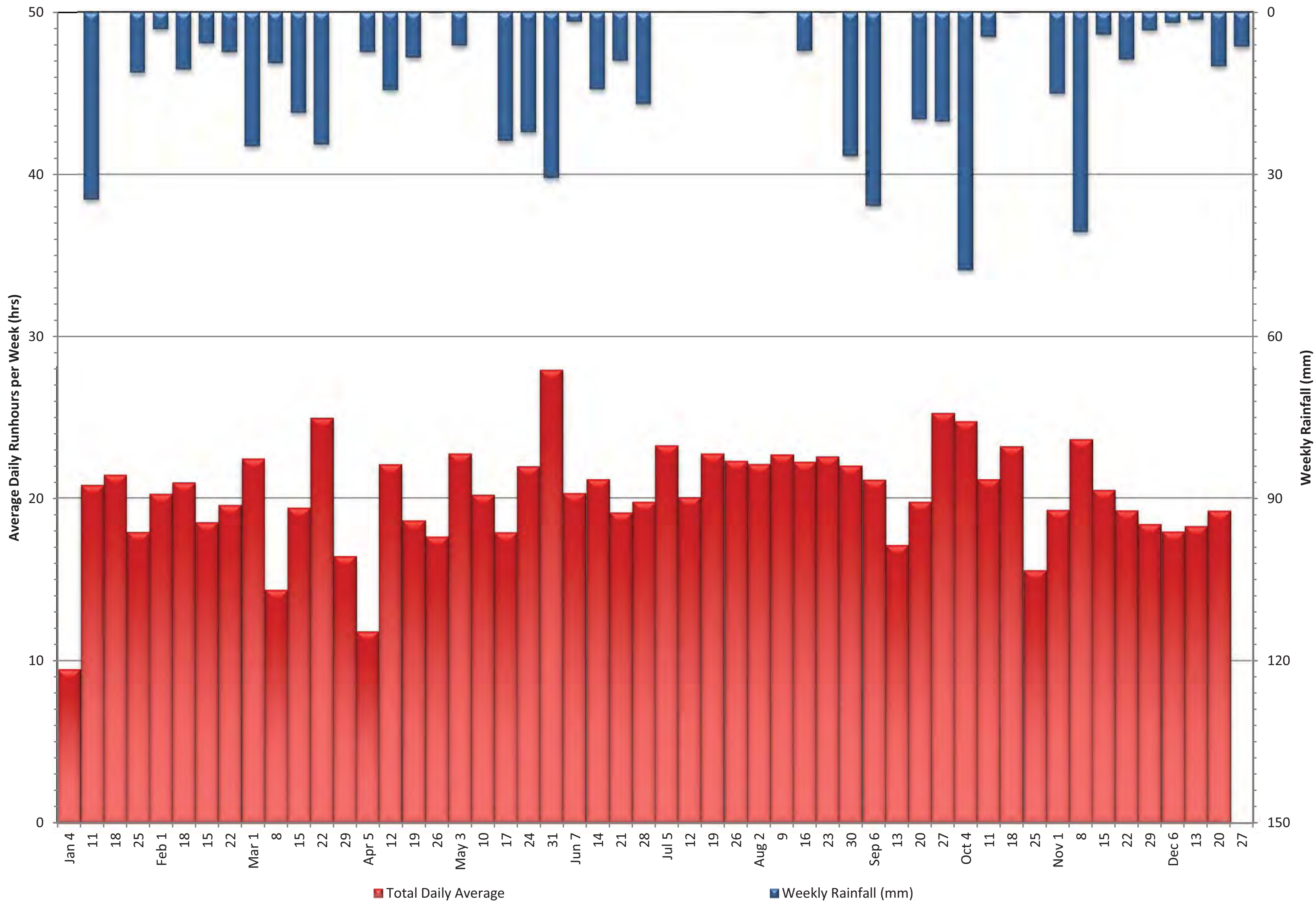

FIGURE 16

rainfall. Of particular note was the increase in total daily runtime from the average of 20 hours each day with no rainfall, to a high of 45 hours on December 11, followed by 43.7 hours on December 12. During the storm, three pumps were recorded to be operating simultaneously for slightly more than 1 hour on December 10, for more than 2 hours on December 11 and for more than 3½ hours on December 12.

The recorded total daily pump run-hours for the Bay Ave pump station for November and December 2014 is graphically presented in **Figure 17** along with the daily rainfall recorded at the City's Community Park rain gauge. The dates appear to be out by one day, as a review of the Ocean Place flow meter recorded flows for the month December shows the flows increasing on December 9, 10 and 11, with the peak occurring on December 11 as seen in **Figure 18** along with rainfall. Prorating the flows back to the Bay Ave pump station based on catchment area (78%) suggests the pumping capacity for two pump operating simultaneously (137 L/s) would have been exceeded for 1½ hours (between 10:30 am and 12 noon) and the station may have pumped at a peak rate of 150 L/s for almost ½ hour.

Table 14 presents a summary of the average day and peak day design flows versus the calculated pumping capacity with one or more pumps operating at the City's two pump stations and the RDN's Bay Ave pump station.

Table 14 - Pump Station Existing Conditions & OCP Build-Out Design Flows vs Current Capacity

Flow into Pump Stations										
Description	Table 11 Design Standards Flow (l/s)				Design Capacity with # of Pumps Operating				Spare Capacity at OCP Build-Out Peak Flow	
	Existing Conditions		OCP Build-Out							
	Ave Day	Peak	Ave Day	Peak	1	2	3	4	(l/s)	(%)
Craig Bay C3201 HT pump	34	53	38	62	38 - 62*	48 - 100**	-	n/a	39	39%
Martindale C3201 HT pump	44	66	62	90	51 - 97#	60 - 173##	-	n/a	85	49%
Bay Ave (RDN) C3201 pump	182&	259&	240&	334&	73	137	195	240	94	39%

Notes:
The flows below are based on a Hazen Williams friction coefficient (C) value of 110 and 130; respectively. They are derived from the pump station O&M Manual, Composite Pump Curves figure, a copy of which is included in **Appendix A**.

*	38 - 42 L/s,	457 impeller, one 200 mm dia. forcemain
	46 - 50 L/s,	452 impeller, one 200 mm dia. forcemain
	52 - 55 L/s,	457 impeller, both 200 mm dia. forcemains
	60 - 62 L/s,	452 impeller, both 200 mm dia. forcemains
**	48 - 54 L/s,	457 impellers, one 200 mm dia. forcemain
	76 - 83 L/s,	457 impellers, both 200 mm dia. forcemains
	91 - 100 L/s,	452 impellers, both 200 mm dia. forcemains
#	51 - 57 L/s,	454 impeller, one 200 mm dia. forcemain
	69 L/s,	452 impeller, one 200 mm dia. forcemain
	80 - 82 L/s,	454 impeller, one 300 mm dia. forcemain
	95 - 97 L/s,	452 impeller, one 300 mm dia. forcemain

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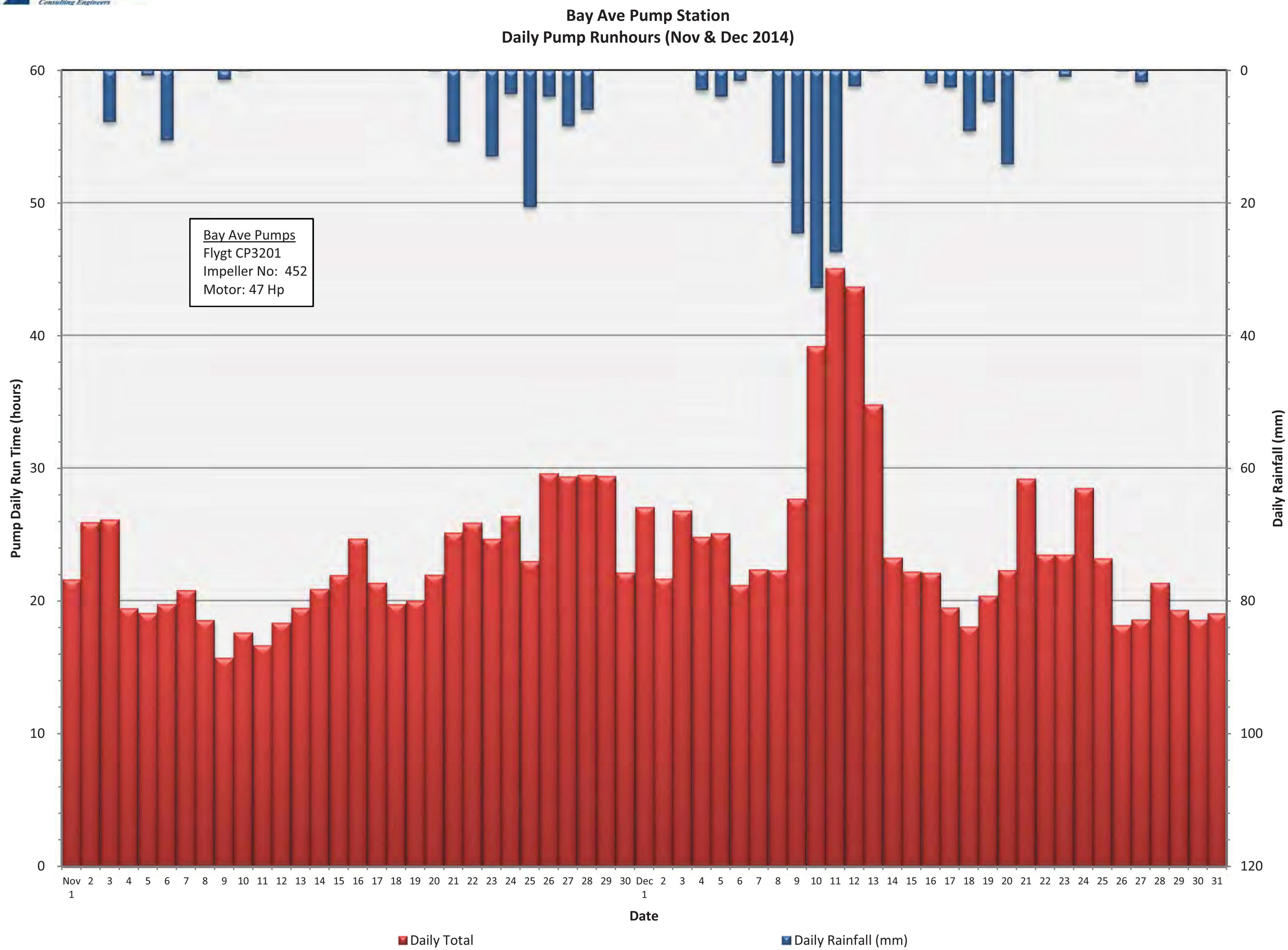
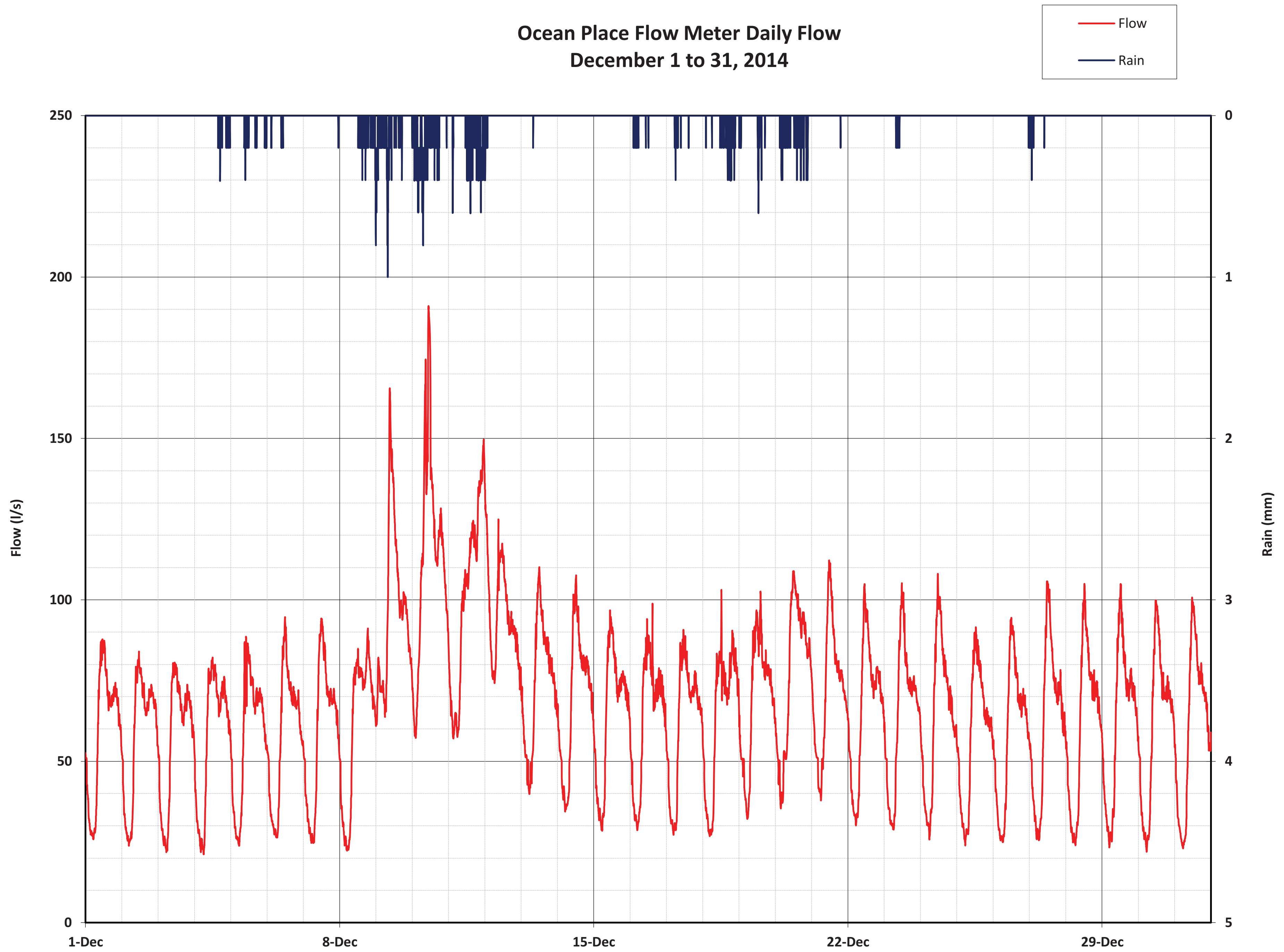


FIGURE 17



Ocean Place Flow Meter Daily Flow
December 1 to 31, 2014
Flow
Rain
250
200
150
100
50
0
5
4
3
2
1
Flow (l/s)
Rain (mm)
1-Dec
8-Dec
15-Dec
22-Dec
29-Dec

FIGURE 18

60 - 69 L/s, 454 impellers, one 200 mm dia. forcemain
122 - 134 L/s, 454 impellers, one 300 mm dia. forcemain
142 - 145 L/s, 454 impellers, 200 mm & 300 mm dia. forcemain
148 - 160 L/s, 452 impellers, one 300 mm dia. forcemain
171 - 173 L/s, 452 impellers, 200 mm & 300 mm dia. forcemain

& Modelled flows are a conservative projection as they are based on full occupancy of the City's tourism accommodation coinciding with the peak I&I allowance.

8.3 OCP Build-Out

The system was modelled using the proposed design standards (**Table 11**) and the anticipated future growth in the areas shown in **Figure 13**. The permanent and tourism population at OCP build-out is projected to be 21,900 and 11,300; respectively, resulting in a calculated peaking factor of 2.15.

As noted in **8.1 Historical & Design Flows**, the design flows modelled are a conservative approach to the system analysis as they are based on full occupancy of the City's tourism accommodation coinciding with a peak I&I allowance. Peak tourism occupancy occurs around the summer vacation long weekends at the beginning of July, August and September.

8.3.1 Gravity Sewers

City pipes that are calculated to be more than 70% full are shown on **drawings 1346-03**, and **1346-07**. A brief discussion of the City owned and RDN owned mains is presented below.

City Owned Mains

Modelling indicates three locations where upgrading work is required to accommodate future growth.

Craig Bay Foreshore West Trunk Main, (Nicklin to Seaway)

For this 147 m long section of 300 mm diameter reinforced concrete main, computer modelling calculates a peak flow of 47 L/s at OCP Build-Out compared to the calculated pipe just full capacity of 50 L/s. For Current Conditions the computer model calculated the pipe at 72% full for the peak flow of 36 L/s.

The need for the upgrading work should be confirmed after flow monitoring, video inspection, and surveying of the pipe grade. If upgrading is required, this should be a DCC eligible project

Despard Ave (Craig to Aurora)

Development of the presently vacant land on the south side of Despard Ave, west of Craig Street is expected to accommodate almost 1,400 people. The downstream Craig Street sewer main does not have sufficient capacity to handle this calculated flow increase. The flows can be diverted into the Corfield Avenue sewer main which has more spare capacity than Craig St main but would require the upsizing of 380 m along Corfield from Standard Ave to north of Jensen Ave. The diversion can be carried out with the construction of approximately 100 m of 200 mm diameter main along Despard between Craig to Aurora. This project was identified in the City's previous Sanitary Sewer Study.

This should be a DCC eligible project.

Corfield Avenue (Stanford to north of Jensen)
This project is required to accommodate future growth including the development of the land on the south side of Despard Ave, west of Craig Street, flow from which will be conveyed onto Corfield with the completion of the Despard Ave (Craig to Aurora) project noted above.

A total of 380 m of the 200 mm diameter pipe should be upsized to 250 mm diameter along Corfield Avenue from Standard Ave to north of Jensen Ave.

This should be a DCC eligible project.

RDN Owned Mains
Modelling indicates three areas where RDN mains may be undersized when conveying the peak flow.

Parksville Bay Trunk Main (Corfield to Bay Ave Pump Station)
Nine of the 12 sections, totaling 1,068 m, of this 600 mm diameter reinforced concrete main are calculated to flow more than 100% full during peak flow. The remaining three sections, totaling 293 m, are calculated to flow between 83% and 97% full during peak flow.

Temple Street Trunk Main (Doehle Ave to 75 m west of Ocean Place flow meter)
12 of the 19 sections, totaling just under 1,500 m of this 675 mm diameter reinforced concrete main are calculated to flow more than 100% full during peak flow. Two other sections, totaling 200 m, are calculated more than 90% full. The remaining 5 sections, totaling 398 m are calculated to flow less than 70%.

It should be noted that all pipes would be adequately sized to convey the projected average day design flow at OCP Build-Out and peak flows during dry weather conditions when there is little to no I&I.

Hwy 19A Trunk Main (Wembley Mall to Wright Road)
Only one section of this trunk main is calculated more than 70% full when conveying the peak flow. The 93 m length of 200 mm diameter main located in the side yard SRW between Aberdeen Dr and Temple St is calculated to flow at 87% full.

Table 13 lists the pipes calculated to be flowing above the design standard at OCP Build-Out and/or under existing conditions; as previously discussed in **8.2.1 Gravity Sewers**. The majority of the identified mains are owned and operated by the RDN.

8.3.2 Pump Stations

The modelling indicates both the Craig Bay and Martindale pump stations and their forcemains have adequate capacity to meet the projected peak flows at OCP Build-Out but will require specific pump impellers and use of one or more of the existing forcemains. For the RDN owned Bay Ave pump station, the calculated peak flow will exceed the current pumping capacity. A brief discussion of each is presented below.

Craig Bay Pump Station (City Owned)
Conveying the peak flow will require the use of one of the existing Flygt C3201 HT pump and both of the existing 200 mm diameter forcemains. The pump is to be equipped with a #452 impeller.

Martindale Pump Station (City Owned)
Conveying the peak flow will require the use of one of the Flygt C3201 HT pump and the 300 mm diameter forcemain; not the 200 mm diameter. The pump is to be equipped with a #452 impeller.

Bay Ave Pump Station (RDN Owned)
While the calculated average day flow at OCP Build-Out (193 L/s) is equivalent to the estimated pumping capacity of three of the four pumps operating simultaneously (195 L/s), the calculated peak flow (285 L/s) will exceed the pumping capacity of all four pumps operating simultaneously (240 L/s)by 45 L/s, or 19%.

Table 14 presents a summary of the average day and peak day design flows versus the calculated pumping capacity with one or more pumps operating at the City's two pump stations and the RDN's Bay Ave pump station.

9 CAPITAL PLANNING

As part of the development of the Master Plan, a risk assessment of the infrastructure (mains and pump station) calculated to be operating beyond the design standard 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 areas were applied in assessing the **condition** and the **conveyance capacity** of each project. The two assessments were then used to develop a capital planning score to rank the priority of the projects.

9.1 Condition Risk Analysis

The condition analysis assesses the 'Likelihood of Failure' based on the age of the infrastructure against the 'Consequence of Failure' based on the cost to restore service. The rating scale used for each is shown in the two tables below.

Likelihood of Failure
(Age of Structure)

Likelihood of Failure Rating	Age of Structure (as % of excepted life span)
5	>150%
4	125% - 150%
3	100% - 125%
2	75% - 100%
1	< 75%

Consequence of Failure
(Cost to Restore)

Consequence of Failure Rating	Severity	Cost to Restore
1	Insignificant	$0 - $100,000
2	Minor	$100,000 - $250,000
3	Moderate	$250,000 - $500,000
4	Major	$500,000 - $750,000
5	Severe	> $750,000

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, the higher the risk.

Condition Risk Score

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

9.2 Capacity Risk Analysis

The capacity analysis assesses the 'Likelihood of Failure' based on exceeding the capacity of the infrastructure against the 'Consequence of Failure' based on the number of people affected. The rating scale used for each is shown in the two tables below.

Likelihood of Failure (Top Water Level)

Likelihood of Failure Rating	Hydraulic Capacity
5	> Ground Elevation or MHFE
4	<= Ground Elevation or MHFE
3	Depth/Diameter = 1.5
2	Depth/Diameter = 1.0
1	Depth/Diameter <= 0.75

Consequence of Failure (Number of people impacted)

Consequence of Failure Rating	Severity	Number of People Impacted
1	Insignificant	0
2	Minor	0 - 20
3	Moderate	20 - 100
4	Major	100 - 300
5	Severe	> 300

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, the higher the risk.

Capacity Risk Score

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

9.3 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.

Capital Planning Priority Score

Capacity \ Condition	1	2	3	4	5
5	2	2	3	3	3
4	2	2	3	3	3
3	1	2	2	3	3
2	1	1	2	2	2
1	1	1	1	2	2

Condition

Where:

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

The calculated priority score for the projects identified in this study is shown in **Table 15**.

Table 15 – Project Priority Score

Project	Condition Risk Analysis			Capacity Risk Analysis			Project Priority Score
	Likelihood	Consequence	Score	Likelihood	Consequence	Score	
City of Parksville Owned Infrastructure							
Trunk Mains							
Craig Bay Foreshore, west end	1	3	2	2	5	3	**2**
Collection Mains							
Corfield Road	1	3	2	3	5	4	**2**
RDN Owned Infrastructure							
Pump Station & Forcemain							
Bay Ave	1	5	3	1	5	3	**2**
Trunk Mains							
Parksville Bay (Corfield Rd to Bay Ave PS)	1	5	3	3	5	4	**3**
Temple St & Wright Rd	1	3	2	3	5	4	**2**
Hwy 19A (sideyard SRW between Aberdeen Dr & Temple St)	1	3	2	2	5	3	**2**

Notes:

Project Priority Score

1 = Low Priority (Low Consequence of Failure Risk Score for Condition & Capacity)
2 = Moderate Priority (Moderate Consequence of Failure Risk Score for Condition & Capacity)
3 = High Priority (High Consequence of Failure Risk Score for Capacity & Condition)

9.4 Inflow & Infiltration Management

As noted previously in **4.4 Inflow & Infiltration** and as seen in **Figure 10**, I&I is occurring in response to rainfall events. The findings of the flow monitoring program revealed I&I at each of the four temporary and two permanent sites, indicating it is occurring in older and recent developed areas.

Design allowances for I&I vary depending on the age and condition of the collection system. Generally accepted allowances for urban development are as follows and are the same allowances in the MMCD Design Guideline, 2014:

- New collection systems: 5,600 L/day per ha
- Old systems (25 years or older): 11,200 L/day per ha

The overall age of the City's collection system as noted in **2.2 Collection System** (50% less than 30 years old and 50% more than 30 years old), in general lends itself to an I&I rate of 11,200 L/day per ha and which the flow monitoring program supported for the Ocean Place catchment. However, higher I&I rates ranging from 11,300 to 12,700 L/day per ha were calculated for the three temporary monitoring sites for an event that resulted in a calculated I&I rate at Ocean Place of only 7,700 L/day per ha as shown in **Appendix C – Table 2.** These I&I rates are higher than expected, most notably for the Acacia catchment which comprises newer residential development.

While I&I will always be a part of a sewer collection system, large I&I flows can result in the unnecessary oversizing of infrastructure (mains, pump stations, forcemains and sewage treatment plants). There are several tools which can be used for locating I&I sources. These include:

- Operational Staff knowledge of problem areas during rainfall events
- Public complaints during rainfall events
- Visual inspection of manhole covers
- Flow monitoring
- Video camera inspection
- Smoke testing
- Dye testing

The results of the flow monitoring data analyses suggests implementation of an I&I management plan would be appropriate for the City of Parksville.

9.5 Infrastructure Condition Verification

The computer model has been developed using the City's GIS database and augmenting it with record drawing information and the conveyance capacity of the mains 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:

- I&I Management program
- prioritizing identified maintenance works
- coordinating the findings with road construction works to ensure the underground infrastructure is dealt with before the road is repaved

This video inspection of up to 8 kms of pipe per year would result in the video inspection of the entire 80 kms of pipes in 10 years.

10 RECOMMENDED IMPROVEMENTS & COST ESTIMATES

Table 16 lists the recommended upgrading works, as discussed in section **8 Modeling Results**, in order of priority to meet the anticipated flows in accordance with the OCP when the City's population is projected to reach 22,000 and the summertime tourism population is projected to reach 11,200; for a combined peak service population of 33,200. The project locations are shown on **drawing 1346-1** to **1346-7**. The area covered by each map sheet is shown on **Figure 19**.

The cost estimates are based on Class 'D' (feasibility study) estimates, 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.

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.

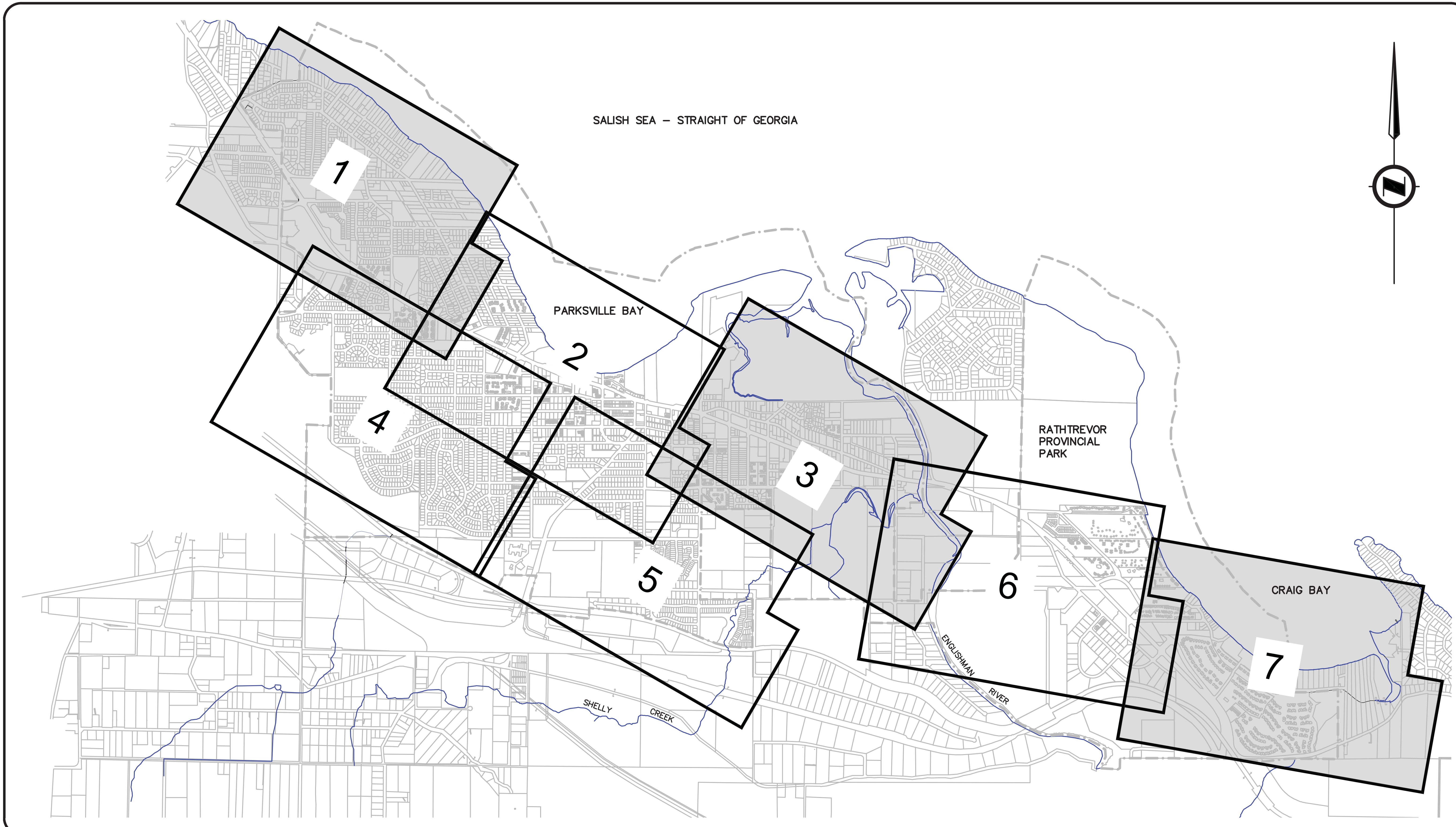
Table 16 – Proposed Works

Priority	Description	Pipe Dia. (mm)	Pipe Length (m)	Cost Estimate Class 'D' (excluding GST)	DCC Eligible Project
1	I&I Identification and Reduction Program Wintertime Flow Monitoring, Video Inspection, Smoke and Dye Testing Summertime works to Reduce I&I	-	-	$90,000 to $125,000	No
2	Corfield St (Stanford Ave to 60 m north of Jensen Ave) Upgrade existing 200 mm diameter	250	380	$300,000	Yes
3	Despard Ave (Aurora St to Craig St) New pipe	200	160	$125,000	Yes
4	Wembley Rd (100 m west of Constantine Pl to Church Rd) New pipe	200	100	$75,000	Yes

A brief discussion of each project is presented below.

1. I&I Identification & Reduction Program
   This includes an allowance of $40,000 to $60,000 for the video inspection and smoke testing 4 to 8 kms of pipe per year. This would result in the video inspection of the entire 85 kms of pipe over a 10 to 20 year period.

   An allowance of $50,000 to $65,000 is suggested to carry out 10 to 20 spot repairs. This would be for works such as: grouting of manholes; digging and replacing of service connection at the main or in-situ repair (injection grouting); notifying property owners in advance of video, inspection and smoke testing; elimination of cross connections on public property; and if required, notifying and working with property owners to eliminate cross connections on private property.



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CLIENT	CITY OF PARKSVILLE
PROJECT	SANITARY MASTER PLAN

TITLE	COLLECTION SYSTEM DRAWINGS KEY PLAN		
APPROVED		SCALE	1:25,000
DATE	AUG 2015	DWG No.	FIGURE 19
PROJECT No.	1346		

In general, federal and provincial infrastructure and study grant programs do not specifically mention I/I programs as being an eligible projects. However, I/I reduction can be part of a tangible asset management program. Mains that are identified for repair/replacement could become part of an infrastructure renewal/beautification project. The case may also be made that a reduction in I/I is part of a sustainability program by reducing power requirements at pump station(s) and at the French Creek Pollution Control Centre.

2. Corfield St

   This upgrading project is required to accommodate the future development of 27.7 ha of vacant land on the south side of Despard Ave, west of Craig Street, and will need to be carried out before or in conjunction with project 3, below.

   This should be a DCC eligible project. It could also be incorporated into part of an infrastructure renewal/road beautification project; both of which may meet eligibility criteria for provincial and federal infrastructure programs.

3. Despard Ave

   This upgrading project is required to accommodate the future development of 27.7 ha of vacant land on the south side of Despard Ave, west of Craig Street. Project 2, above, is to be carried out before or in conjunction with this project. The estimated future service population of the 27.7 ha is 1,360.

   This should be a DCC eligible project. It could also be incorporated into part of an infrastructure renewal/road beautification project; both of which may meet eligibility criteria for provincial and federal infrastructure programs.

4. Wembley Rd

   This project extension of an existing sewer main is required in order to provide service to 1 ha of vacant land on south side of Wembley Road, west of Church St. The estimated future service population of this 1 ha is 80.

   These works may only benefit a single development and the timing of the work connected to the start of the development. As such, the construction of these off-site works would be required as part of a Works & Services Agreement that would be created as part of the development approval process.

11 CONCLUSIONS

The following conclusions are drawn from the work presented in this report:

Collection System

1 The sewage collection system consists of more than 85 kilometres of gravity sewer main as well as three large pumping stations. Approximately 50% of the mains are less than 30 years old and the remaining 50% are between 30 and 60 years old.

2 Within the City, the Regional District of Nanaimo owns and operates more than 4 kms of gravity trunk sewer main as well as the Bay Ave pump station and its 1 km forcemain.

3 Some lands within the RDN are serviced by sanitary sewer mains that discharge into the City of Parksville's collection system. East of the Englishman River, the Pacific Shores resort discharges into the City's foreshore trunk main that conveys sewage to the Craig Bay pump station. West of the Englishman River, flows from approximately 250 properties in the Neden Way/Riley Road subdivision area enters the City at Hwy 19A at the Aberdeen Drive intersection.

Service Population

4 The City's population is expected to continue to grow and is planned for within the City's Official Community Plan, which was adopted in 2013.

5 City planning Staff indicated that development in accordance with the OCP can result in a future permanent population of 21,900 plus a peak seasonal tourist population of 11,300. Extrapolation of available population projection data suggests OCP Build-Out would occur by Year 2072 for the combined peak service population of 33,200. This is lower than the 41,600 projected in the *City of Parksville Sanitary Sewer Study Update, September 1996*.

Flow Monitoring

6 The use of three temporary and two permanent flow monitoring stations revealed that I&I is occurring in both new and old developed areas of the City. The catchment area land-uses ranged from: new, completely developed; to older, completely developed; to partially developed. The areas covered by the site ranged from just under 5% of the system (43 ha of 930 ha) to 100% of the system. The catchment areas as shown in **Appendix C Figure 1.**

7 The flows recorded at the City's permanent monitoring site in the Community Park were much lower than expected. An inspection of the station in the spring of 2014, confirmed the station was under-recording by approximately 50%. The station was subsequently recalibrated in September 2014 by SFE Global. An attempt by SFE Global Staff to apply a correction factor to the recorded flow data was not successful.

Inflow/Infiltration

8 I&I rates in the collection system vary depending on the:
   i) size of the catchment area,
   ii) intensity of the rainfall,
   iii) duration of the storm event, and
   iv) length of time between rainfall events.

I&I During 3 ½ month Flow Monitoring Period (Dec 2013 – mid March 2014)

January 10/11, 2014 Rainstorm

9 During the 3 ½ month flow monitoring period (Dec 2013 to mid Feb 2014), the largest rate of I&I was in response to the January 10/11, 2014 rainstorm event.

10 The increase in the average daily flow at the five monitoring sites ranged from 2.2 L/s for the smallest monitored catchment area (43 ha), to more than 41 L/s for the largest monitored catchment area (930 ha). This equates to a 24 hour average I&I rate ranging from 3,600 L/day per ha to 5,800 L/day per ha. This is lower than the City's design standard of 8,640 L/day per ha.

11 The largest instantaneous difference in flow ranged from 5.3 L/s (43 ha) which was sustained for approximately 5 minutes, to 81 L/s (930 ha) which was sustained for approximately 30 minutes. These differences when extrapolated over a 24 hour period, equate to a daily I&I rate of 12,700 L/day per ha to 7,700 L/day per ha.

12 The January 10/11, 2014 storm event was not an unusual storm. It plotted just above a 2 year return period for the 24 hour duration on the City's proposed updated IDF curves.

Ocean Place Flow Monitor I&I (July 2007 – May 2015)

13 During the past 8 years, there have been 10 storm events that resulted in a total daily flow of more than 8,000 m^3/day; this includes the Jan 11, 2014 event which recorded a flow of 8,108 m^3/day. Five of ten events resulted in a daily flow of more than 9,000 m^3/day. One event exceeded 10,000 m^3/day.

December 24, 2010 Rainstorm

14 The highest daily flow was 10,095 m^3/day recorded on December 24, 2010. The volume increase as a result of the rainfall was estimated at 6,040 m^3; equating to an average I&I rate of 6,700 L/day per ha. The peak flow that day was 186 L/s; an increase of 120 L/s over the expected flow. Extrapolating this increased flow over 24 hours, equates to a constant I&I rate of 11,500 L/day per ha.

December 10, 2014 Rainstorm

15 This storm event resulted in a recorded flow of 9,851 m^3/day; the second highest in the past 8 years. The calculated 5,000 m^3 of additional flow equates to an average I&I rate of 5,500 L/day per ha.

16 For approximately ½ hour, between 10:50 am and 11:20 am, a peak flow of 115 L/s was recorded. This equates to an I&I rate of 11,000 L/day per ha when extrapolated over a 24 hour period.

17 While, the December 10, 2014 rainstorm had less than a 2 year return period, it was preceded by a similar storm on December 9 and followed by another on December 11. A review of cumulative rainfall over a three day period for Environment Canada's Comox Airport weather station (ID No. 1021830), revealed the events frequency at once every 5 to 7 years for the cumulative rainfall over three days.

September 2, 2013 Rainstorm

18 A short duration high intensity rainfall on September 2, 2013 caused localized flooding; most notably in the Corfield Rd/Hwy 19A intersection area. A total of 33.2 mm of rain was recorded at the Community Park rain gauge during the 30 minute storm, which occurred between 7:00 to 7:30 pm. While only an additional 516 m^3 of flow passed through the flow meter as a result of the storm, the flow at 8:20 pm increased to 190 L/s compared to 72 L/s the previous day; an increase of 118 L/s. While the 516 m^3 volume equates to an average I&I rate of

only 570 L/day per ha, the 118 L/s equates to an I&I rate of 11,300 L/day per ha when extrapolated over 24 hours.

19 The 30 minute storm plotted well above the 100 year return period.

Per Capita Flows

20 Flows in the City vary seasonally, exhibiting a diurnal pattern (two peak and two troughs). Flows rise in the winter and summer months and fall during the spring and fall months. As a result, per capita flows vary seasonally.

21 In 2014, the lowest flow month was May, resulting in a calculated average day per capita flow of 371 lpcd. For the highest flow month of December, the calculated per capita flow was 452 lpcd. The average for the year was 403 lpcd. These per capita flows are based on all flow generated by the permanent population as well as all non-residential development flows (commercial, industrial, and institutional) and I&I.

22 The City's per capita design flow for the residential population is 410 lpcd. This does not include any flows from non-residential development (commercial, industrial, and institutional) or for I&I; which are calculated separately.

23 Calibration of the computer model indicates a per capita flow of 300 lpcd for the permanent population.

Hotel & Motel Flows

24 The City design standards do not have a designated unit flow for hotels or motels, which make up a significant component of the commercial development. For modelling purposes, a value of 300 L per patron per day (lppd) for hotel and 500 lppd for motel were applied for the estimated 8,000 patrons (tourists) that are reported to stay in the City during the peak of the summer vacation season.

Commercial/Industrial Flows

25 The City's design standard flow for commercial and industrial lands is 22,500 L/day per ha. This equates to a residential population of 55 people/ha, which equates to 26 dwelling units per ha. This is similar to the OCP's transitional neighbourhood which has a medium density (20 – 40 units/ha) and is between that of a single unit residential area and a multi-unit residential area.

26 The current 22,500 L/day per ha unit flow is considered to be conservative for the perceived relatively low water demands generated by the service commercial and light industrial businesses operating in the City.

27 For modelling purposes, commercial and industrial land flows were modelled to occur between 9 am to 5 pm, to reflect typical working hours.

Peaking Factor

28 The City's design standards call for a peaking factors:

PF = 5 for populations of less than 1,000,

PF = 4 for populations up to 3,000.

It does not state what values is to be applied for more than 3,000 people, but it is assumed to be 4.

29 A calculation of Peaking Factors based on the flow data recorded during a four week period of very little rain during the December 2013 to mid-March 2014 flow monitoring, resulted in:

PF = 3.4 to 2.5 for an estimated population of 800 to 1,000

PF = 2.0 for an estimated population of 2,500 to 3,500

PF = 1.6 for an estimated population of more than 12,000

This suggests the design standard PF may be high. The Peaking Factors more closely reflect those calculated by the MMCD Peaking Factor formula of $6.75P^{-0.11}$.

<u>Modelling Results</u>

Calibrated Model

30 A good fit of the computer model generated hydrograph to the recorded hydrograph for the 35 ha area of fully developed residential (SHM 547 at 101 Acacia St) was achieved using a 300 lpcd unit flow.

31 For the flow monitoring catchment areas that also contained commercial, institutional, and/or industrial development, the model generated hydrographs were higher than the recorded. This suggests the design flow of 22,500 L/day per ha for commercial and industrial development and the 410 lpcd for institutional development is high. The calculation of a higher flow, and not a lower flow, compared to the recorded flow is preferred for planning purposes to provide a more conservative approach with a better factor of safety.

Existing Conditions

City Owned Infrastructure

32 The results from the computer model, which was developed using the City's GIS database and some record drawings, suggest the City mains and the two City pump stations should have adequate capacity to convey the calculated peak flow.

RDN Owned Infrastructure

33 Three sections of the gravity trunk main downstream of the Bay Ave pump station are calculated to be just over the RDN design requirement of 100% full when conveying the peak flow which includes the peak tourist season (July and August long weekends) flows and peak I/I rates.

34 The 93 m long section of the trunk main in the side-yard SRW between Aberdeen and Temple is calculated to be operating at 100% capacity. The RDN design standard for this diameter of pipe is 50%.

OCP Build-Out

City Owned Infrastructure

35 At OCP Build-Out, the results of the computer model which was developed using the City's GIS database and some record drawings, two areas within the City are calculated to have undersized mains when conveying the peak design flow.

- A 147 m length of the Craig Bay foreshore main west of the pump station is calculated to flow at 94% full during peak flow.
- The 380 m section of Corfield Rd, north of Stanford Ave is calculated to flow more than 100% full during peak flow. The main should be upsized to 250 mm diameter subject to confirmation of the available grade.

RDN Owned Infrastructure

36 The gravity trunk main leading to the Bay Ave pump station, the pump station, and the gravity main downstream of the station are calculated to be operating above the design capacity when conveying the peak design flow which includes the peak tourist season (July and August long weekends) flow and peak I/I rates.

37 The trunk main in the sideyard SRW between Aberdeen and Temple is calculated to be operating at 100% capacity. The RDN design standard for 200 mm diameter pipe is 50%.

Infrastructure Conditions Verification, Computer Modelling & Capital Planning

38 Video inspection of all mains in the sewer 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 could 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
- Development of an I&I Management program
- 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 4 to 8 kms of pipe per year would result in the inspection of the entire 85 kms of pipe over a 10 to 20 year period.

I&I Management Plan

39 While I&I will always be a part of a sewer collection system, large I&I flows can result in the unnecessary oversizing of infrastructure (mains, pump stations, forcemains and sewage treatment plants) and the fees that the City pays to the RDN for the conveyance, treatment and discharge of sewage.

40 The findings of the flow monitoring program supports the development and implementation of an I&I management plan.

Peak Design Sewage Flows

41 As noted in section **8.1 Historical & Design Flows**, the design flows modelled are a conservative approach to the system analysis as they are based on full occupancy of the City's tourism accommodation coinciding with a peak I&I allowance. Peak tourism occupancy occurs around the long weekends in the beginning of July, August and September. This conservative peak flow is not likely to occur on a regular or sustained basis.

150 mm Diameter Mains

42 According to the City's GIS database, almost 14 kms (16%) of the more than 84 kms of pipe making up the City's distribution system is 150 mm diameter pipe or less, which does not meet the City's current minimum design standard of 200 mm diameter.

12 RECOMMENDATIONS

Based on the conclusions reached in this report, it is recommended that the following be carried out in the order listed:

Engineering Design Standards Update

1 Update the Engineering Design Standards using the bold items noted in **Table 11**.

Infrastructure Conditions Verification,

2 Carry out a video inspection of 4 to 8 kms of mains each year to confirm the current condition of each pipe, service connection and manhole. The findings are to be used in identifying system weaknesses and prioritizing of capital works.

I&I Management Program

3 The recommendations in the Technical Memorandum No SSFM (located in **Appendix D**) be carried out. Note recommendation 2 has been completed (see report in **Appendix C**).

4 Prior to issuing an occupancy permit on any structure that is connected to the sanitary sewer system, a cross connection test should be carried out on the storm and sanitary sewer connections, if not already doing so. During a dye test, City personnel should carry out a visual inspection at the nearest downstream manhole (and not at the inspection chamber at property line in case the services after the inspection chamber are cross connected). Each drainage fixture in the building should be checked including perimeter and roof leader drains.

5 Develop an up to date library of all existing video inspection reports to confirm which mains have been video inspected and for which the video inspection reflects current conditions, i.e., no upgrading works have been carried out since the time of the video.

6 Develop an annual video inspection program to be carried out during the winter months when I&I is the highest. The program should video a 4 to 8 kms of pipe each winter. This would result in the mains being video inspected once every 10 to 20 years. The video information (pipe diameter, material, service connection locations and diameters, pipe condition) should be checked against the City's GIS database and the sanitary computer model and both updated as necessary.

7 Operate and maintain the Community Park flow monitoring station. Carry out monthly inspections of the station and confirm it is recording correctly by manually measuring the depth of flow and comparing it against the recorded depth during the monthly inspection. Download and analyze the data monthly to establish seasonal trends and to identify unusual flows. Tabulate and graph daily flow vs rainfall for the Community Park station and include the flow from the RDN's Ocean Place flow monitor on the same graph. This flow monitoring data is to be incorporated into the City's I&I Management Plan

8 Develop an annual flow monitoring program to be carried out during the wet winter months to aid in identifying and eliminating I&I sources. The monitoring catchment area must be of an adequate size such that the probe's minimum flow depth requirement is met. Carry out smoke and dye testing to locate inflow

sources. Carry out video and manhole inspections to identify inflow and infiltration sources.

9 Calculate I&I rates for the catchment areas upstream of the flow monitoring site. After I&I reduction works are completed for the catchment, re-monitor the site to document the extent of reduction in I&I.

10 Carry out smoke testing on the storm drainage system to identify cross sections starting with the newest developed areas.

Pump Station Flow Monitoring

11 Install a flow meter on the forcemain of the Craig Bay and Martindale pump stations. Flows should be recorded at the same time and interval as the Community Parks flow monitoring site (5 minute increments). The data should be downloaded monthly and analyzed (tabulated and graphed) to identify trends; confirm the extent of I&I in this part of the system; and to refine resort tourism design flow allowances.

Upgrading Work

12 Complete the design for the upgrading work on Corfield St between Jensen Ave and Stanford Ave (Project 2 in **Table 16**).

13 Monitor the flows on the 147 m section of the Craig Bay foreshore trunk main to confirm the need of this project.

14 Update on a yearly basis the list of capital improvements to reflect the findings of the video camera program.

15 Assess the condition of all 150 mm and smaller mains and add to the capital improvement list, if warranted.

16 Carry out repair works on the 12 problem pipes and two manholes identified by City Public Works Staff.

DCC Update

17 Update the City's Sanitary Sewer DCCs to include the projects listed in **Table 16**.

RDN Collaboration

18 Consult and coordinate with the RDN the monitoring of the infrastructure they own and operate within the City.

19 Establish a means by which to jointly share and review monitoring results and schedule potential improvements. It is anticipated that upgrading works on RDN infrastructure will be the responsibility of, and will be paid by, the RDN.

20 If not already done so, the City dialogue with the RDN regarding the need for repair work on the foreshore armouring (rip-rap) at the Bay Avenue pump station.

21 If not already in place, ensure there is a process by which the City is made aware of and has input on proposed land-use changes beyond its boundaries that have an impact on future loading conditions of City owned infrastructure. Most notable would be in the areas to the east and south of the City, RDN Areas E and F, where the current OCPs do not anticipate the development of community sewer systems that would connect to the City of the Parksville.