KERR WOOD LEIDAL
consulting engineers

Vancouver Island
201 - 3045 Douglas Street
Victoria, BC V8T 4N2
T 250 595 4223
F 250 595 4224

Sanitary Sewer Master and Asset Renewal Plan

Final Report
December 2016
KWL Project No. 2148.012-300

Prepared for:

District of North Saanich

NORTH SAANICH

Contents

Figures

Tables

Appendices

Executive Summary

The District of North Saanich (the District) contracted Kerr Wood Leidal Associates Ltd. (KWL) to develop a sewer master plan and a prioritized asset replacement and upgrading program based on system condition, and current and forecasted capacity issues. Sanitary sewer system capacity is measured as the system's ability to convey existing and future peak wet weather flows without resulting in system surcharging or overflows.

A list of priorities and funding requirements for long term sustainable management of the sanitary sewer system was developed.

System Condition Assessment

System condition assessment in this study was inferred from the pipe material and age information provided by the District through the GIS database. No field assessments where completed using methods such as CCTV inspection.

The gravity piping material is comprised of approximately 13% AC pipe with an average age of 40 years and 87% PVC pipe with an average age of 17 years. The forcemain piping material (pressure pipe) is comprised of 89% PVC or HDPE piping and has an average age of 12 years.

The District owns eleven sewage pumping stations. A sewage pump station condition assessment was not completed as part of this project; however District staff identified required upgrades to the electrical controls at Amity, Trincomali, and Bazan Bay pump stations.

Inflow and Infiltration (I&I) Assessment

I&I analysis was conducted for the Reay Creek pump station catchment, using "RDI&I Envelope" method. The analysis derived I&I flows and area-based rates associated with 5-year and 25-year return period storms, based on rainfall statistics from the Victoria Airport rain gauge operated by Environmental Canada. A 5-year Chicago design storm was generated for modeling RDI&I using the three triangular unit hydrograph (RTK) method. Modeling of design RDI&I was carried out by adjusting the fraction of rainfall (R parameter), that becomes RDI&I, until the modelled peak RDI&I was reasonably representative of the calculated design peak RDI&I for the Reay Creek pump station catchment.

A 5-year peak 1-hour I&I was selected for PWWF modelling in order for capacity assessment. Calculated RDI&I rates are below those established for the CRD Core Area, as such an I&I reduction program is not warranted at this time.

Capacity Assessment

The sanitary sewer model was developed using the PCSWMM platform based on the sanitary infrastructure GIS data, with existing and future development loadings. The model calibration exercise yielded an average per-capita sanitary loading rate of 110 L/cap/day, lower than the typical range observed in the capital region (160 to 225 L/cap/day). Upon discussion with the District, both the existing and future development scenarios were run using a slightly conservative rate of 185 L/cap/day, which was deemed appropriate to serve the purpose of this study. The modeled dry weather flows (at 185 L/cap/day) were compared against the flow monitoring data collected at the Amity, Reay Creek, and McDonald Park pump stations to confirm the appropriateness of the model settings.

Hydraulic performance analysis of the gravity sewer system was conducted based on the model results using a Hydraulic Level of Service rating assessment method. The identified locations with potential hydraulic concerns were further reviewed by examining hydraulic grade line profiles to determine the severity of impact on servicing. The analysis indicated that all the gravity sewers are capable of conveying existing and future peak wet weather flows under a design five year return period storm event, with a couple of locations being identified as nearing full pipe capacity under future conditions. Upgrades to these locations are not recommended at this time.

The pumping systems together with the equipped storage facilities generally have capacities to accommodate existing and future peak wet weather flows, with the exception of the Reay Creek Pump Station. Under the future development scenario, the peak flow at the Reay Creek PS is currently predicted to exceed the pump station's regulated capacity (38 L/s) by about 2 L/s, implying additional storage might be required to handle the peak flow. Options to accommodate the potential flows at the Reay Creek PS could include reducing I&I to free up capacity at the Reay Creek PS.

The majority of pump station forcemains have velocities between 0.76 m/s and 3.0 m/s, except for the Mills Road Pump Station forcemain and the forcemain that receives discharges from BC Ferries. Periodical flushing of the forcemains at a higher flow rate is recommended to prevent solids buildup.

Upgrades and Long-Term Renewals Funding

Based on the results of the hydraulic modeling, projects targeting system capacity improvements were not identified as being required in the current planning horizon. A long term funding strategy was developed which recommended:

CCTV Inspection Program

- A CCTV inspection program is recommended to aid in determining the current condition of the system and to assist the development of AC main rehabilitation projects. The inspections might be repeated on a 10 year basis for AC gravity piping and a 20 year basis for PVC gravity piping.
- Annual funding may be on the order of:
  - $5,000 per year for AC gravity piping; and
  - $15,500 per year for PVC gravity piping.

AC Gravity Main Rehabilitation

- In the absence of a condition assessment for the existing AC mains, the funding strategy is based on rehabilitating the entire AC system over a period of 50 years starting in 2025 via relining. Actual rehabilitation would be a function of the needs identified as part of the condition assessment program (CCTV inspection).
- An estimate of annual funding for AC rehabilitation commencing in 2025 is:
  - $65,000 per year.

Pump Station Upgrades

- The proposed annual funding would provide funds for the known near term upgrades as well as finance other long term pump station upgrades.
- An estimate of annual funding for pump station upgrades:
  - $30,000 per year.

Overall Annual Funding

- Based on the above the following overall annual funding is estimated:
  - $50,500 annually from 2017 to 2025 for CCTV inspections and pump station upgrades; and
  - $115,500 annually commencing in 2025 for CCTV inspections, pump station upgrades, and AC gravity main renewals.

1. Introduction

1.1 Terms of Reference

The District of North Saanich (the District) hired Kerr Wood Leidal Associates Ltd. (KWL) to develop a complete sanitary sewer system model including existing and future development scenarios in order to develop a long term upgrade and renewal plan. The model results along with age and condition information are used for this purpose.

Prior to this assignment, KWL had completed a number of sanitary sewer planning and analysis studies for the District of North Saanich (the District) since 2013. These various studies had looked at approximately half of the system components in total and were used for the development of the model.

The project was divided into two phases:

- Phase 1 is the hydraulic capacity component of the project; and
- Phase 2 is the renewal plan.

1.2 Study Objectives

The objective of this study is to gain a detailed understanding of the sanitary sewer system and the District's current and future needs for maintaining levels of service. This is accomplished through the following project deliverables:

1. Memorandum requesting data for the model build.
2. I&I envelope figures.
3. Hydraulic level of service figures.
4. Draft and final hydraulic evaluation.
5. PCWMM model with existing and future scenarios.
6. Class 'D' cost estimates for renewals including summary into funding cycles.
7. Draft and final asset renewal report.

1.3 Abbreviations

The following abbreviations have been used throughout the report.

ADWF	Average Dry Weather Flow
AC	Asbestos Cement
BSF	Base Sanitary Flow
ca	Capita (Person)
GWI	Ground Water Infiltration
ha	Hectare
HLoS	Hydraulic Level of Service
HP	Horsepower
ICI	Industrial, Commercial and Institutional
PE	Population Equivalent
PWWF	Peak Wet Weather Flow
PDWF	Peak Dry Weather Flow
$PWWF_5$	Peak Wet Weather Flow associated with a 5-year return period storm
$PWWF_{25}$	Peak Wet Weather Flow associated with a 25-year return period storm
RDI&I	Rainfall Dependant Inflow and Infiltration
SF	Single Family
VAA	Victoria Airport Authority

2. Hydraulic Model Development (Sewer Modelling)

PCSWMM (a product of CHI) was selected as the modeling platform for this study. This software uses USEPA (The United States Environmental Protection Agency) SWMM5 as the computational engine (dynamic wave), and includes the most up-to-date GIS capabilities. It is capable of handling sophisticated hydraulic conditions in a mixed gravity and pressure hydraulic network, with dynamic effects of flow routing, attenuation, backwater, as well as complex pumping.

2.1 Infrastructure Model

The PCSWMM model was built based on the sewer infrastructure GIS data provided by the District. The GIS source data is generally of reasonably good quality, with some missing pipe inverts and sizes for gravity sewers (about 2% of total gravity main sections). In order to ensure the accuracy of the model, the GIS data was thoroughly examined and a number of elements were updated with assumptions that are normally accepted in general practice. The modifications made to those elements are described in the model data field "Note".

Through a data mapping process, the sewer network GIS source data was transferred to the model database. There are several key physical attributes that are required for hydraulic modeling. For pipes this includes invert elevations, diameter and length. For manholes the required data are rim elevation and diameter. Fields not mapped to the model database, and required by the model software were entered within the model environment. These include:

- Pipe roughness coefficient – 0.011 (Manning's 'n') for PVC gravity; 0.013 (Manning's 'n') for AC gravity; or 120 (Hazen-Williams 'C') for forcemain; and
- For modelling purposes, all the manholes were assumed to have a bolted cover to prevent loss of water from the hydraulic model. The surcharge depth was artificially set to be 99 m for all manholes and junctions.

Missing manhole rim elevations were assigned based on the available 1-m LiDAR data. Where pipe inverts were not available because the GIS inverts were incorrect or missing, reasonable assumptions were made to fill in those data gaps. The methodologies used to calculate pipe inverts include:

- Assuming pipe slope: this method was commonly used at the very top run of a sanitary sewer, where the upstream end is a cap or cleanout. In this case, a slope of 0.5% was assumed to calculate the pipe upstream invert;
- Using manhole inverts for the connected pipe inverts;
- Linearly interpolating between upstream and downstream known inverts if the pipes are continuous sections;
- Using upstream pipe end inverts and downstream pipe start inverts if the pipe is a single section; and
- Assuming pipe inverts at forcemain chambers (generally bends, valves, etc.) are two meters below ground for all forcemains and low pressure mains.

Pump stations and wet wells were manually created in the model, together with their pump performance curves, control settings, and wet well dimensions. Eleven pump stations were included in the model. All of the pump stations are in a duplex arrangement (1 duty pump and 1 on standby). The pumps were modelled to operate at their full speed (except for the Mills Road pump station, where a reduced speed pump was modeled as advised by the District) and are controlled by wet well level settings. For the Mills Pump Station, a constant pumping rate of 19 L/s was modelled. Table 2-1 provides a summary of the District's sanitary pump stations.

Table 2-1: Summary of Sanitary Pump Station and Forcemain

Pump Station	Pump Model	Arrangement	Wet Well Top Elev. (m)	Wet Well Size (m)	Wet Well Invert (m)	Lead Pump Start Level (m)	Lag Pump Start Level (m)	Stop Level (m)	Forcemain Diameter (mm)	Forcemain Material	Forcemain Length (m)	Forcemain Install Year
Amity	Flygt NP 3153-453MT, 18 Hp	Duplex	7	2.4 m x2.4 m	2.8	1.46	1.66	1.25	150	PVC	383	2001
Bazan Bay	Flygt NP 3102-462MT, 5 Hp	Duplex	9	2.4 m x2.4 m	2.2	1.2	1.35	0.85	100	PVC	270	2001
Cromar	Flygt CP 3127-433MT, 7.5 Hp	Duplex	10.75	1.82 m dia.	3.51	1.85	2.25	0.95	150	PVC	174	2005
Deep Cove	Flygt NP 3127-488HT, 10 Hp	Duplex	10.4	1.82 m dia.	4.15	1.1	1.5	0.7	100	PVC	306	2005
McDonald Park	Flygt NP 3153-462HT, 20 Hp	Duplex	5.13	1.82 m dia.	-2.11	1.6	1.7	1.1	150	PVC	2,371	2006
Mills	Flygt CP 3127-432MT, 10 Hp	Duplex	14.35	2.43 m dia.	5.34	2.4	2.8	1.8	250	HDPE	1,696	2005
Munro	Flygt NP 3171-275SH, 35 Hp	Duplex	6.95	1.82 m dia.	0.72	2	2.4	1.2	200	HDPE	1,519	2005
Reay Creek	Flygt NP3171-275SH, 35 Hp	Duplex	5.15	2.4 m x2.4 m	-0.55	0.95	1.1	0.35	200	PVC	14	2001
Towner	Flygt NP 3153-433MT, 20 Hp	Duplex	5.46	2.43 m dia.	0.05	1.5	1.9	0.9	200	HDPE	3,212	2005
Trincomali	Flygt NP 3153-454MT, 18 Hp	Duplex	81.923	2.4 m x2.4 m	75.82	1.4	1.6	1.05	150	PVC	626	2001
West Saanich	Flygt CP 3102-433MT, 5 Hp	Duplex	5.73	1.82 m dia.	-0.5	1.5	1.9	0.6	100	PVC	178	2005

There are a total of nine (9) air release valves in the District's sanitary pumping system. Although these air release valves were not hydraulically modelled in PCSWMM, an inventory of these valves is provided in Table 2-2 for the District's asset management purposes.

Table 2-2: Air Release Valve Inventory

Air Valve Model ID	Location	From Pump Station	Diameter (mm)	Installation Year
JCT6005	10364 McDonald Park Rd.	McDonald Park		
JCT6008	Calvin Ave east of Patricia Bay Hwy.	McDonald Park		
JCT6006	Northwest of Ocean Ave. & Stirling Way (In Airport)	Mills		
JCT6007	Near West End of Beacon Ave. (In Airport)	Mills		
JCT6010	930 Towner Park Rd.	Towner		
JCT6011	10629 Derrick Rd.	Towner		
JCT6012	844 Towner Park Rd.	Towner		
JCT6013	1340 John Rd.	Towner		
JCT6171	2075 Patricia Bay Hwy	From Swartz Bay connection		

2.1.1 Outfalls

The District's collection system has a total of 5 discharge (outfall) locations as follows:

- Outfall 1, is located at the end of forcemain from the McDonald Park PS and connects to the Sidney Peninsula Trunk;
- Outfall 2 is located at the end of the forcemain from the Reay Creek PS and connects to the Sidney Peninsula Trunk;
- Outfall 3 is downstream of SMH1233 which connects directly to Peninsula WWTP serving the north portion of Dean Park;
- Outfall 4 is located at the end of the forcemain from the Amity PS and connects to the Central Saanich Trunk; and
- Outfall 5 is downstream of SMH2184 connection to the Central Saanich Trunk serving the south portion of Dean Park.

Figure 2-1 shows the outfall locations and their associated sewer catchment areas. Table 2-3 provides a summary of the modeled boundary conditions for each outfall.

Table 2-3: Modeled Outfall Boundary Conditions

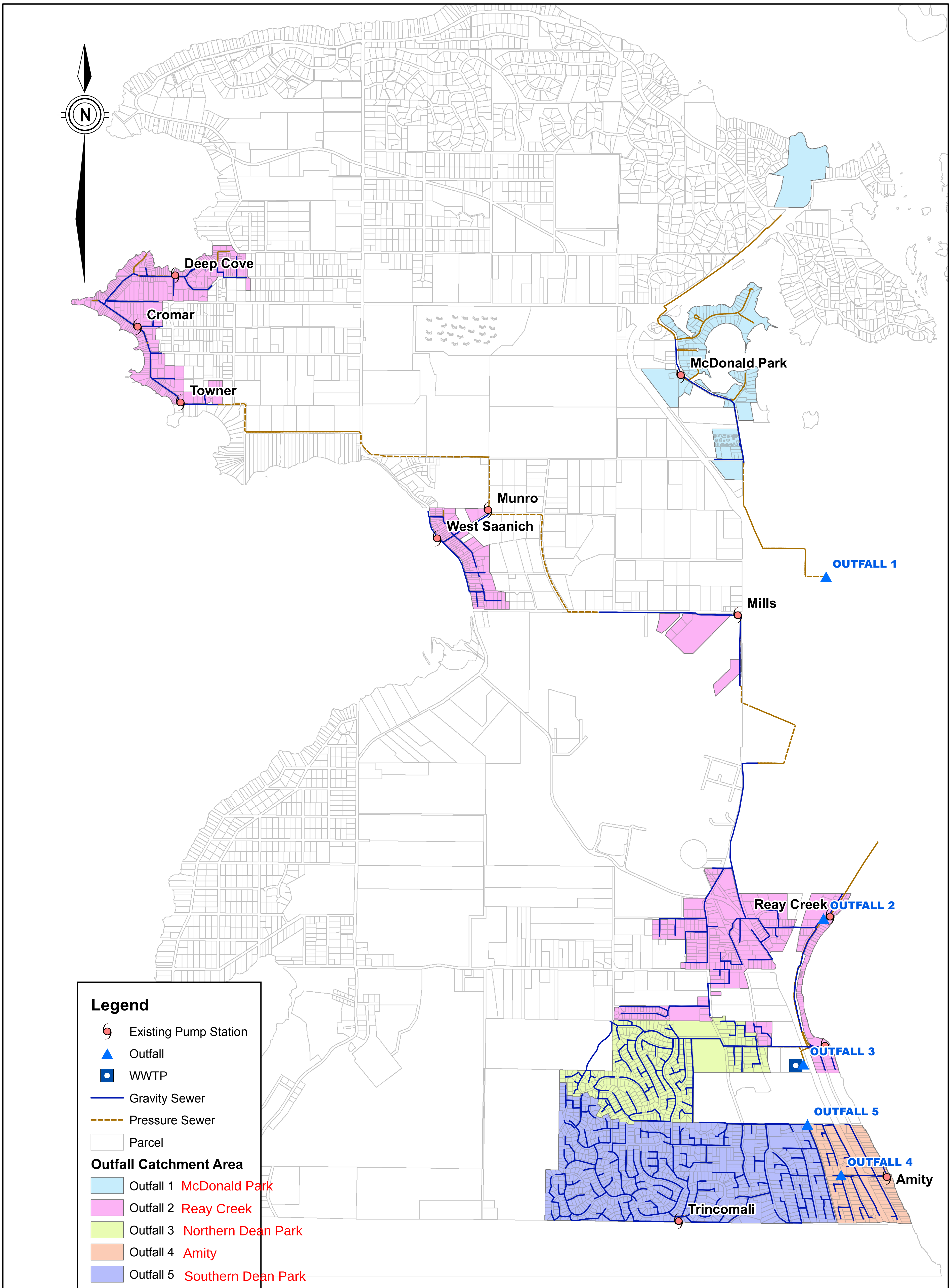
Outfall Location	Outfall Model ID	Modeled Boundary Condition	Comments
Outfall 1 – McDonald Park	SMH5024	Free Outfall	To Sidney Peninsula Trunk
Outfall 2 – Reay Creek	OUT_2	Free Outfall	To Sidney Peninsula Trunk
Outfall 3 – Northern Dean Park	OUTLET_3	Free Outfall	Directly to Peninsula WWTP
Outfall 4 – Amity	JCT6133	Free Outfall	To Central Saanich Trunk
Outfall 5 – Southern Dean Park	JCT6168	Free Outfall	To Central Saanich Trunk

2.1.2 CRD Design Flow Allocations

The following summarizes the breakdown of the District's total peak design flow allocation of 107.6 L/s (9,300 m^3/day) to the Peninsula Treatment Plant based on Schedule A to CRD Bylaw 2439, refer to Appendix C.

- North Saanich Peninsula Trunk – 34.7 L/s (3,000 m^3/day);
- Sidney Peninsula Trunk – 56.7 L/s (4,900 m^3/day); and
- Central Saanich Peninsula Trunk – 16.2 L/s (1,400 m^3/day).

Path: Q:\2100-2199\2148-012\430-GIS\MXD-Rp\Figure2-1_Sewer Outfall Location and Catchment.mxd Date Saved: 02/12/2016 3:46:33 PM
Author: hbai



Reference: Orthophoto from Bing aerial image map.

KERR WOOD LEIDAL
consulting engineers

Sanitary Sewer Master and Asset Renewal Plan

Project No. 2148-012

Date December, 2016

500 0 500 (m)
1:30,000

Sanitary Outfall Location and Catchment Area

Figure 2-1

2.2 Model Loading Development

KWL's methodology for loading sewer models uses each legal lot in the District's GIS cadastral dataset as an individual catchment. This ensures that nearly every pipe in the system receives some level of sanitary loading.

The serviced lots were connected with the sewer loading manholes primarily based on the service connection GIS data. Where the properties are serviced by the sanitary sewer system but service connection information is not available, a nearest feature GIS routine was conducted followed by a topography review to verify the reasonableness of the connections. Connections that were deemed incorrect were reassigned manually in GIS.

2.2.1 Populations

Sanitary loads for model input were calculated based on populations. For model development and hydraulic analysis purposes, only serviced populations were used for sanitary load calculation.

The District's Water System Master Plan study is being conducted by KWL concurrently. The existing population distribution, future population projection, and Industrial-Commercial-Institutional (ICI) equivalent population estimates based on land uses developed as part of the water mater plan study were used in this sanitary study in order to maintain consistency. The only exception is the Victoria International Airport (VAA)'s North Camp along Mills Road. The populations from the Airport lands were taken from a previous servicing study for the Airport[1].

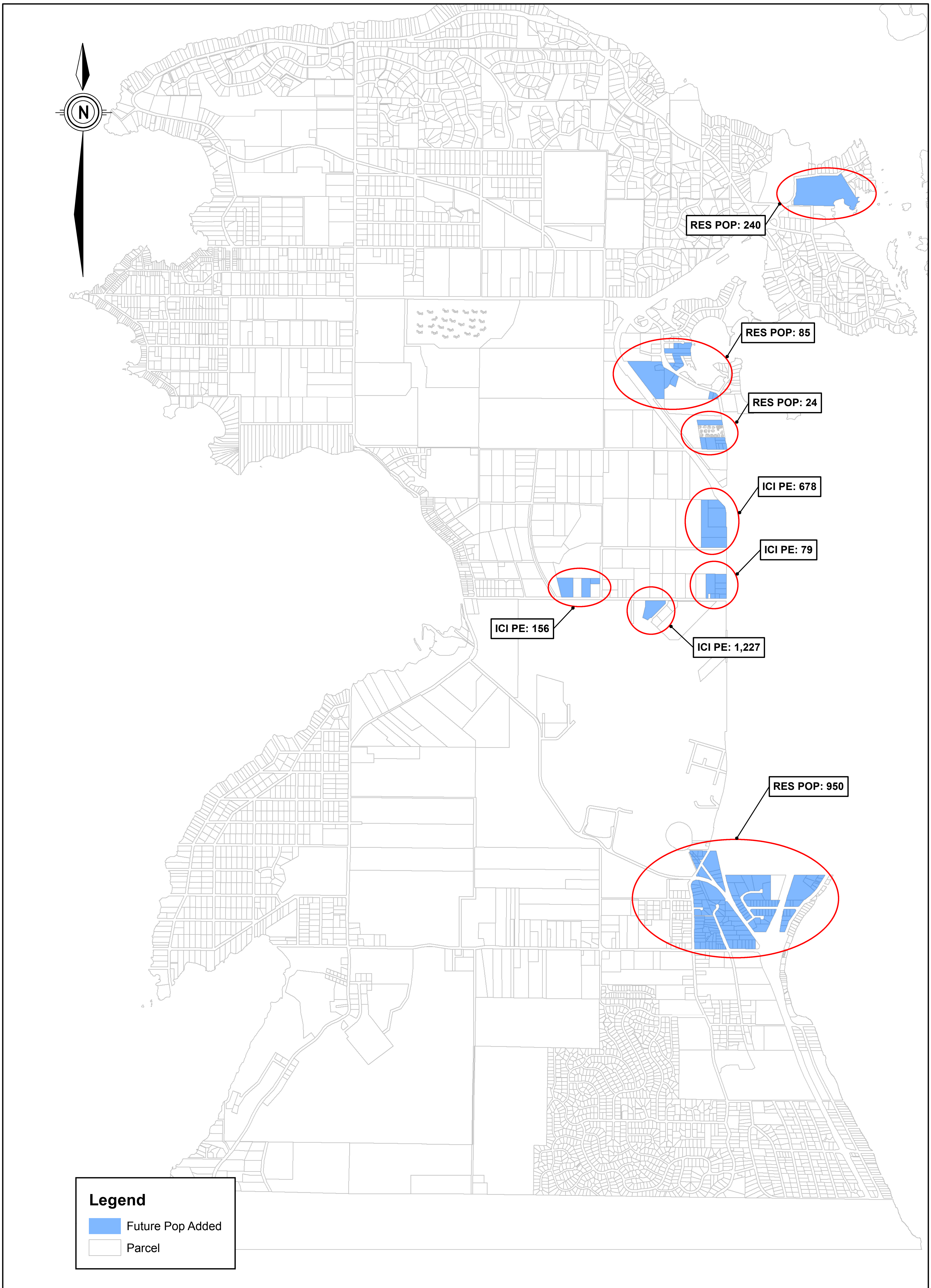
Based on the existing serviced parcels and additional parcels to be serviced by the sanitary system in the future, as advised by the District, existing and future serviced populations were developed, as shown in Table 2-4. Figure 2-2 displays where the future population was allocated. These areas were included in the analysis to account for potential future flows. The assumptions that were made in the analysis and presented in the report are not intended to imply development in these areas has or will be approved or that these areas will be added to the sewer servicing area.

Table 2-4: Existing and Future Populations

Land Use	Existing	Future	Note
Residential	5,466	6,766	Including Rideau, Brackman, McTavish, and Tsehum Development areas.
Commercial	134	817	Including Sandown development.
Industrial	663	1,931	Including VAA.
Institutional	627	815	
Total PE	**6,890**	**10,329**	

[1] Underground Services Condition Survey at Victoria International Airport, July 2014, KWL File 2083.009.

Path: Q:\2100-2199\2148-012\430-GIS\MXD-Rp\Figure2-2_Future_Pop_Allocation.mxd Date Saved: 02/12/2016 4:03:53 PM
Author: hbai



Reference: Orthophoto from Bing aerial image map.

KERR WOOD LEIDAL
consulting engineers

Project No. 2148-012

Date December, 2016

500 0 500 (m)

1:30,000

Sanitary Sewer Master and Asset Renewal Plan

Allocation of Future Additional Population

Figure 2-2

2.2.2 Base Sanitary Loading

Flow data derived from pump station SCADA records for the period from November 1, 2014 to April 30, 2015 for the Amity, McDonald Park, Mills Rd, and Reay Creek pump stations, together with rainfall data collected at the Sidney pump station rain gauge, were used for flow analysis in this study.

For the existing development scenario, the per-capita base sanitary flow (BSF) loading rate was calibrated using the measured sewage flows under dry weather conditions. Based on the rainfall data for the flow monitoring period, several dry weather periods were identified.

To calculate BSF, the baseline Groundwater Infiltration (GWI) was subtracted from the Average Dry Weather Flow (ADWF). GWI is typically determined by calculating 85% of the minimum nightly flow during a dry weather flow period which is free from Rainfall-Dependent Inflow and Infiltration (RDI&I) influence.

The inflows from BC Ferries are described in the McDonald Park Road Sewage Study[2]. The peak permitted flow from BC Ferries is 9.1 L/s, pumped from a 250 m^3 equalization tank, and, "will only occur when the McDonald Park Rd Pump Station's sanitary level is below the high alarm level". Therefore, as part of our analysis we have assumed an inflow of 9.1 L/s from BC Ferries outside of peak inflow times, with a minimum daily run time of 8 hours (10 p.m. to 6 a.m.) in order to fully empty the 250 m^3 equalization tank on a daily basis. Currently the observed flows from BC Ferries range from 5 to 7 L/s and are discharged for a 3 to 4 hour period daily which corresponds to an average daily volume of approximately 75 m^3.

Table 2-5 shows the calculated loading rate for each flow catchment. Figure 2-4 displays the flow monitoring catchments.

Table 2-5: Calculation of Sanitary Loading Rate

Catchment	Residential Population	ICI PE	Total PE	ADWF (L/s)	GWI (L/s)	BSF (L/s)	Per-cap Rate (L/Cap/d)	Note
Amity PS	341	0	341	0.69	0.19	0.50	126	
McDonald Park PS	582	46	627	0.97	0.20	0.77	106	Excluding BC Ferries tank discharge; assumed GWI
Mills PS	876	523	1,399	2.58	0.68	1.90	117	Including VAA North Camp
Reay Creek PS Gross	1,583	750	2,333	3.69	0.74	2.95	109	Including VAA North Camp
Reay Creek PS Net	706	227	934	1.11	0.06	1.05	97	
				Weighted Average			**110**	

The above table shows the calculated BSF loading rate is from 97 to 126 L/cap/day. These values are lower than the typical range observed in the capital region (160 to 225 L/cap/day). For reference, the loading rate in the District of Saanich is estimated to be 165 L/cap/day. Possible reason(s) for the low per-capita rate in North Saanich are:

- The existing serviced populations were over-estimated.

---

[2] McDonald Park Rd Sewage Study, Delcan Corporation, March 4, 2013.

- GWI was over-estimated by the industry standard 85% rule of thumb estimate and legitimate water use contributes to a larger portion of minimum nightly flow. This would seem plausible given that legitimate night use is commonly estimated to be between 2-4 L/property/hour in water loss studies and the relatively young age of the sanitary system.

To account for this discrepancy and upon discussion with the District, the model was run with a slightly conservative rate of 185 L/cap/day for existing and future design condition scenarios.

2.2.3 Flow Diurnal Patterns

A diurnal pattern specifies the shape of the base sanitary flow as a function of time of day. Several diurnal patterns have been used in the model, as explained in the following table.

Table 2-6: Diurnal Patterns

Pattern Name	Source
RES	Residential – Derived from “Reay Creek” weekday dry weather flow signals.
IND	Industrial – KWL stock pattern derived from other flow monitoring studies.
COM	Commercial – KWL stock pattern derived from other flow monitoring studies.
INST	Institutional – KWL stock pattern derived from other flow monitoring studies.
BCFERRIES_THEORETIC	BC Ferries Tank Discharge – Permitted discharge rate and operation hours.
GWI	Groundwater Infiltration – Set to be fixed, representing a constant distribution.

All of the patterns are plotted on Figure 2-3.

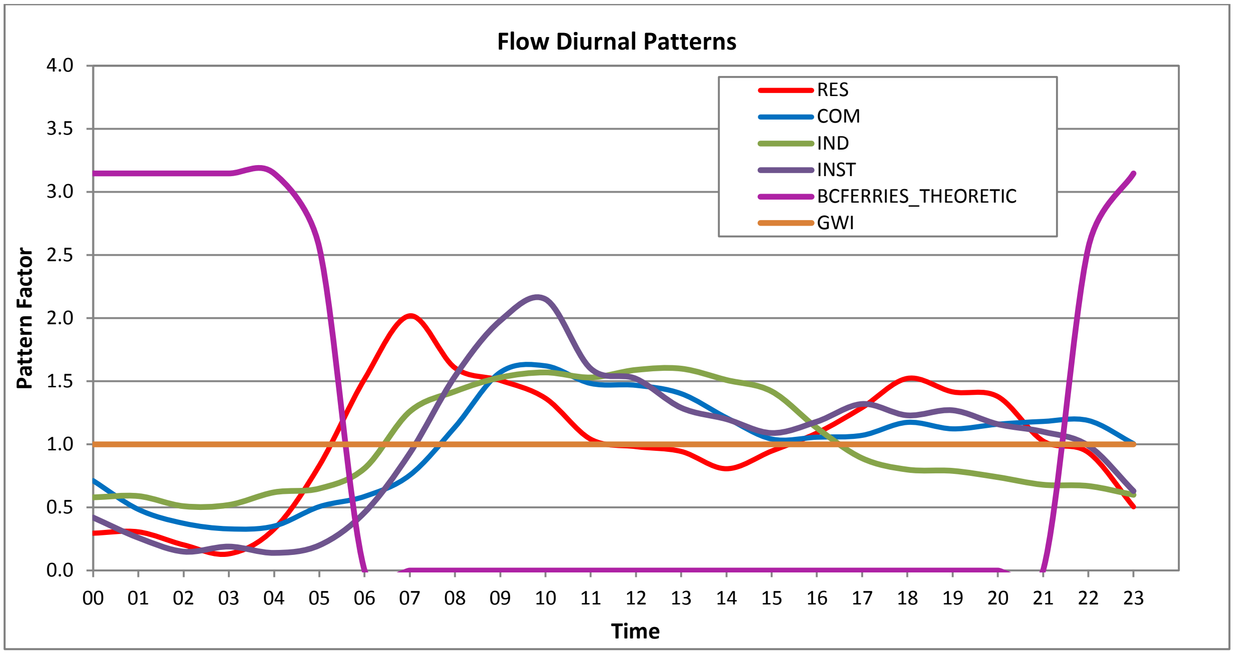


Figure 2-3: Flow Diurnal Patterns

2.2.4 DWF Calibration and Results

The Amity, McDonald Park, and Reay Creek pump station catchment dry weather flows were used to compare with the modeled flows, in terms of flow volume, peak flow, and peak timing. A special load was added into the model for modeling of BC Ferries' regulated discharge. A total discharge volume of 66 m^3 was estimated based on the flow signals at the McDonald Park pump station for dry weather flow calibration. (The total tank volume of 250 m^3 and a discharge rate of 9.1 L/s were used for wet weather flow modelling.)

As the model was run with a per-capita loading rate of 185 L/cap/day, it was expected that the modeled average flow and peak flow would be higher than the observed flows, by a range between 45 – 65 %, depending on the level of GWI contribution in each flow catchment.

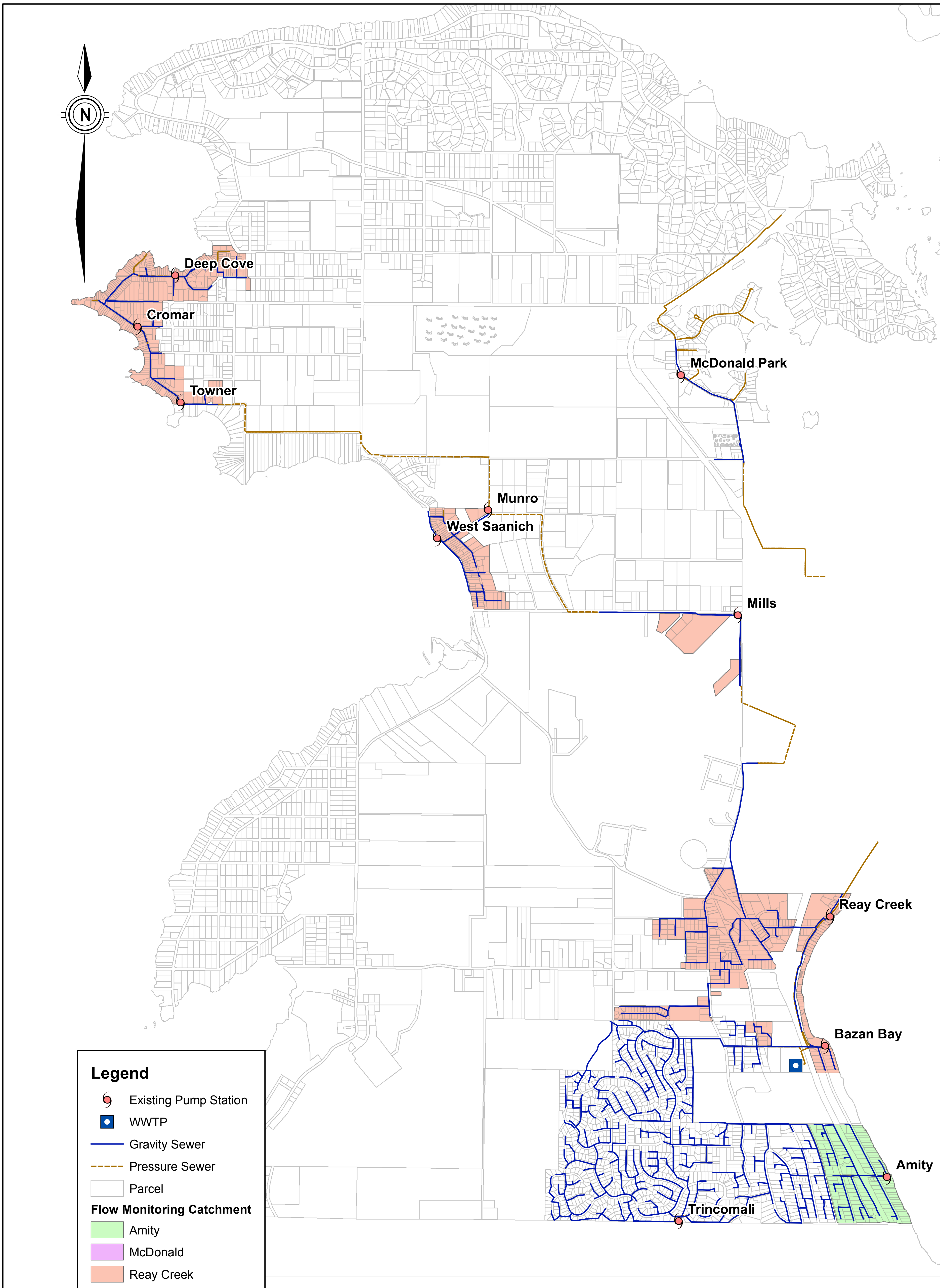
The dry weather flow comparison results are presented in Table 2-7. The dry weather flow comparison hydrographs for each site are attached in Appendix A.

Table 2-7: Weekday DWF Calibration Results

Site	ADWF (L/s)			PDWF (L/s)			Peak Timing
	Measured	Modelled	Difference	Measured	Modelled	Difference	
Amity PS	0.62	0.88	40.9%	1.09	1.58	44.8%	+/- 1 hr
Reay Creek PS	3.47	6.00	72.8%	6.11	10.30	68.6%	+/- 1 hr
McDonald Park PS	1.65	2.31	40.3%	5.97	6.20	3.9%	varies; +/- 1 hr typical

The above table shows that the modeled peak dry weather flows for the Amity and Reay Creek sites are close to the expected values. The average dry weather flow is slightly over-predicted for the Reay Creek site, while slightly under-estimated for the Amity site. For the McDonald Park site, due to the regulated discharge from BC Ferries (volume and peak), both the modeled average dry weather flow and peak dry weather flow are in the expected range when compared with the flow monitoring data. Therefore, through this verification process, the model is deemed to be a representative tool which can be used with reasonable confidence to perform hydraulic analysis of the sanitary system under different loading conditions.

Path: Q:\2100-2199\2148-012\430-GIS\MXD-Rp\Figure2-4_FlowMonitoringCatchment.mxd Date Saved: 02/12/2016 4:05:41 PM
Author: hbai



Reference: Orthophoto from Bing aerial image map.

KERR WOOD LEIDAL
consulting engineers

Sanitary Sewer Master and Asset Renewal Plan

Project No.	Date
2148-012	December, 2016

500 0 500 (m)
1:30,000

Flow Monitoring Catchment

Figure 2-4

3. Inflow and Infiltration (I&I)

This report uses a graphical method to estimate RDI&I which is based on a summary of rainfall and sewer flow events taken from the flow monitoring period. By plotting these results, the relationship between rainfall and RDI&I can be developed. It is then possible to develop 'return-period' design values for RDI&I, based on the rainfall analysis. KWL refers to this specific methodology as the "RDI&I Envelope".

In this study, the flow monitoring data from the Reay Creek pump station was used to calculate I&I rates. The derived I&I rates were then applied to the entire sewerage area for the model I&I loading.

3.1 Return Period and Duration

Values for I&I are usually presented with a statement on the rainfall return period and the duration over which the I&I is averaged. The reason is that different applications require different return periods and durations. Two of these are described below.

3.1.1 5-Year, 24-Hour I&I Rates

The 5-year, 24-hour I&I rates provide the peak I&I rate averaged over a 24-hour duration, based on a 5-year return period storm. This represents a suitable return period and duration for comparing the relative I&I severity between catchments, both within the study area in North Saanich as well as for comparison with other municipalities. The Municipal Wastewater Regulation (BC) states that "A discharger must ensure that an overflow does not occur during storm or snowmelt events with a less than 5-year return period" (Division 2, item 42). This requirement can be exempted if the party responsible for the municipal wastewater collection system develops a liquid waste management plan, and develops and implements measures to eliminate overflows. The use of 24-hour duration values is also more appropriate given some of the larger catchment sizes, and the period frequency of storm used is consistent with the Municipal Wastewater Regulations.

3.1.2 5-Year/25-Year, Peak 1-Hour I&I Rates

These values are the I&I values averaged over 1-hour. The 5-year peak 1-hour I&I rates were used for hydraulic performance assessment. This is because the municipal sewer infrastructure is required to convey short duration peak flows rather than more attenuated flows (24-hour I&I) that are expected in the regional trunk sewers (e.g., CRD's interceptors). The 5-year return period frequency of storm is also consistent with the Municipal Wastewater Regulations thus is suitable for assessing capacity of existing infrastructure.

The 25-year return period is often chosen as a design value for I&I rates within a system. This is done to size the sanitary system marginally above most minor storm sewer systems that are designed to convey storms of up to 10 year return period. The 25-year, peak 1-hour I&I rate was reported in this study and can be used for sizing new infrastructure or infrastructure upgrades at the District's discretion.

3.2 I&I Analysis Procedure

In order to develop the I&I rates, the following process is followed:

- Determine an estimate for the GWI (85% of the minimum nighttime flow from dry weather flow period);

- Use RDI&I envelope method in order to make estimates of the 5-Year/24-Hour, 5-Year/Peak 1-Hour and 25-Year/Peak 1-Hour RDI&I rates; and
- Combine the RDI&I and GWI into total I&I rates.

3.3 Rainfall Intensity-Duration-Frequency Statistics

The rainfall Intensity-Duration-Frequency (IDF) relationship curves are based on rainfall data collected at the Victoria Airport rain gauge which is operated by Environment Canada (39 years of rainfall data from 1965 to 2005). The return period rainfall amounts (mm) are summarized in Table 3-1.

Table 3-1: Victoria Airport Rainfall Statistics (mm)

Duration	Return Period					
	2 year	5 year	10 year	25 year	50 year	100 year
1 hour	8.6	10.7	12.1	13.8	15.1	16.4
2 hours	13.5	16.7	18.8	21.5	23.5	25.4
6 hours	27.9	34.6	39.0	44.6	48.8	52.9
12 hours	41.2	52.2	59.5	68.7	75.6	82.4
24 hours	53.2	71.2	83.0	98.0	109.1	120.2

3.4 I&I Analysis Results

The RDI&I envelopes together with graphic demonstration of RDI&I response for selected storm events for the Reay Creek pump station catchment are attached in Appendix B.

Table 3-2 provides a summary of the calculated I&I values.

Table 3-2: Summary of I&I Rates

Catchment Gross Area	181.7	ha
GWI	0.7	L/s
5-Year 24-Hour RDI&I	15.6	L/s
5-Year 24-Hour Total I&I	16.3	L/s
5-Year 24-Hour I&I Rate	7,700	L/ha/day
5-Year Peak 1-Hour RDI&I	32.8	L/s
5-Year Peak 1-Hour Total I&I	33.5	L/s
5-Year Peak 1-Hour I&I Rate	15,900	L/ha/day
25-Year 24-Hour RDI&I	21.9	L/s
25-Year 24-Hour Total I&I	22.7	L/s
25-Year 24-Hour I&I Rate	10,800	L/ha/day
25-Year Peak 1-Hour RDI&I	44.0	L/s
25-Year Peak 1-Hour Total I&I	44.7	L/s
25-Year Peak 1-Hour I&I Rate	21,300	L/ha/day

3.5 Modelling of I&I for PWWF

The system performance under wet weather conditions was analyzed using a Runoff-Time-Recession modelling approach (Tri-triangular RTK method), where RDI&I is modelled after a rainfall-runoff hydrological process to represent a more realistic system response to storms. The hydrological modelling can also characterize RDI&I into rapidly, moderately, and slowly responding inflow components, providing the basis for asset and I&I management practices. The RTK modelling approach for I&I may result in a lower PWWF estimation than the peak-on-peak approach (where I&I has a constant distribution over the contributing catchment area), and therefore requires a prudent selection of design storm events to produce reasonable yet conservative PWWFs.

3.5.1 Design Storm

A 5-year 24-hour Chicago design storm was generated based on the IDF curve from Victoria Airport rainfall station, as shown on Figure 3-1. The calculated total 24 hour rainfall is 69.1 mm. The design storm was used in the model to simulate RDI&I in conjunction with domestic flow for a design flow condition.

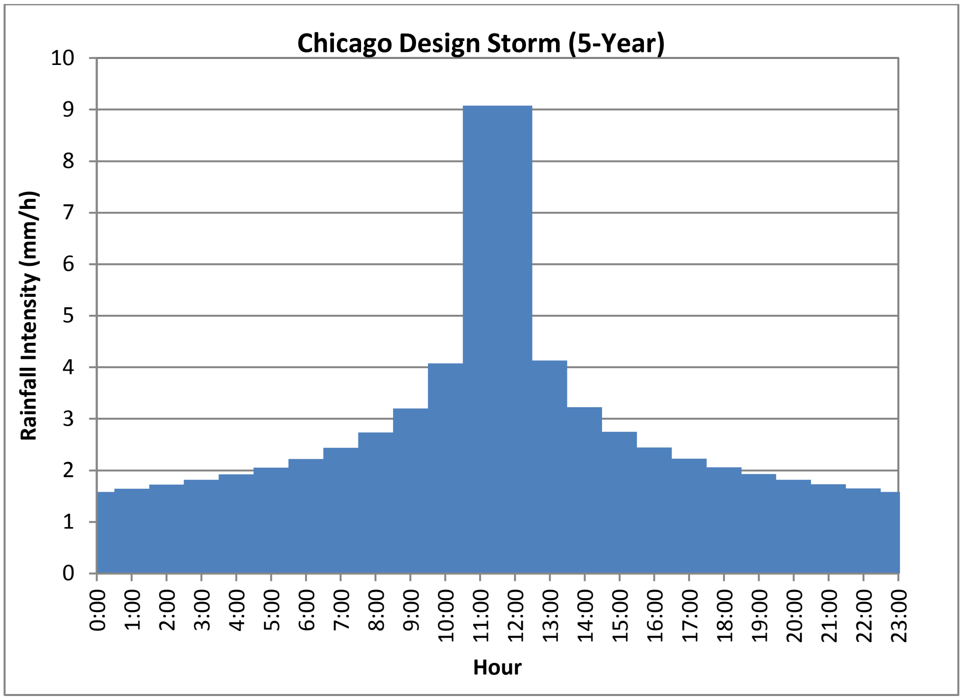


Figure 3-1: 5-Year 24-Hour Design Storm

3.5.2 RDI&I Generation

PCSWMM uses the three triangular unit hydrograph (RTK) method to determine the amount of inflow, fast infiltration, and slow infiltration measured in a sanitary sewer as a percentage of rainfall. R is the fraction of rainfall volume that enters the sewer system for the selected storm. T is the time from the onset of rainfall to the peak of the unit hydrograph in hours. K is the ratio of time to recession to the time to peak of the unit hydrograph (dimensionless).

Generally, determination of R, T, and K parameters requires extensive calibration efforts so that the modeled RDI&I can match the measured RDI&I as a result of a storm event, in terms of RDI&I flow volume, peak RDI&I, peak timing, and hydrographic shape of RDI&I. As a full calibration of R, T, and K parameters is beyond the scope of this assignment, KWL developed a simplified method as described below:

- Review RDI&I response to storms based on historical data collected at Reay Creek Pump Station to determine its general characteristics, then select a set of R, T, and K parameters that represent similar characteristics from KWL's previous projects as default values;
- Run model with the 5-year 24-hour design storm and determine the peaking timing of RDI&I relative to the peak timing of DWF at Reay Creek Pump Station;
- Make adjustments to the rainfall input by shifting the timing of rainfall so that the two peaks are approximately superimposed; and
- Adjust parameter R and run model until the resulting peak RDI&I at Reay Creek Pump Station matches the projected 5-year 1-hour RDI&I (32.8 L/s) from the RDI&I Envelope.

Table 3-3 displayed the assumed R, T, and K parameters used in the model. Figure 3-2 shows a plot of each unit hydrograph and summation of three unit hydrographs, resulting from a unit rainfall (i.e., 1 mm/hr).

Table 3-3: Assumed RTK Parameters

Component Set	R (fraction of total rainfall)	T	K
Short-term	0.004	1	1.5
Medium-term	0.01	5	1.8
Long-term	0.006	12	2.5

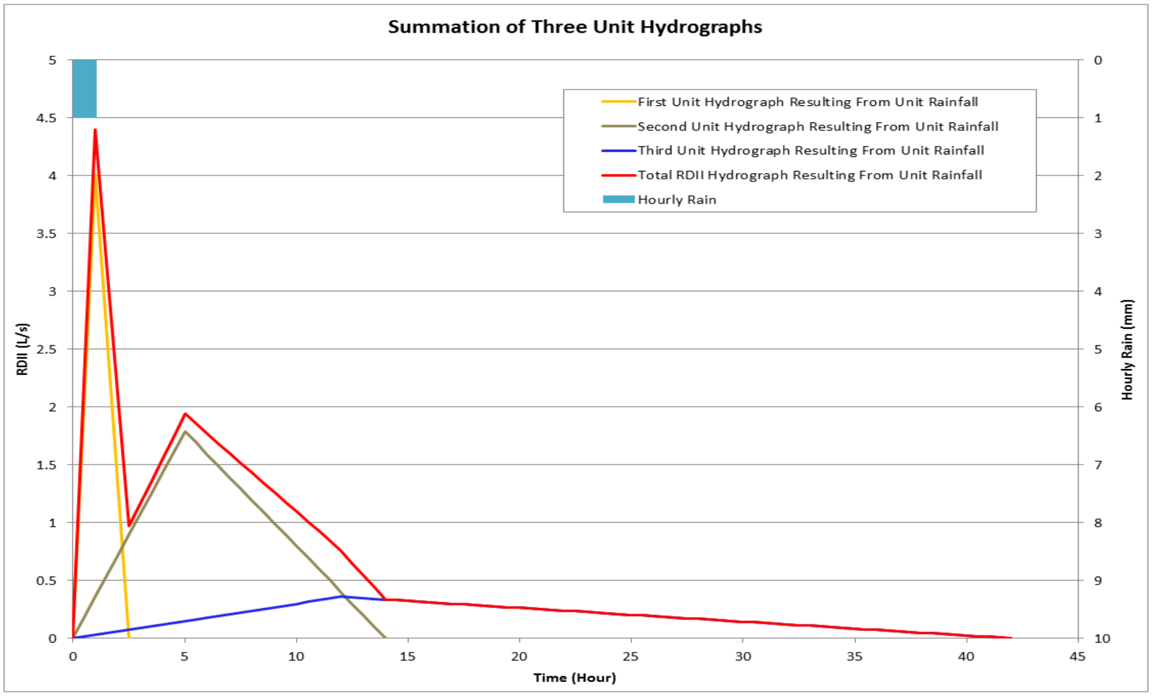


Figure 3-2: Summation of Three Unit Hydrographs

Figure 3-3 displays the modelled existing PWWF at Reay Creek Pump Station.

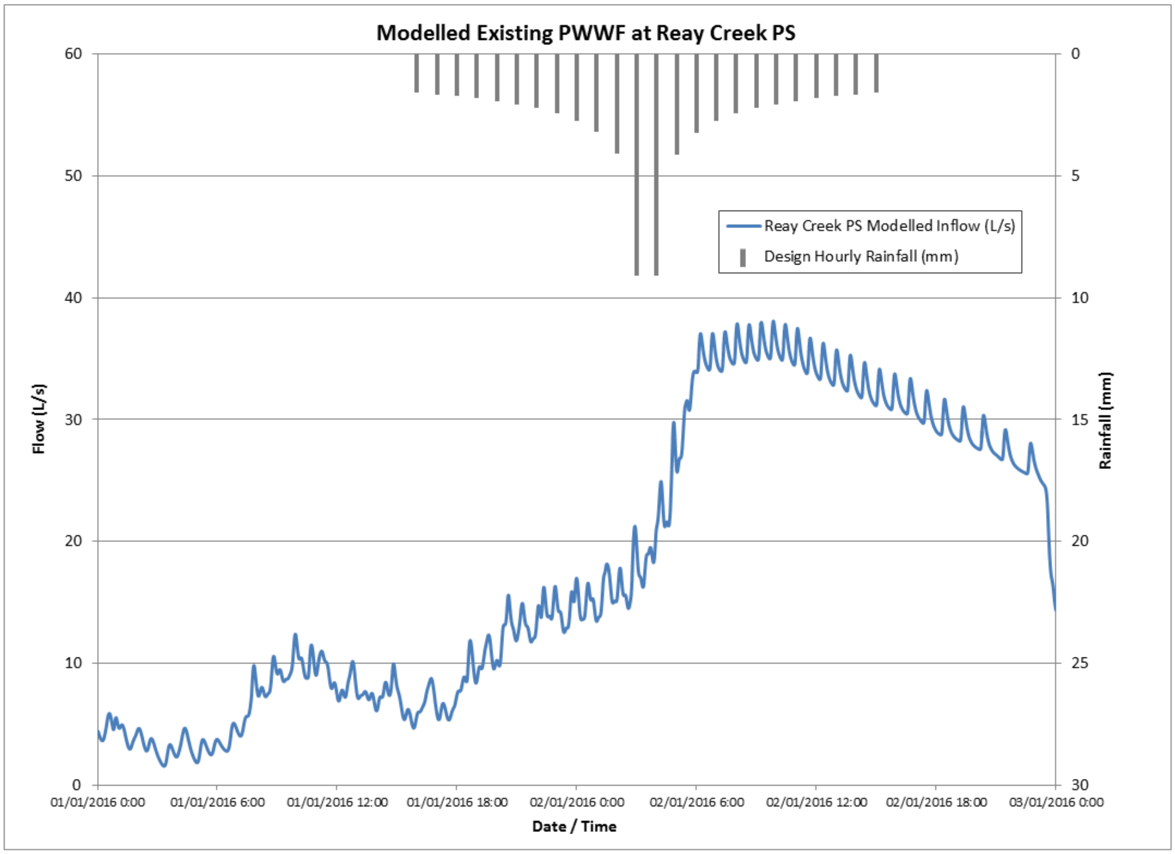


Figure 3-3: Modelled Existing PWWF at Reay Creek Pump Station

4. Hydraulic Performance Analysis

4.1 Hydraulic Assessment Criteria

Flows used for analyzing system capacity are peak wet weather flows with 5-year peak 1-hour I&I ($PWWF_5$) calculated by the hydraulic model. Capacity deficiencies resulting from these flows at the future development level identify potential system upgrades which may be required in the future.

4.1.1 Gravity Sewers

To assess the system hydraulic performance, a 'Hydraulic Level of Service' (HLoS) rating system was used as a preliminary assessment of the collection system review. Based on the results of this initial screening, hydraulic grade lines are analyzed in detail for any HLoS rating of D or lower.

The HLoS system assigns a rating to each pipe based on three criteria categories:

- Hydraulic Capacity – d/D ratio and/or friction slope in surcharged pipes;
- Hydraulic Grade Line – height of HGL with respect to pipe crown and ground elevation; and
- Velocity – whether minimum scouring velocity is achieved at peak flow.

Table 4-1 describes the HLoS criteria for each pipe class. A score of 1 to 3 is assigned to each criterion, with 1 indicating adequate performance, 2 indicating marginal performance and 3 indicating a failure condition.

Table 4-1: Hydraulic Level of Service Criteria Scoring

Rating	Score
Hydraulic Capacity	
d/D <= 0.7	1
d/D < 1.0	2
d/D = 1.0	3
HGL	
HGL < Crown	1
HGL <= 0.3 m above Crown	1
HGL <= Ground Elevation	2
HGL > Ground Elevation	3
Velocity	
v < 0.6 m/s	Fail
v >= 0.6 m/s	Pass

A letter-grade indicating the HLoS rating is assigned based on the above criteria scores. The letter grades are described in the following table.

Table 4-2: Hydraulic Level of Service Ratings

Grade	Capacity	HGL	Velocity	Description
A	1	1	Pass	Pipe performing as designed.
B	1	1	Fail	Adequate capacity, low velocity may indicate potential sedimentation.
C	1	2 or 3	N/A	Adequate capacity, downstream condition causing backwater.
D	2	N/A	N/A	Nearing pipe full condition.
E	3	2	N/A	Pipe full condition exceeded, no over flow expected.
F	3	3	N/A	Capacity exceeded and potential risk of overflow.

In general, ratings A-C will not trigger a further detailed review as there is capacity in the pipe to convey flows. A rating of D, E, or F indicates potential deficiencies in the system and requires further investigation and review of detailed hydraulic grade lines.

4.1.2 Pressurized Systems

Pump Stations

The pump stations were assessed for capacity by comparing the peak inflow rate with the modelled pumping capacity (firm capacity). The firm capacity is the capacity of a pumping station with the largest pump out of service or on standby.

If the peak inflow rate was greater than the pump firm capacity, the pump station was identified as not having adequate capacity and requiring capacity upgrade. This was also verified if all pumps turn on under peak flow conditions.

Forcemains

The forcemains were examined for both low and high velocities.

In general practice, forcemain velocities between 0.76 m/s to 3.0 m/s are deemed acceptable. The purpose of the minimum forcemain velocity criteria is to maintain minimum scouring velocity to reduce build-up in the forcemain. Exceeding the maximum velocity could indicate the forcemain is undersized, that there is excessive power consumption, or that higher pressure rating requirements are needed for the pipe.

Forcemain upgrades are dependent upon the requirements of the pump station upgrade, either for hydraulic capacity or maximum rated pressure.

4.2 System Performance Analysis

The model was run at a BSF rate of 185 L/cap/d plus I&I associated with a 5-year return period frequency of storm for both the existing and future development scenarios for hydraulic performance assessment of the existing sewer system.

4.2.1 Total Outflow

The District's wastewater is discharged into the CRD's sanitary system at several locations. Table 4-3 provides a summary of the modeled outflows at different discharge locations.

Table 4-3: Summary of Model Outflow

Outfall Location	Outfall Model ID	Existing			Future		
		ADWF (L/s)	PDWF (L/s)	$PWWF_5$ (L/s)	ADWF (L/s)	PDWF (L/s)	$PWWF_5$ (L/s)
Outfall 1 – McDonald Park	SMH5024	2.3	6.2	16.5	5.3	11.7	16.4
Outfall 2 – Reay Creek	OUT_2	7.0	27.8	36.1	12.6	29.1	39.8
Outfall 3 – Northern Dean Park	OUTLET_3	3.1	5.4	18.1	3.2	5.5	18.1
Outfall 4 – Amity	JCT6133	0.9	1.7	6.6	0.9	1.8	6.6
Outfall 5 – Southern Dean Park	JCT6168	5.0	10.3	35.9	5.7	10.6	36.0

The following table provides a comparison of the CRD allocated flows to the modeled future $PWWF_5$ flows.

Table 4-4: Comparison of CRD Sewage Capacity Allocation to Modeled Flows

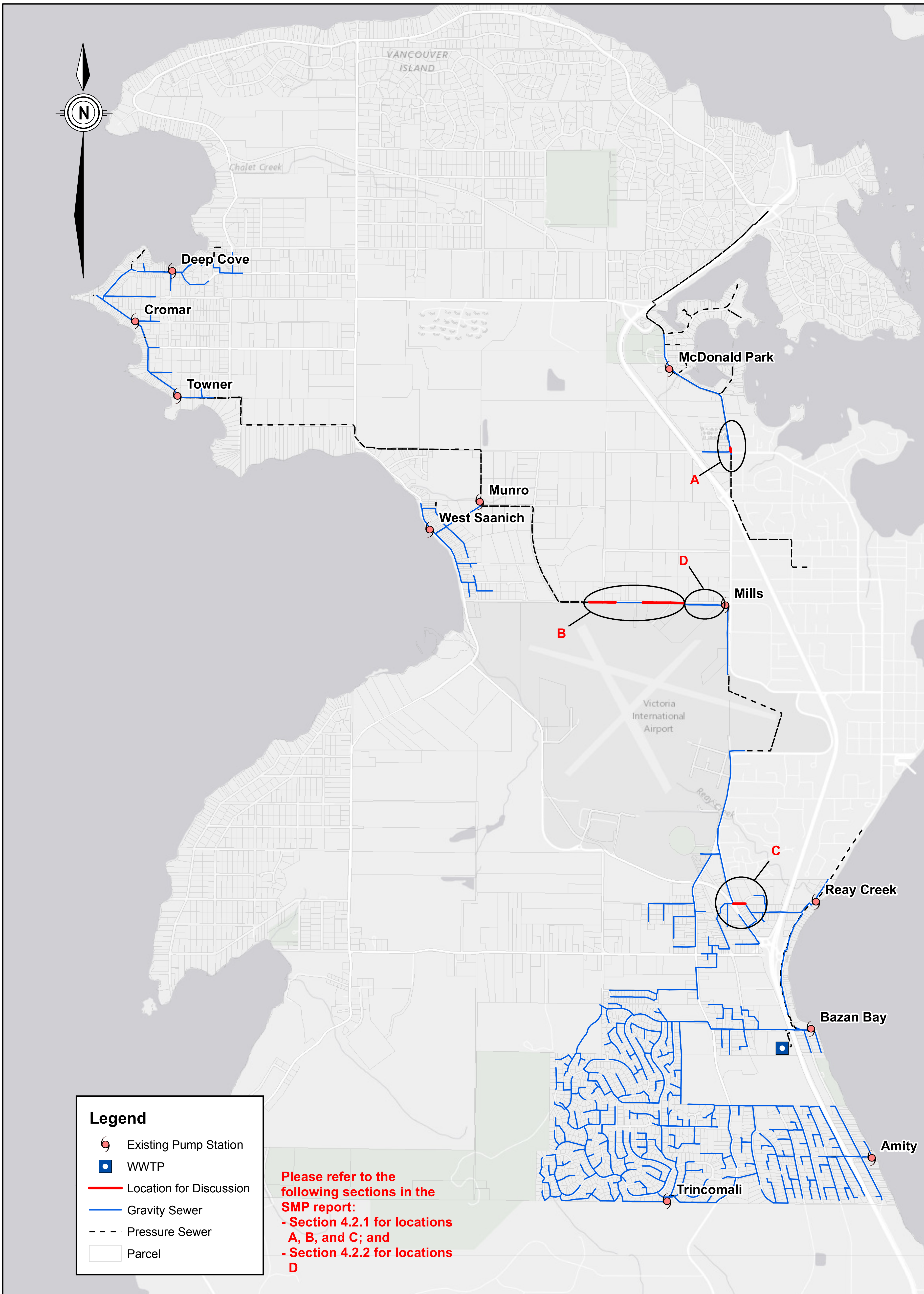
	CRD Allocated Peak Flow (L/s)	Modeled Future $PWWF_5$ Flow (L/s)	Remaining Capacity (L/s)	Note
Sidney Peninsula Trunk	56.7	56.2[1]	0.5	
North Saanich Peninsula Trunk	34.7	18.1	16.6	
Central Saanich Peninsula Trunk	16.2	42.6[2]	*(26.4)*	Existing PDWF is within the allocated capacity. An I&I analysis of the area is recommended to justify the capacity allocation requirements.

1 – Total modeled flow includes 16.4 L/s from McDonald Park Forcemain and 39.8 L/s from Reay Creek Forcemain.
2 – Total modeled flow includes 6.6 L/s from Amity Forcemain and 36.0 L/s from the southern Dean Park.

4.2.2 Gravity Sewer System

Based on the hydraulic assessment criteria in Section 4.1.1, the District's gravity sewer system is generally capable of conveying both existing and future peak wet weather flows, with a few locations noted on Figure 4-1, which were analyzed more closely and are discussed individually below.

Path: Q:\2100-2199\2148-012\430-GIS\MXD-Rp\Figure4-1_ModelingResults.mxd Date Saved: 04/11/2016 10:21:15 AM
Author: hbai



Reference: Orthophoto from Bing aerial image map.

KERR WOOD LEIDAL
consulting engineers

Sanitary Sewer Master and Asset Renewal Plan

Project No. 2148-012

Date November, 2016

500 0 500 (m)
1:30,000

Locations for Discussion

Figure 4-1

Location A: The 200 mm dia. sewer on McDonald Park Road north of John Road was flagged with a HLoS rating of "E" (capacity exceeded) due to its flat grade. Surcharging to pipe crown is not predicted. An HGL profile together with peak flow and velocity for this location are shown on Figure 4-2. No pipe upgrades are warranted or recommended for this section. It is recommended the District confirm the invert elevations for this section of piping. Regular flushing is also recommended as this pipe may be more prone to sedimentation which could result in blockages.

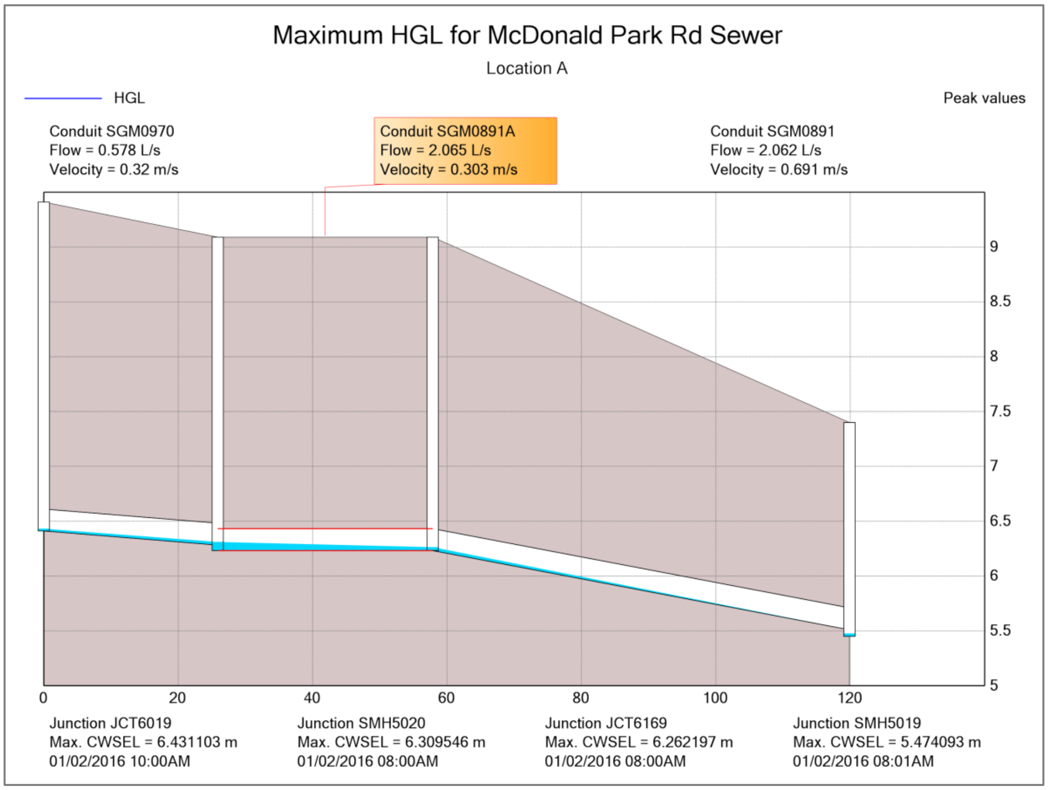


Figure 4-2: HGL Profile under Future PWWF$_5$ for Location A

Location B: The sewers receive pumped flows from the upstream Munro Pump Station. Under the existing $PWWF_5$ condition, two sewers (SGM0866, and SGM0871) were flagged as near capacity (HLoS rating "D"). Under the future $PWWF_5$ condition, additional three sewers will become near capacity (SGM0867, SGM0870, and SGM0872) due to future developments abutting Mills Road. Figure 4-3 displays the hydraulic profile together with peak flow and velocity of sewers at Location B under the future $PWWF_5$ condition. Upgrades are not anticipated to be required for these sewers in the current planning horizon.

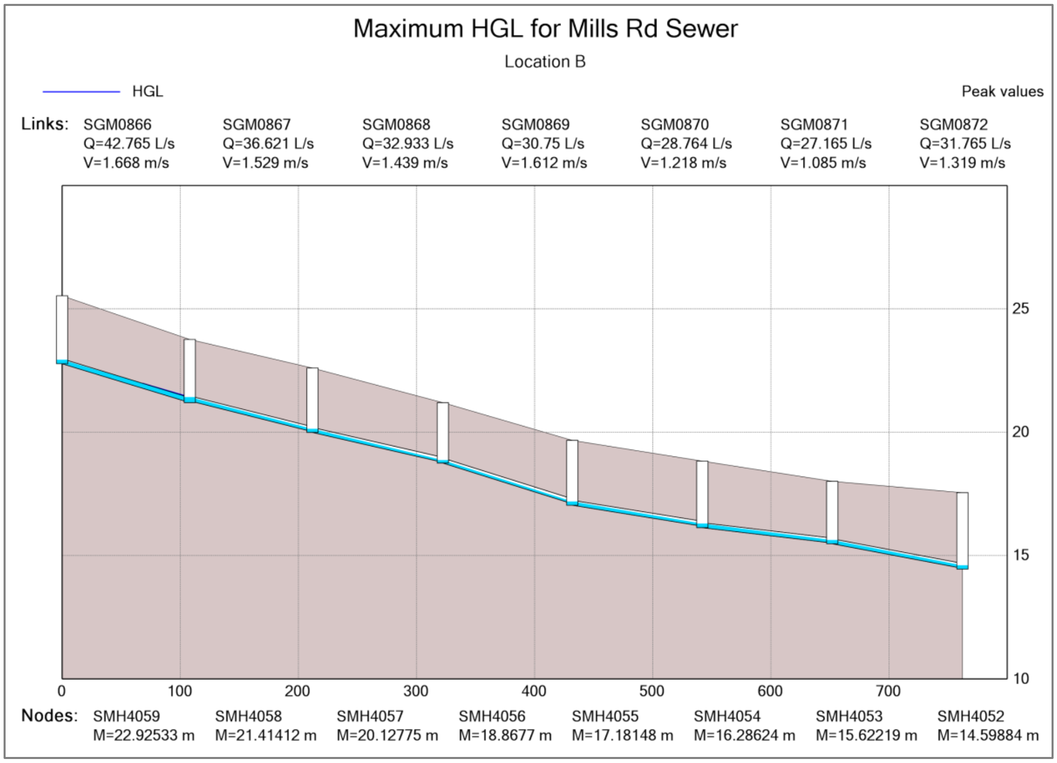


Figure 4-3: HGL Profile under Future $PWWF_5$ for Location B

Location C: Under the future $PWWF_5$ condition, the 250 mm dia. sewer (SGM0696) at Canora Road and Rideau Avenue was identified as near capacity (HLoS rating "D"). This flow condition has assumed that the upstream Mills Pump Station operates at a reduced pumping rate of 19 L/s. The hydraulic profile together with peak flow and velocity for Location C are shown on Figure 4-4. At $PWWF_5$, the pipe does not surcharge and therefore, no upgrades are required at this location in the current planning horizon.

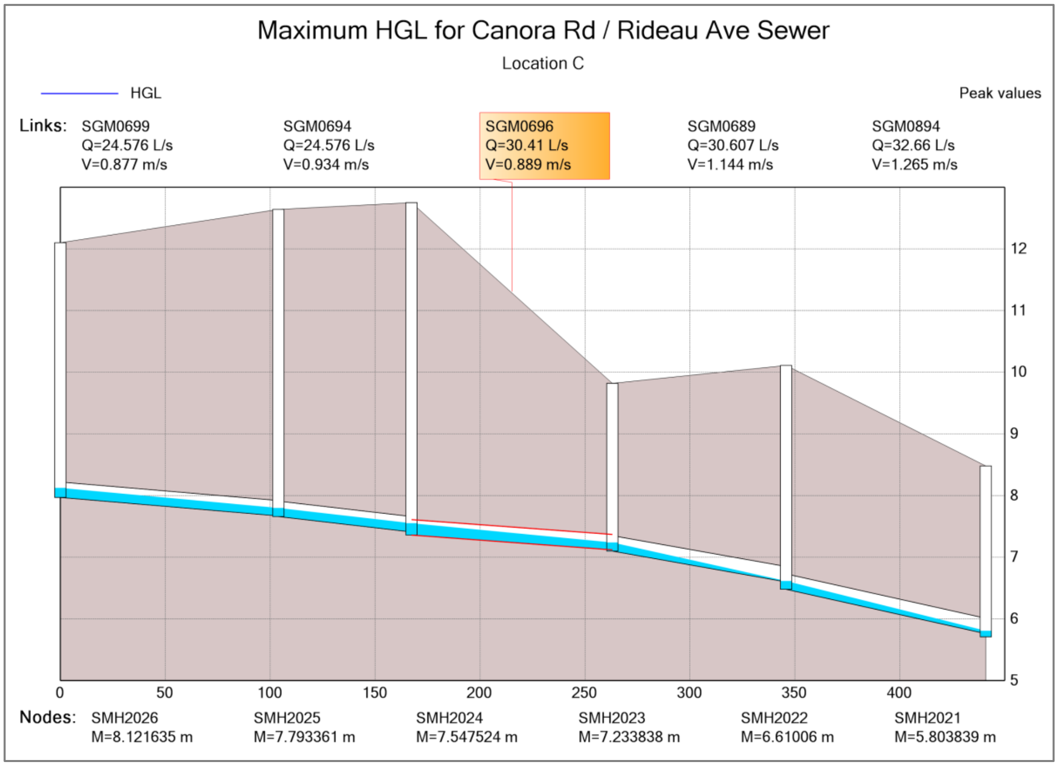


Figure 4-4: HGL Profile under Future $PWWF_5$ for Location C

4.2.3 Pumping System

Forcemains

Flows and velocities in the pump station forcemains were summarized in Table 4-5. The table also shows the forcemain size, material, length, and year of installation for each pump station. A theoretical capacity at an assumed 2 m/s velocity for each forcemain was calculated to compare with the modeled maximum flow.

The low pressure mains were not included.

Table 4-5: Summary of Forcemain Flow and Velocity

Pump Station	Diameter (mm)	Material	Length (m)	Installation Year	Capacity @ 2 m/s (L/s)	Modeled Max. Flow (L/s)	Average Velocity (m/s)
Amity PS	150	PVC	383	2001	35.3	24.3	1.38
Bazan Bay PS	100	PVC	270	2001	15.7	10.2	1.30
Cromar PS	150	PVC	174	2005	35.3	25.9	1.46
Deep Cove PS	100	PVC	306	2005	15.7	8.6	1.1
McDonald Park PS	150	PVC	2,371	2006	35.3	22.7	1.29
Mills PS	250	HDPE	1,696	2005	98.2	19.0	0.39
Munro PS	200	HDPE	1,519	2005	62.8	41.0	1.30
Reay Creek PS	200	PVC	14	2001	62.8	38.0	1.21
Swartz Bay to McDonald Park	150	PVC	131	2006	35.3	10.3	0.6
		Unknown	1,419	Unknown			
Towner PS	200	HDPE	3,212	2005	62.8	36.4	1.16
Trincomali PS	150	PVC	626	2001	35.3	28.7	1.62
West Saanich PS	100	PVC	178	2005	15.7	9.7	1.24

The table above indicates that restricting the pumping rate of the Mills Road pump station at 19 L/s results in sub-standard velocities in its forcemain. Similarly, the forcemain from Swartz Bay to the McDonald Park Road gravity sewer has low velocities at a theoretical pumping rate of 9.1 L/s from BC Ferries. Periodical flushing of the forcemains at a higher flow rate is recommended to prevent solids buildup.

Pump Station Capacity Analysis

Table 4-6 shows the existing and future $PWWF_5$ into the pump station compared with the estimated pumping capacity (for 1 pump running). There are a number of cases that a pump station receives pumped flows from upstream pump station(s). The instantaneous peak inflow due to upstream pumping is not appropriate for use when comparing with the pumping capacity, as storage is provided by the pump station wet well. For those pump stations, a 30-minute average flow was calculated for capacity analysis.

Table 4-6: Pump Station Capacity Analysis

Pump Station	Estimated Capacity for 1 Pump (L/s)	Existing $PWWF_5$ (L/s)	Future $PWWF_5$ (L/s)	Note
Amity	24.3	6.3	6.3	Discharges to WWTP via Central Saanich Peninsula Trunk
Bazan Bay	10.2	1.3	1.3	Upstream of Reay Creek PS
Cromar	25.9	8.8	8.8	Upstream of Towner PS
Deep Cove	8.6	4.1	4.1	Upstream of Cromar PS
McDonald Park	22.7	14.8	16.0	Discharge to Sidney Peninsula Trunk via Sidney PS
Mills	19.0 (restricted flow rate)	24.7	30.7	Tank provides storage for shaving of peak inflows and accommodating restricted flow rate.
Munro	41.0	16.9	16.9	Upstream of Mills PS
Reay Creek	38	36.5	40.0	Discharge to Sidney Peninsula Trunk
Towner	36.4	12.4	12.4	Upstream of Munro
Trincomali	28.7	19.8	19.8	Ultimately discharges to Central Saanich Peninsula Trunk via south Dean Park collection system
West Saanich	9.7	3.8	3.8	Upstream of Munro

As can be seen in the above table, most pump stations have adequate capacity to convey existing and future $PWWF_5$.

Mills Road Pump Station

The existing storage tank (3-barrel 30.5 m long, 2.4 m dia. pipe) has a total capacity of 366 m^3, which is adequate to accommodate existing $PWWF_5$ when the pumps are restricted to pump at a reduced rate of 19 L/s. Under the future development scenario, additional population from the proposed developments that contribute sanitary flows to the Mills Road Pump Station were estimated at 2,140 PE, including Sandown, Airport (south of Mills Rd.), and others. The model results indicated that the existing storage will be capable of accommodating future peak wet weather flows under a design 5-year storm event, as displayed on Figure 4-1 and Figure 4-5 for Location D. No additional storage was predicted to be required to accommodate flows from the proposed developments in the Mills Road Pump Station catchment.

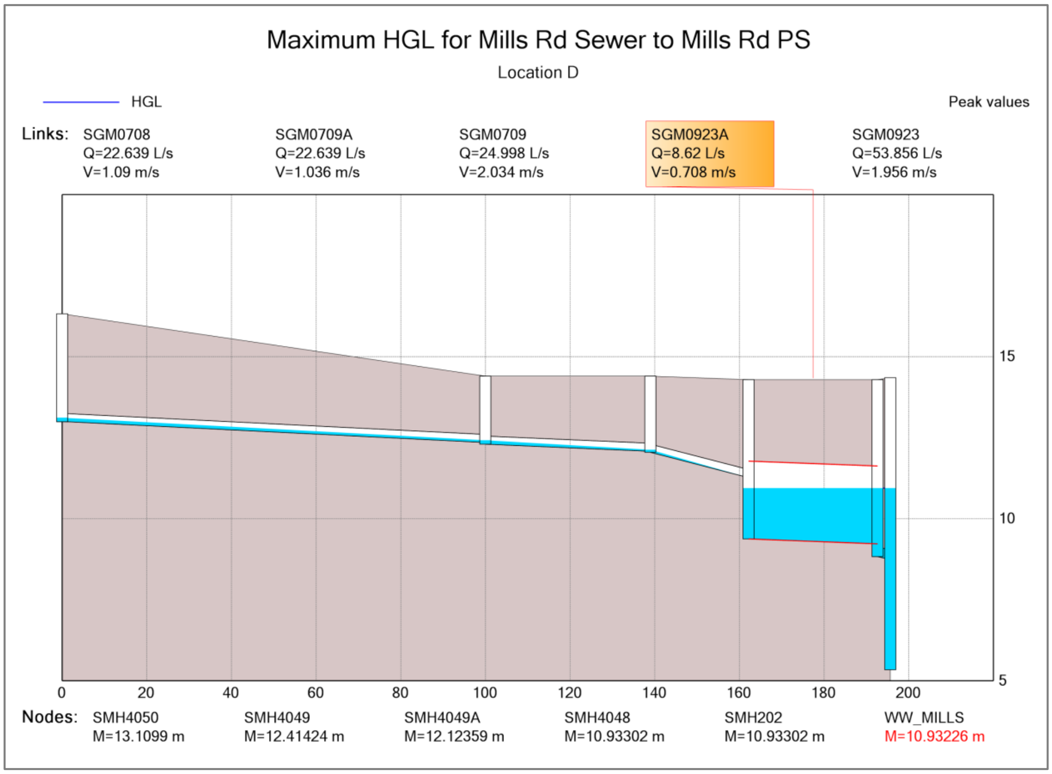


Figure 4-5: HGL Profile under Future $PWWF_5$ for Location D

Reay Creek Pump Station

For the Reay Creek Pump Station, it is understood the District is limited to pumping a peak rate of 56.7 L/s into the CRD's Sidney Peninsula Trunk Sewer. The pump station is currently restricted to pump at a maximum rate of 38 L/s with one (1) pump running. The model predicted a future PWWF to be 40 L/s, implying additional storage may be required to handle the future peak flow. Other improvement options could include conducting an I&I investigation (vapour/dye test) and rehabilitation to remove stormwater direct inflow (sanitary-storm cross connections).

5. Renewal Plan

The sanitary sewer infrastructure renewal plan presented in this section is intended to provide input into the District's overall asset management plan being completed by the District. The assessment is based on a review of pipe material and age information provided by the District through the GIS database. No field inspection programs were completed as part of this assessment (such as CCTV inspection). It is recommended that the District conduct required visual inspections of the system components prior to finalization of their asset management plan.

In addition to the age and material information provided, KWL met with District staff to discuss operational areas of concerns that should be included. During this discussion the following items were identified:

- CCTV Inspections;
- AC main rehabilitation; and
- Pump station upgrades.

5.1 Future Capacity

Based on the results of the hydraulic modeling, gravity system deficiencies noted can be generally addressed through optimization of pump station flow rates and utilizing available wet well storage in the existing pump stations. KWL recommends investigating the pump station operating conditions further prior to finalizing gravity sewer upgrade projects.

In addition to the above, a discussion on I&I rehabilitation is included.

5.2 Gravity System

As indicated above, gravity system deficiencies noted can be generally addressed through optimization of pump station flow rates and utilizing available wet well storage. The following is a summary of the District's existing gravity collection system.

The District has a total of 7,706 m of AC gravity mains ranging in size from 150 to 300 mm as summarized in the table below. These mains are all located within the Dean Park area of the municipality and were installed between 1975 and 1979; refer to Table 5-1. The pipes are currently reaching 40+ years of age.

Table 5-1: AC Gravity Pipe Inventory

Year Installed	Pipe Diameter			
	150 mm	200 mm	250 mm	300 mm
1975[1]	1,700	2,566	288	64
1977	760	1,175	-	-
1979	758	395	-	-
TOTALS	**3,218**	**4,136**	**288**	**64**

Note:
1. Data for a total of 692 m of 200 mm and 64 m of 300 mm AC pipe did not include age information and has been assumed to be installed in 1975.

The District has a total of 51,238 m of PVC gravity mains ranging in size from 100 to 300 mm as summarized in the table below. Use of PVC piping started in 1979 and remains the current standard material selection. The pipes are currently reaching an average age of 17+ years.

Table 5-2: PVC Gravity Pipe Inventory

Year Installed	Pipe Diameter				
	100 mm	150 mm	200 mm	250 mm	300 mm
1979	-	758	404	-	-
1980	-	178	163	-	-
1981	25	878	1444	-	-
1982	-	138	534	-	-
1983	-	252	590	-	-
1984	--	71	348	-	-
1985	-	-	334	-	-
1987	-	556	1,955	-	-
1988	-	299	782	-	-
1989	-	265	899	-	-
1990	-	226	176	-	-
1991	-	875	499	-	-
1992	-	477	543	-	-
1996	-	-	46	-	-
2001		220	51	-	-
2002	41	4,287	18,369	1,277	227
2003	-	-	39	-	-
2005	-	-	73	-	-
2006		778	874	-	-
2007	-	763	8,272	477	959
2008	-	138	89	-	-
2009	-	-	111	-	-
unknown	-	-	481	-	-
TOTALS	**66**	**11,158**	**37,076**	**1,753**	**1,185**

5.2.1 Rehabilitation

AC Gravity System

The design life expectancy of AC gravity sewer pipe is generally documented to be in the order of 100 years. The actual lifespan is dependent on soil conditions, groundwater levels, installation quality, etc., and can range from as low as 40 years in corrosive soils to 150 + years in well drained non-corrosive soil conditions.

Rehabilitation of the AC gravity sewers could be completed via:

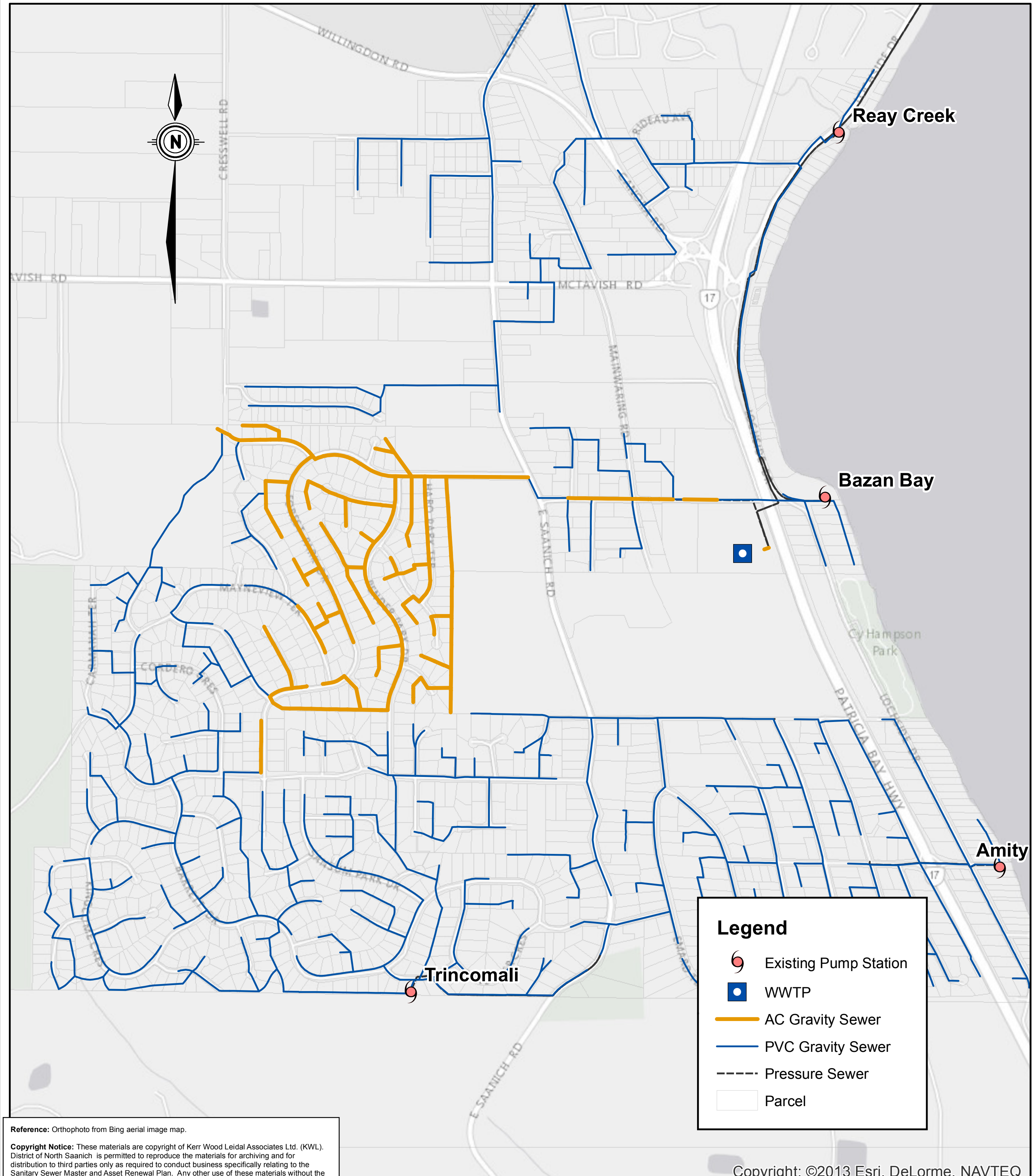
- Open cut replacement in the same alignment;
- Installing a new pipe on a new alignment (abandon existing pipe in place);
- Relining the existing piping; or
- A combination of all of the above.

As previously stated, a comprehensive CCTV inspection program is recommended to assess the condition of the collection system. The results of the inspection will confirm pipe shape, degree of joint separations, misalignments, cracking/fracturing, partial collapses, blockages, settlement (ponding), etc. Depending on the system deficiencies found, a detailed replacement strategy and cost benefit analysis can be developed which optimizes the lifespan of the existing pipe and prioritizes replacements to minimize long term costs.

In the absence of a CCTV inspection and based on discussions with District staff (December 3, 2015), the renewal plan presented is based on rehabilitating the entire AC gravity sewer system via lining. Typical lining installations can reduce the internal pipe diameter by up to 12 mm (4 to 6 mm thick lining). A model scenario was performed which reviewed the effects of AC lining within the system. The results of the scenario indicated no adverse hydraulic deficiencies or changes in the Hydraulic Level of Service assessment.

PVC Gravity System

The long term performance and rehabilitation requirements of the District's PVC pipes are currently unknown; replacements due to pipe lifespan concerns are not warranted in the current planning horizon.



Reference: Orthophoto from Bing aerial image map.

KERR WOOD LEIDAL
consulting engineers

Sanitary Sewer Master and Asset Renewal Plan

Project No.	Date
2148-012	October 2016

200 0 200 (m)
1:15,000

Location of AC Gravity Sewers

Figure 5-1

Path: Q:\2100-2199\2148-012\430-GIS\MXD-Rpt\Figure5-1_AC_Sewer.mxd Date Saved: 05/10/2016 4:33:55 PM
Author: hbai

5.3 Forcemains

The District has a total of 12,299 m of forcemains mains ranging in size from 100 to 250 mm as summarized in the table below. The piping material is a combination of PVC and HDPE. The pipes are currently reaching an average age of 12+ years.

Table 5-3: Forcemain Inventory

Pump Station	Diameter (mm)	Material	Length (m)	Installation Year
Amity PS	150	PVC	383	2001
Bazan Bay PS	100	PVC	270	2001
Cromar PS	150	PVC	174	2005
Deep Cove PS	100	PVC	306	2005
McDonald Park PS	150	PVC	2,371	2006
Mills PS	250	HDPE	1,696	2005
Munro PS	200	HDPE	1,519	2005
Reay Creek PS	200	PVC	14	2001
Swartz Bay to McDonald Park	150	PVC	131	2006
		Unknown	1,419	Unknown
Towner PS	200	HDPE	3,212	2005
Trincomali PS	150	PVC	626	2001
West Saanich PS	100	PVC	178	2005

Based on the hydraulic assessment all of the forcemains have adequate capacity for the current planning horizon and no upgrades are warranted at this time. The modeling did indicate lower than optimal velocities in the Mills Road and McDonald Park forcemains. These systems could be flushed periodically to prevent sediment accumulation in the pipe which may result in pipe blockages over time.

The long term performance and rehab requirements of the District's PVC and HDPE pipes are currently unknown; replacements due to pipe lifespan concerns is not warranted in the current planning horizon.

5.4 Pump Station Upgrades

The District currently operates a total of eleven (11) sewage pump stations. As noted in the hydraulic assessment of the pump station, all stations have adequate capacity to service existing and future demands.

5.4.1 Mills Road Pump Station Storage

The modeling results indicate there is adequate storage at the Mills Road pump station in the future scenario however the following are recommended if the District would like to reduce dependency on the storage:

- Calibrate wet weather flows and re-assess future storage requirements;
- Re-assess in 10 years to confirm flows and rate of build-out within the catchment; and
- Conduct vapour testing to reduce potential cross connections.

5.4.2 PS Control System Replacement

The District has indicated that four of the pump stations constructed in the early 2000s are in need of upgrades to the electrical systems, while the remaining systems should not require significant replacements for the next twenty (20) years. The upgrades are required due to aging control systems and increasing difficulty in accessing replacement parts and degradation of components as a result of weather exposure. Pump replacement costs have not been included as these costs are included in the District's Operation and Maintenance budget.

The stations affected are:

1. Reay Creek;
2. Amity;
3. Trincomali; and
4. Bazan Bay

Reay Creek pump station is currently being upgraded and works will be completed in 2016. The remaining stations are proposed to be upgraded in order as shown in the list above. The pump station upgrades will generally consist of:

- New control / electrical kiosks;
- Standby generator replacements; and
- VFD system replacements (as required).

The District has identified the following performance requirements for the pump station upgrades:

- All enclosures (kiosks and standby generators) to be painted marine grade aluminum; and
- All standby generators to be Cummins units.

5.5 Inflow and Infiltration

I&I for the District system has been calculated based on the Reay Creek pump station catchment, using "RDI&I Envelope" method. The resulting 5-year rates are as follows:

- 24-Hour I&I Rate is 7,700 L/ha/day; and
- 1-hour I&I Rate is 15,900 L/ha/day

5.5.1 Cross Connections

The 5-year, 1-hour I&I rate was determined to be approximately double the 5-year, 24-hour rate as shown in Table 3-2. While this 1-hr rate is not high relative to other Vancouver Island rates, the ratio is higher than would be normally expected for a predominately PVC collection system which is relatively new. This potentially indicates cross connections within the system. Cross connections are often a result of improperly connected roof drains or perimeter drains and may be identified with vapour testing. While vapour testing is relatively inexpensive, correction of the cross connection can be difficult to enforce as it is frequently the responsibility of the property owner. However, cross connections directly consume peak flow capacity, and correction can increase available capacity for future development. As part of the CCTV program, vapour testing is recommended to identify potential cross connections.

5.5.2 Comparison to CRD Core Area

KWL has conducted extensive analysis of I&I rates within the CRD Core Area. As part of this extensive work, KWL investigated a relationship between Sewer Age and I&I rates (Kerr Wood Leidal Associates Ltd., 2012) (Kerr Wood Leidal Associates Ltd., 2015). Based on this work, a relationship between sewer age and the 5-Year 24-Hour I&I rate was established as follows:

- I&I = 8,314 * EXP(0.0165 * Age).

The GIS database was used to determine the Sewer Age. Sewer Age is determined by linearly weighting pipe age by its length. Based on the data, the District's system has a Sewer Age of 23 years. It is noted that the sewer age for AC and PVC systems within the catchment are 40 and 17 years respectively. The system is predominantly PVC.

The results are presented on the Figure 5-2.

Figure 5-2

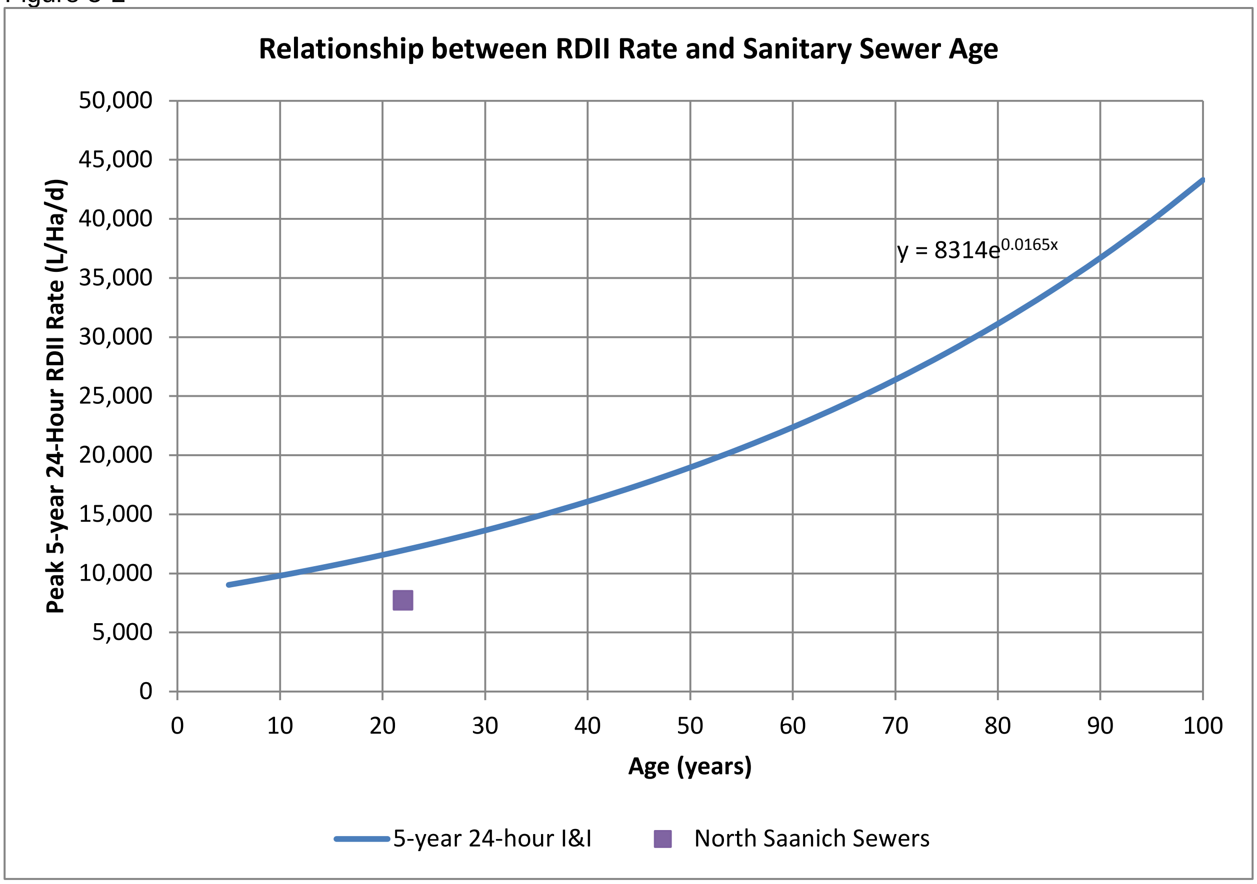


Figure 5-2: Sewer Age vs. RDII

Based on the data, the District's sanitary sewer system is experiencing less I&I than the rate developed for the CRD core area based on extensive data collection on existing systems. It is also noted that the District's rate is less than 8,314 L/Ha/day which is the base rate for a new system (i.e. sewer age = 0). It should be noted that the above curve is based on the overall CRD sewer system which is comprised of piping which is in excess of 100 years of age and contains older materials such as brick, AC, non-corrode, etc., as a result the curve for a predominately PVC system would be expected to be flatter as the system ages.

Based on the forgoing, a I&I reduction program is not currently warranted for the District's system. It is recommended that the District maintain pump station flow records and conduct sewer flow monitoring in 2020 to update I&I rates. This program should focus on the areas outside of the AC sewers located in Dean Park. The AC rehabilitation outlined in Section 5.1 is proposed to address pipe end of life concerns; however I&I will be reduced indirectly by these proposed works.

Given the low I&I rates calculated for the system, an extensive manhole rehabilitation program is not warranted. Manhole rehabilitation can be completed as required based on operational concerns.

5.6 Long-Term Funding Strategy

Components for consideration in a long term funding strategy include annual amounts for the following sewer system rehabilitation work:

- CCTV inspection program;
- AC gravity main rehabilitation; and
- Pump station upgrades.

5.6.1 CCTV Inspection Program

A CCTV inspection program is recommended for the entire collection system. Common re-inspection intervals for different pipe types are:

- Every 10 years for AC gravity piping; and
- Every 20 years for PVC gravity piping.

An initial inspection would assist the District in prioritizing a system wide rehabilitation program and may justify pushing back the replacements to extend the useful life of the assets and reduce long term replacement costs. A CCTV inspection typically ranges in cost from $5 to $6/m.

Table 5-4: CCTV Inspection Program

Pipe Type	Estimated CCTV Costs ($/m)	Length (m)	Estimated Cost
AC	$6	7,706	$46,500
PVC	$6	51,238	$308,000
		TOTAL ESTIMATED CCTV COST	**~ $354,500[3]**

Notes:
1. Costs are based on 2016 dollars.
2. Excludes engineering & contract management.
3. Assumes the entire AC and PVC gravity main collection system is inspected.

Based on the findings of the CCTV program, localized vapor testing may be warranted to identify cross connections. Costs for conducting a vapour testing program are not included in the above costs.

The following summarizes the estimate of annual funding for CCTV inspections:

- $5,000 per year for AC gravity piping; and
- $15,500 per year for PVC gravity piping.

5.6.2 AC Gravity Main Rehabilitation

As discussed previously, in the absence of a condition assessment, the following costs are presented based on relining the entire AC gravity collection system. Actual rehabilitation would be a function of the needs identified as part of the condition assessment program (CCTV inspection) discussed above.

The lineal meter costs for lining listed in the table below are based on the following assumptions:

- Mains are in suitable condition for lining;
- Lining length is a maximum of 70 m; and
- A total of 7 - 100 mm diameter lateral connection reinstatements per lining length.

Table 5-5: AC Main Lining Capital Costs

Pipe Diameter (mm)	Estimated Lining Costs ($/m)	Lining Length (m)	Estimated Cost
150	$350	3,218	$1,126,300
200	$375	4,136	$1,551,000
250	$410	288	$118,080
300	$440	64	$264,000
		TOTAL ESTIMATED LINING COST	**~ $3,100,000[4]**

Notes:
1. Costs are based on 2016 dollars.
2. Excludes engineering & contract management.
3. Excludes lateral relining to property line and installation of IC chambers.
4. Assumes the entire AC gravity main collection system was relined.

The cost for relining lateral connections to the property line was also investigated. The costs are in the range of $2M based on the following assumptions:

- Lateral lengths are 10 m from main to property line on average;
- IC chambers are installed at the property line;
- A total of 245 connected properties; and
- Laterals are a minimum of 100 mm diameter.

Based on the relative high cost of relining the laterals, replacement by conventional open cut may be employed. It is not recommended to conduct a complete service lateral replacement program and replacements should be completed on an as required basis. Based on this recommendation an allowance has not been included in the funding strategy for lateral replacements within the AC system.

The funding strategy assumes works will not commence until 2025 and would be completed over a 50 year horizon (2025 to 2075). The following summarizes the estimated annual funding for AC gravity system rehabilitation:

- $65,000 per year for AC gravity piping.

5.6.3 Pump Station Upgrades

The estimated costs for the pump station upgrades are summarized in the table below and will generally consist of:

- New control / electrical kiosks;
- Standby generator replacements; and
- VFD system replacements (as required).

The District has identified the following performance requirements for the pump station upgrades:

- All enclosures (kiosks and standby generators) to be painted marine grade aluminum; and
- All standby generators to be Cummins units.

Table 5-6: Pump Station Upgrade Costs

Station	Estimated Capital Costs
Amity	$260,000
Trincomali	$200,000
Bazan Bay	$200,000

Notes:
1. Costs are based on 2016 dollars.
2. Excludes engineering & contract management.
3. Excludes pump replacements.

The proposed annual funding assumes the above proposed works will be completed over a 25 year horizon. This annual funding could finance other long term pump station upgrades. The following summarizes the estimated annual funding for pump station upgrades:

- $30,000 per year.

5.6.4 Annual Funding

Based on the above estimates, the estimated annual funding is as follows:

- $50,500 annually from 2017 to 2025 for CCTV inspections and pump station upgrades; and
- $115,500 annually commencing in 2025 for CCTV inspections, pump station upgrades, and AC gravity main renewals.

6. Summary and Recommendations

6.1 Summary

6.1.1 Hydraulic Modeling

The District of North Saanich's sanitary sewer model was developed using the PCSWMM platform based on the sanitary infrastructure GIS data, with existing and future development loadings. The model was run with a per-capita loading rate of 185 L/cap/day, higher than the calculated per-capita loading rate of 110 L/cap/day from flow monitoring. The modeled dry weather flows (at 185 L/cap/day) were compared against the flow monitoring data collected at the Amity, Reay Creek, and McDonald Park pump stations. The comparison indicated that the model was deemed to be a representative tool which can be used with reasonable confidence to perform hydraulic analysis of the sanitary system under different loading conditions.

Hydraulic performance analysis of the gravity sewer system was conducted based on the model results using a Hydraulic Level of Service rating assessment method. The identified locations with potential hydraulic concerns were further reviewed by examining hydraulic grade line profiles to determine the severity of impact on servicing. The analysis indicated that all the gravity sewers are capable of conveying existing and future peak wet weather flows under a design five year return period storm event, with a couple of locations being identified as nearing full pipe capacity under future conditions. Upgrades to these locations are not recommended at this time.

The pumping stations in combination with the associated upstream storage facilities have capacities to accommodate existing and future peak wet weather flows, with the exception of the Reay Creek Pump Station. Under the future development scenario, the peak flow at the Reay Creek PS is currently predicted to exceed the pump station's regulated capacity (38 L/s) by about 2 L/s, implying additional storage might be required to handle the peak flow. Options to accommodate the potential flows at the Reay Creek PS could include reducing I&I to free up capacity at the Reay Creek PS.

The majority of pump station forcemains have velocities between 0.76 m/s and 3.0 m/s, except for the Mills Rd Pump Station forcemain and the forcemain that receives discharges from BC Ferries. Periodical flushing of the forcemains at a higher flow rate is recommended to prevent solids buildup.

I&I analysis was conducted for the Reay Creek pump station catchment, using "RDI&I Envelope" method. The analysis derived I&I flows and area-based rates associated with 5-year and 25-year return period storms, based on rainfall statistics from the Victoria Airport rain gauge operated by Environmental Canada. A 5-year Chicago design storm was generated for modeling of RDI&I using the three triangular unit hydrograph (RTK) method. Modeling of design RDI&I was carried out by adjusting the fraction of rainfall (R parameter), that becomes RDI&I, until the modelled peak RDI&I was reasonably representative of the calculated design peak RDI&I for the Reay Creek pump station catchment.

A 5-year peak 1-hour I&I was selected for PWWF modelling in order for capacity assessment. Calculated RDI&I rates are below those established for the CRD Core Area, as such an I&I reduction program is not warranted at this time.

6.1.2 Renewal Program

Based on discussions with District staff and the results of the hydraulic modeling, sanitary sewer renewals identified are:

- CCTV inspection program;
- A future AC Main rehabilitation program as warranted based on results of CCTV program; and
- Pump station upgrades.

Review of the estimated RDII rate for the sanitary sewer system indicates RDII rates are below those established for the CRD Core Area, as such an I&I reduction program is not warranted at this time. However cross connections may be present and vapour testing, as part of a CCTV inspection program, could be used to determine if/where these exist.

6.2 Recommendations

Based on the results of the hydraulic modeling, no projects targeting system capacity improvements were identified as being required in the current planning horizon. The renewal projects that have been identified are based on service life of assets. The following is a summary of the recommended renewal projects and annual funding levels:

6.2.1 CCTV Inspection Program

- A CCTV inspection program is recommended to aid in determining the current condition of the AC system, to assist the development of AC main rehabilitation projects, and to identify possible cross connections. The inspections could be repeated on a 10 year basis.
- A CCTV inspection is also recommended for the PVC gravity collection system to monitor pipe condition. The inspections could be repeated on a 20 year basis.
- The estimated annual funding is:
  - $5,000 per year for AC gravity piping; and
  - $15,500 per year for PVC gravity piping.

6.2.2 AC Gravity Main Rehabilitation

- In the absence of a condition assessment for the existing AC mains, the funding strategy presented is based on rehabilitating the entire AC system over a period of 50 years starting in 2025 via relining. Actual rehabilitation would be a function of the needs identified as part of the condition assessment program (CCTV inspection).
- It is recommended that the funding strategy be updated based on the results of the CCTV inspection program.
- The estimated annual funding for AC rehabilitation is:
  - $65,000 per year.

6.2.3 Pump Station Upgrades

- The proposed annual funding will provide funds for the known near term upgrades as well as finance other long term pump station upgrades.
- The estimated annual funding for pump station upgrades:
  - $30,000 per year.

6.2.4 Overall Annual Funding

Based on the above the following overall annual funding is estimated:

- $50,500 annually from 2017 to 2025 for CCTV inspections and pump station upgrades; and
- $115,500 annually commencing in 2025 for CCTV inspections, pump station upgrades, and AC gravity main renewals.

6.3 Report Submission

Prepared by:

KERR WOOD LEIDAL ASSOCIATES LTD.

Dec 16/16

Rob Rutherford, P.Eng.
Project Engineer

Dec 16, 2016

Hua Bai, P.Eng.
Hydraulic Modeling

Reviewed by:

for

Andrew Boyland, P.Eng.
Sr. Review Engineer

Statement of Limitations

This document has been prepared by Kerr Wood Leidal Associates Ltd. (KWL) for the exclusive use and benefit of DISTRICT OF NORTH SAANICH for the Sanitary Sewer Master and Asset Renewal Plan. No other party is entitled to rely on any of the conclusions, data, opinions, or any other information contained in this document.

This document represents KWL's best professional judgement based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by members of the engineering profession currently practising under similar conditions. No warranty, express or implied, is made.

Copyright Notice

Revision History

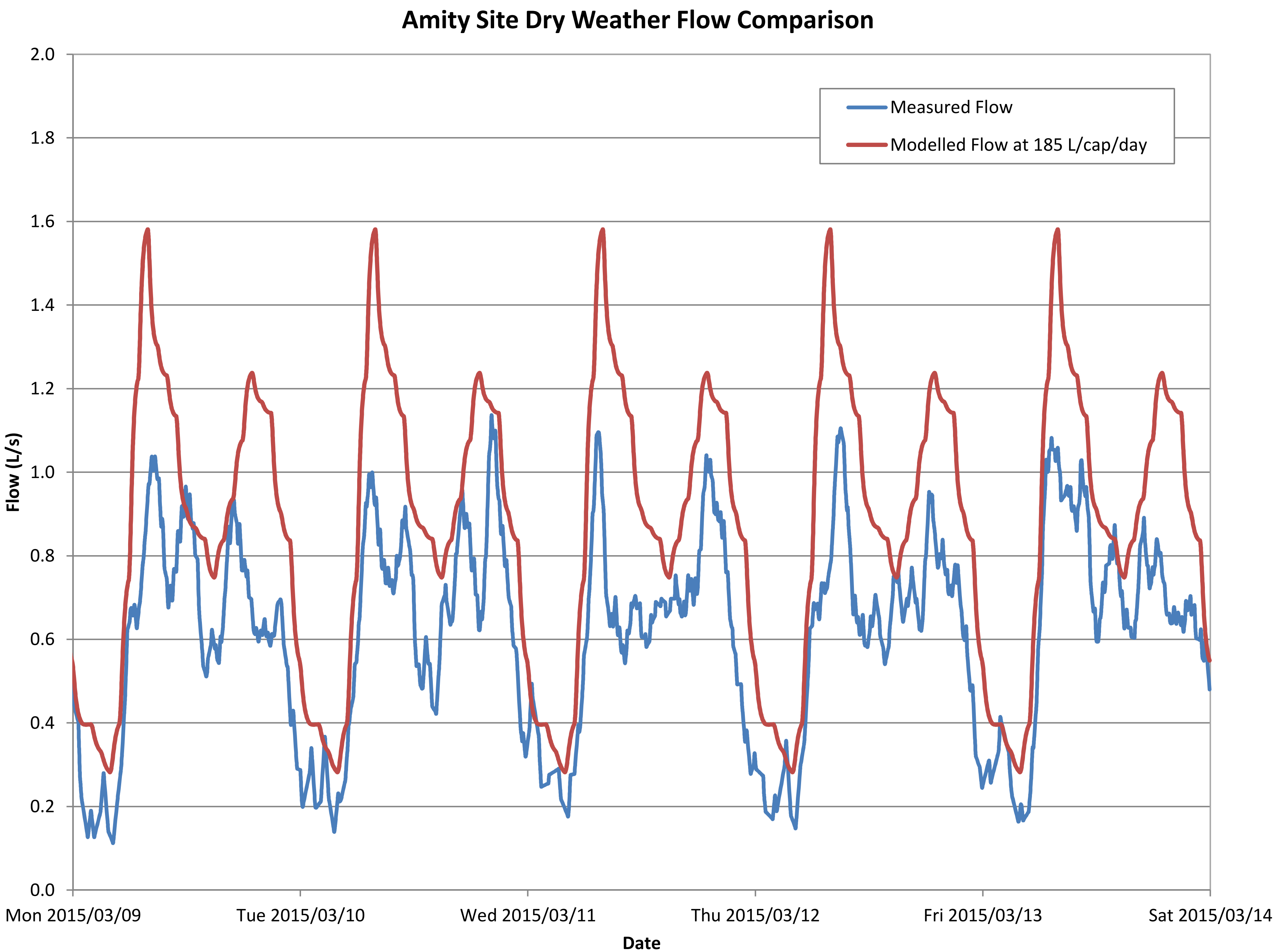
Revision #	Date	Status	Revision	Author
0	Jan. 27, 2016	Draft	Issued for Client Review	RAR / HB
1	March 11, 2016	Draft	Re-issued for Client Review	RAR / HB
2	December 16, 2016	Final	Final	RAR / HB

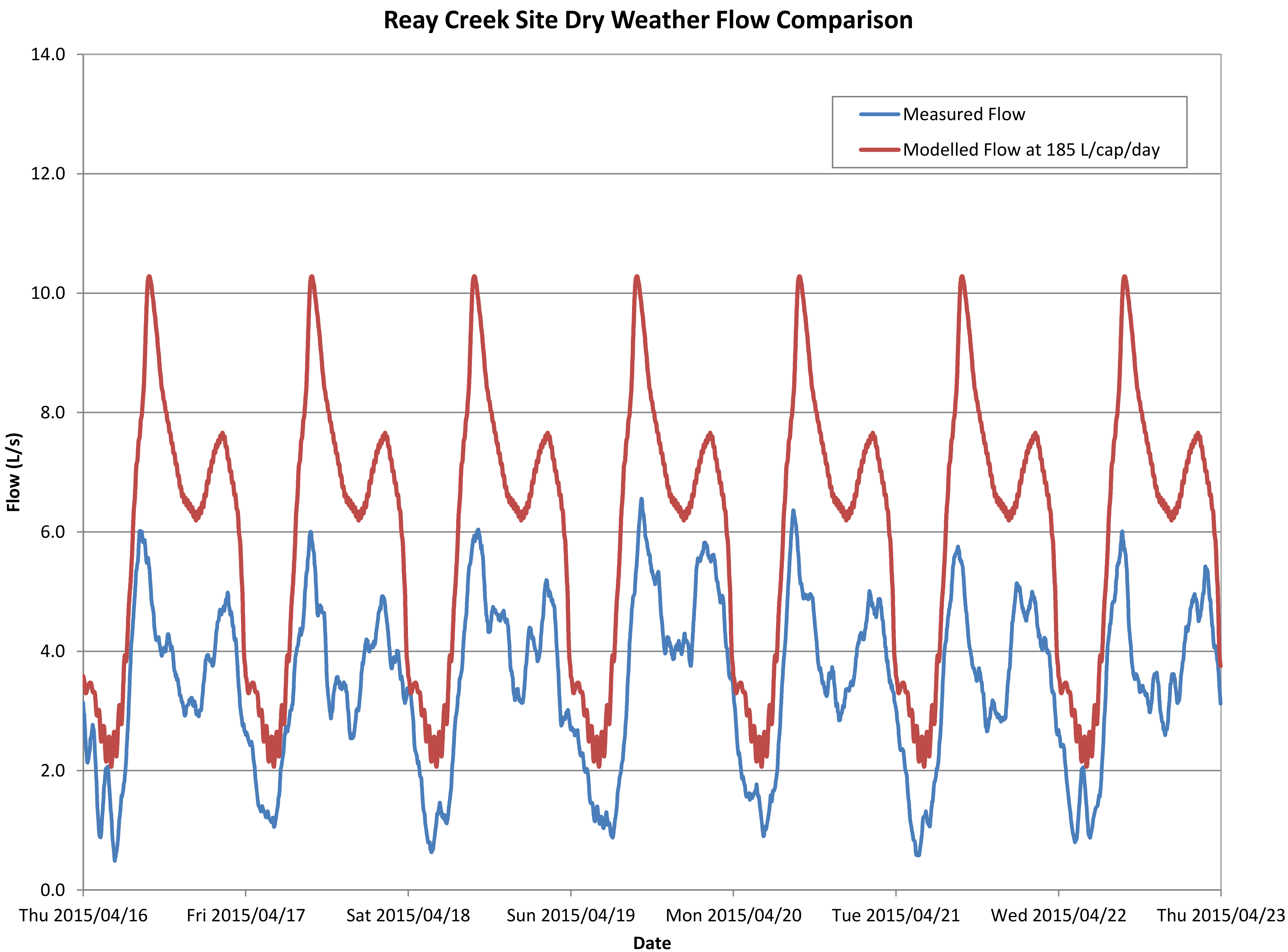
OQM | Organizational Quality Management Program

KERR WOOD LEIDAL
consulting engineers

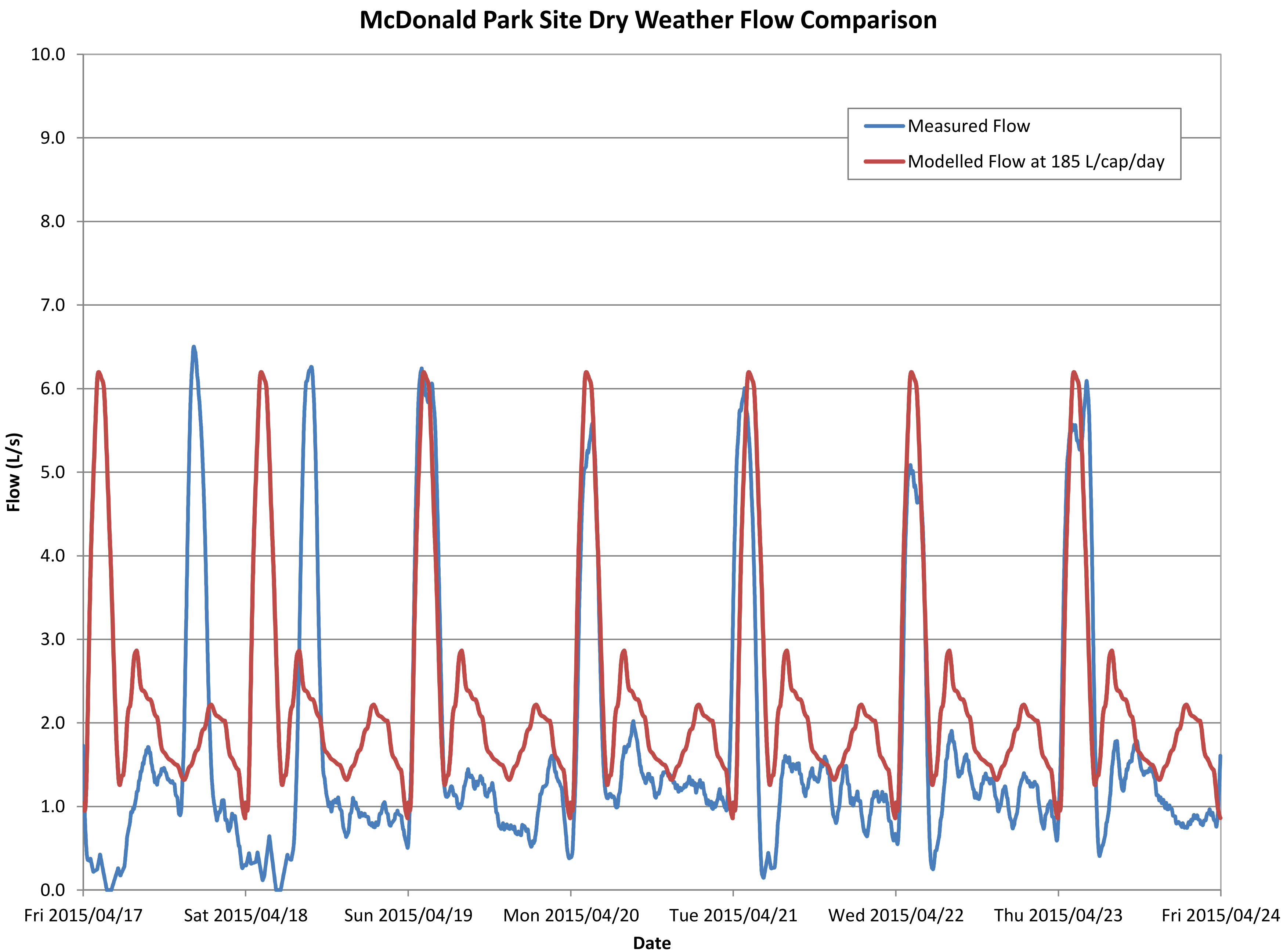
Appendix A

Dry Weather Flow Comparison Charts


Amity Site Dry Weather Flow Comparison
Measured Flow
Modelled Flow at 185 L/cap/day
Flow (L/s)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Mon 2015/03/09
Tue 2015/03/10
Wed 2015/03/11
Thu 2015/03/12
Fri 2015/03/13
Sat 2015/03/14
Date



Reay Creek Site Dry Weather Flow Comparison
Measured Flow
Modelled Flow at 185 L/cap/day
Flow (L/s)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Thu 2015/04/16
Fri 2015/04/17
Sat 2015/04/18
Sun 2015/04/19
Mon 2015/04/20
Tue 2015/04/21
Wed 2015/04/22
Thu 2015/04/23
Date

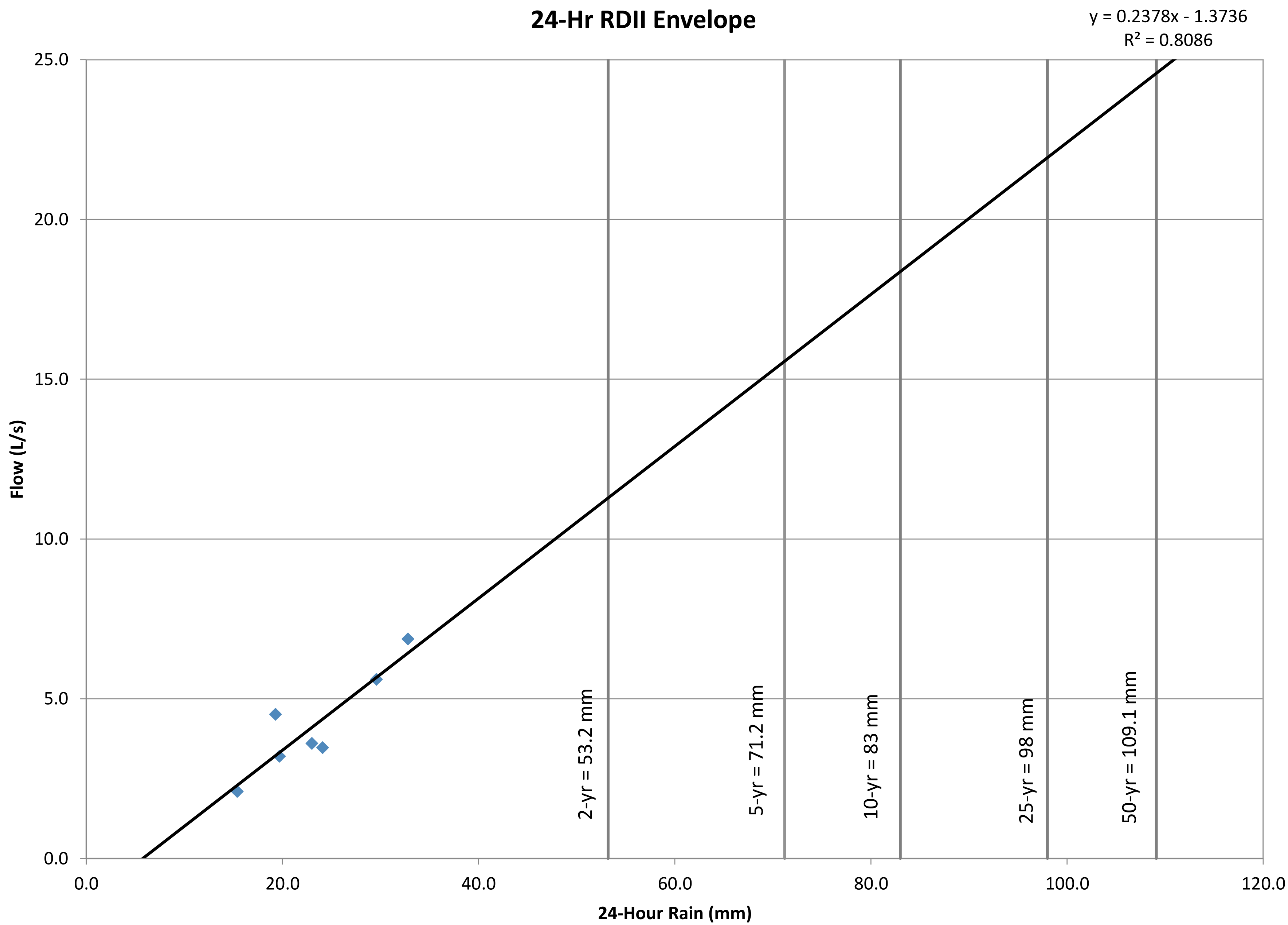


McDonald Park Site Dry Weather Flow Comparison
Measured Flow
Modelled Flow at 185 L/cap/day
Flow (L/s)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Fri 2015/04/17
Sat 2015/04/18
Sun 2015/04/19
Mon 2015/04/20
Tue 2015/04/21
Wed 2015/04/22
Thu 2015/04/23
Fri 2015/04/24
Date

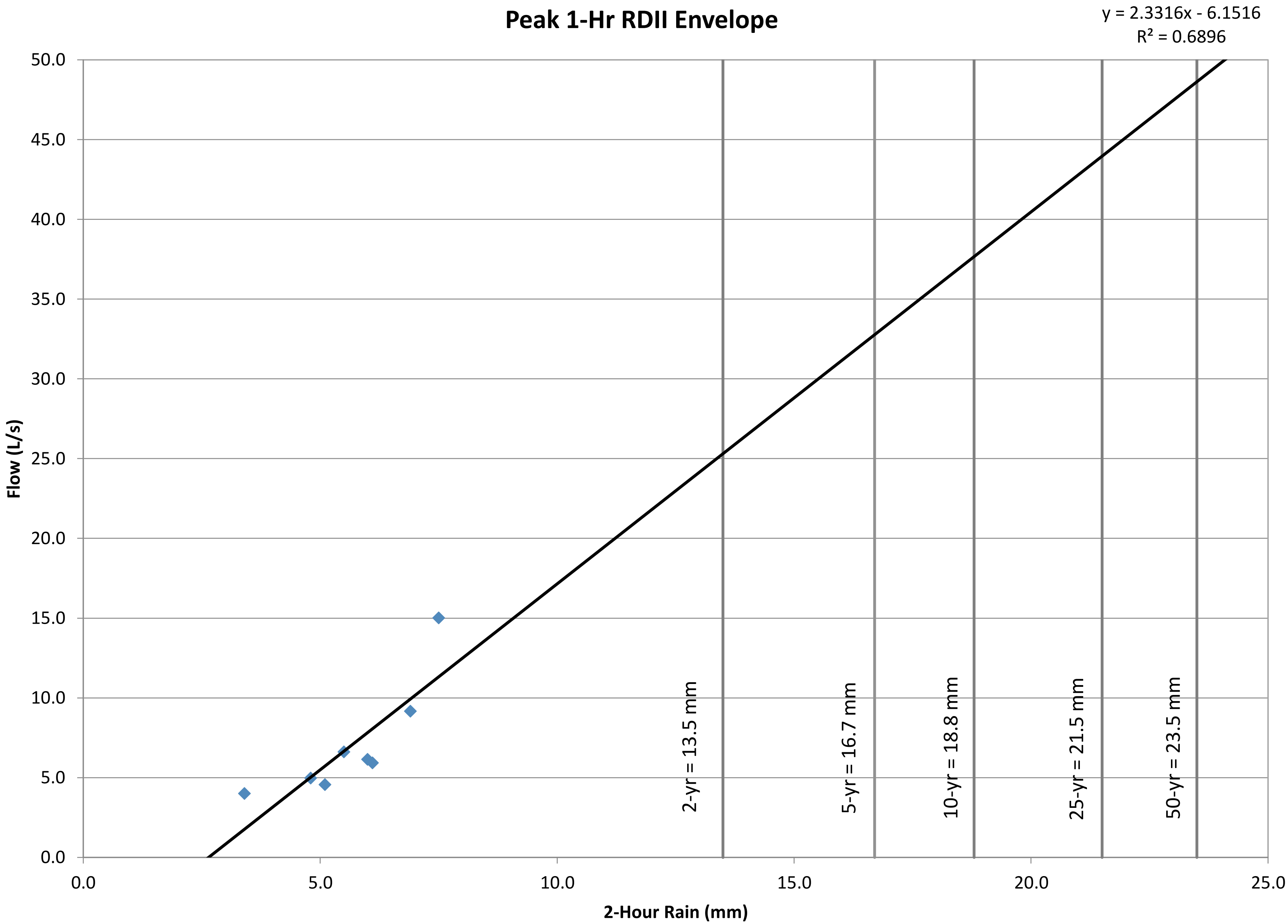
KERR WOOD LEIDAL
consulting engineers

Appendix B

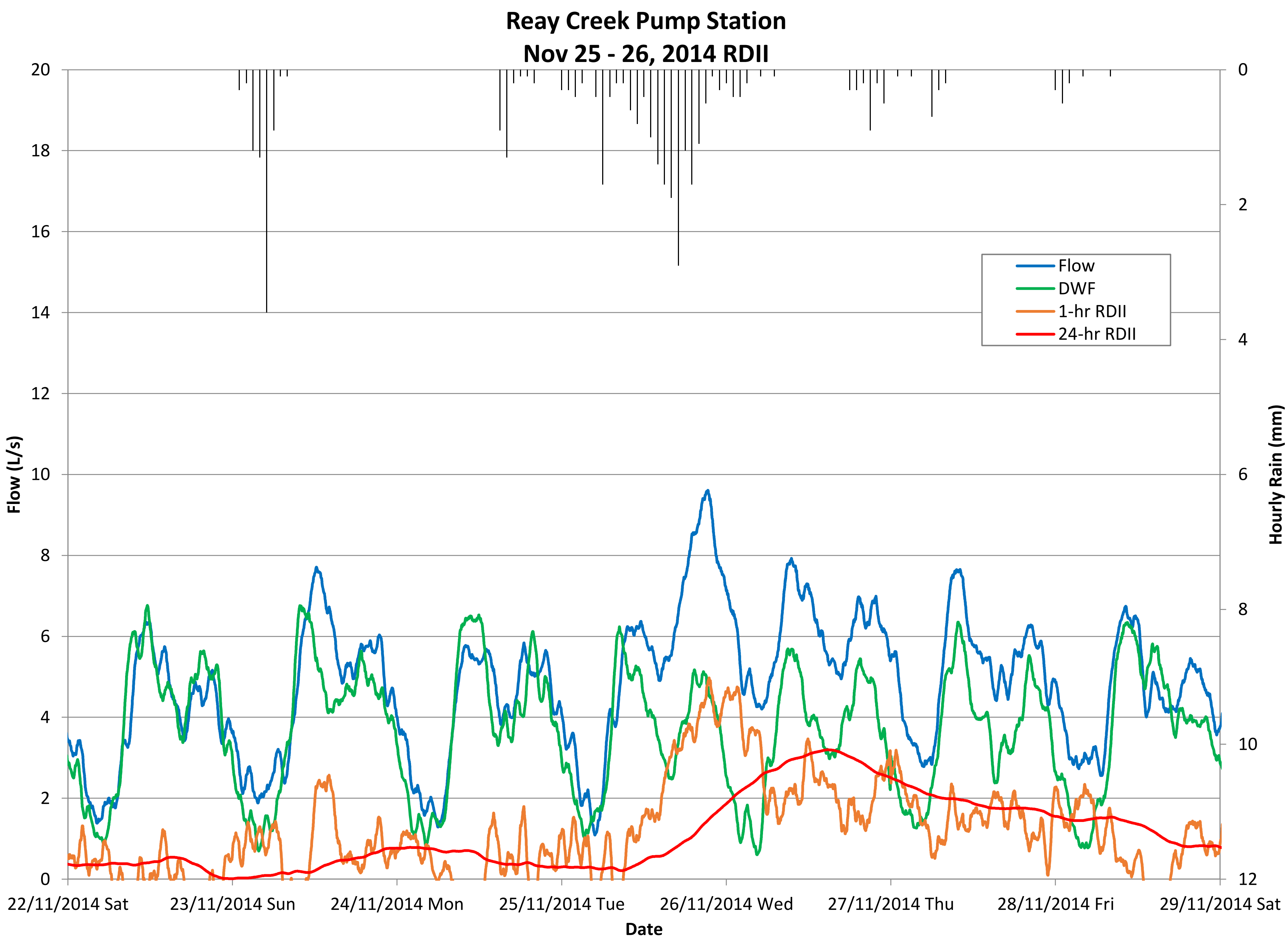
RDI&I Envelopes and RDI&I Response for Selected Storm Events



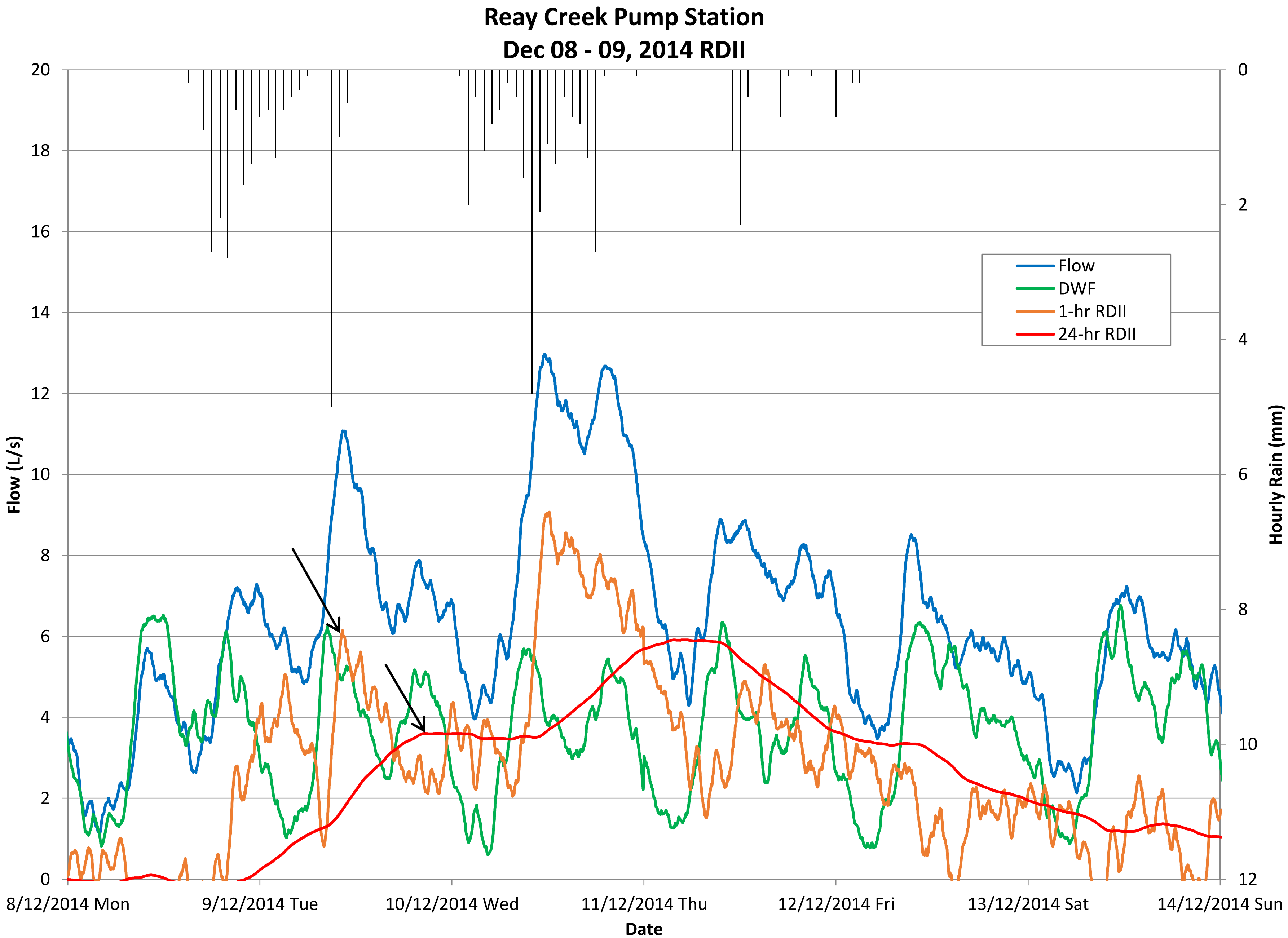
24-Hr RDII Envelope
y = 0.2378x - 1.3736
R² = 0.8086
Flow (L/s)
24-Hour Rain (mm)
25.0
20.0
15.0
10.0
5.0
0.0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
2-yr = 53.2 mm
5-yr = 71.2 mm
10-yr = 83 mm
25-yr = 98 mm
50-yr = 109.1 mm



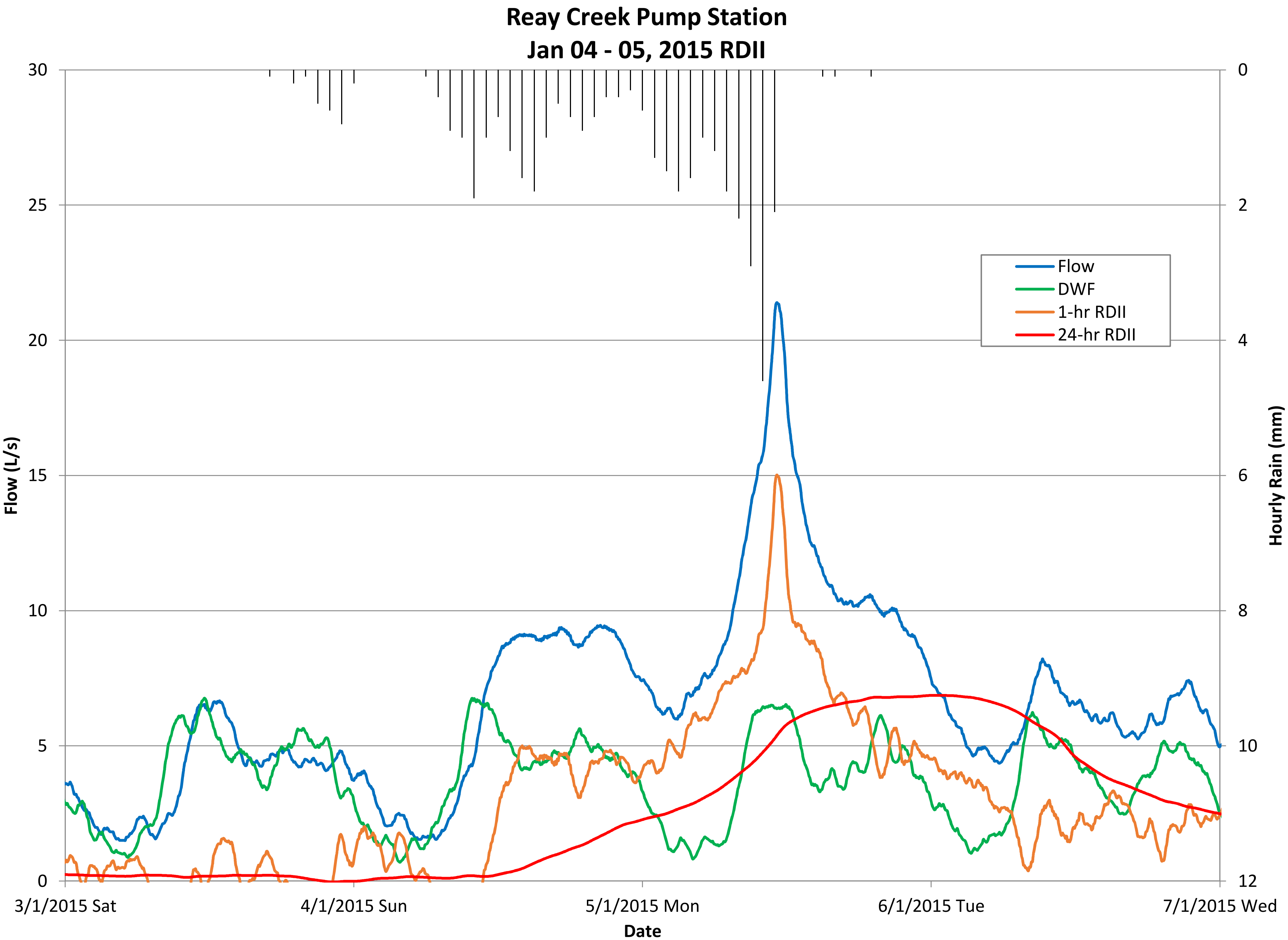
Peak 1-Hr RDII Envelope
y = 2.3316x - 6.1516
R² = 0.6896
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
Flow (L/s)
0.0
5.0
10.0
15.0
20.0
25.0
2-Hour Rain (mm)
2-yr = 13.5 mm
5-yr = 16.7 mm
10-yr = 18.8 mm
25-yr = 21.5 mm
50-yr = 23.5 mm



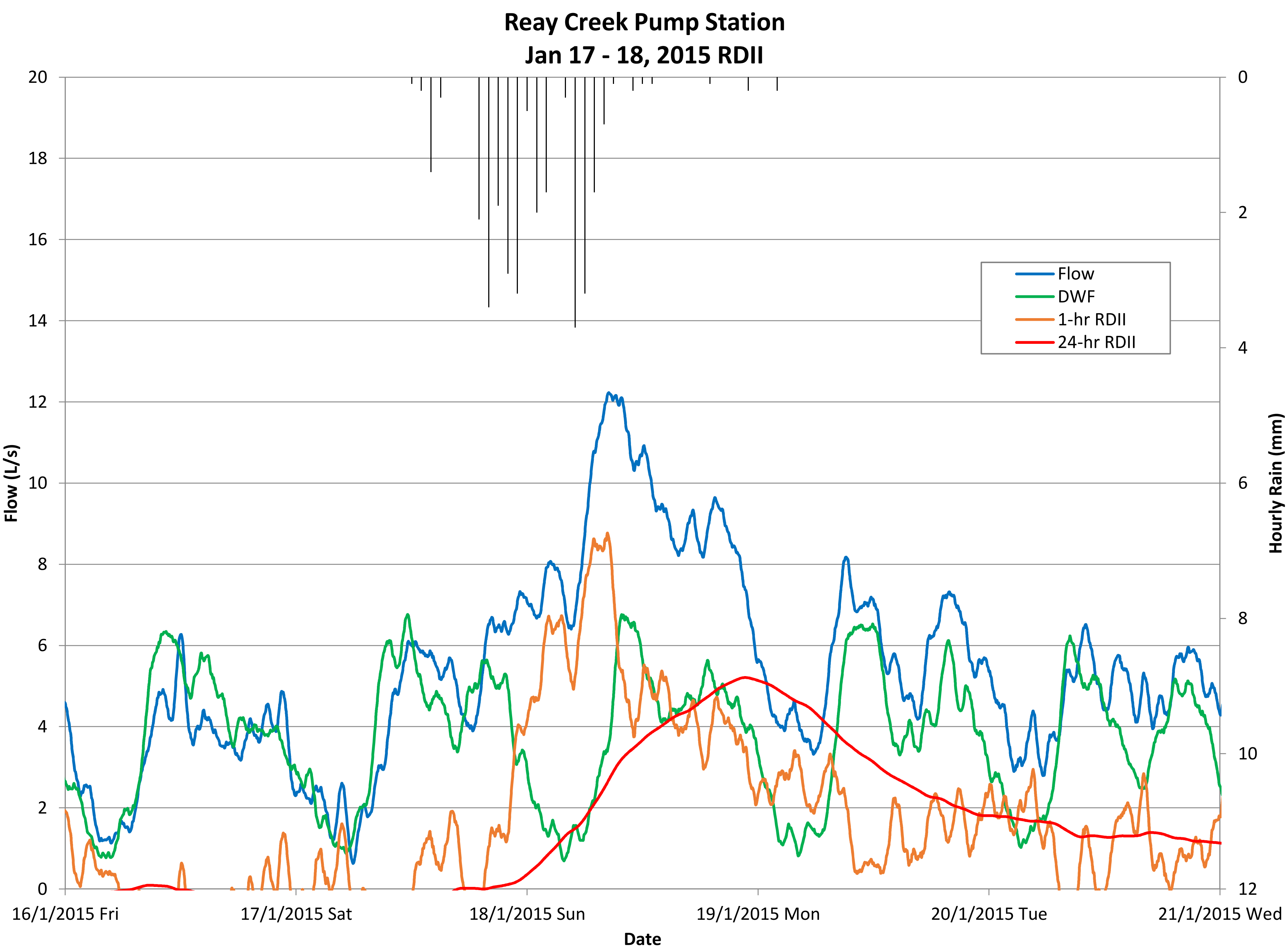
Reay Creek Pump Station
Nov 25 - 26, 2014 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
20
18
16
14
12
10
8
6
4
2
0
0
2
4
6
8
10
12
22/11/2014 Sat
23/11/2014 Sun
24/11/2014 Mon
25/11/2014 Tue
26/11/2014 Wed
27/11/2014 Thu
28/11/2014 Fri
29/11/2014 Sat



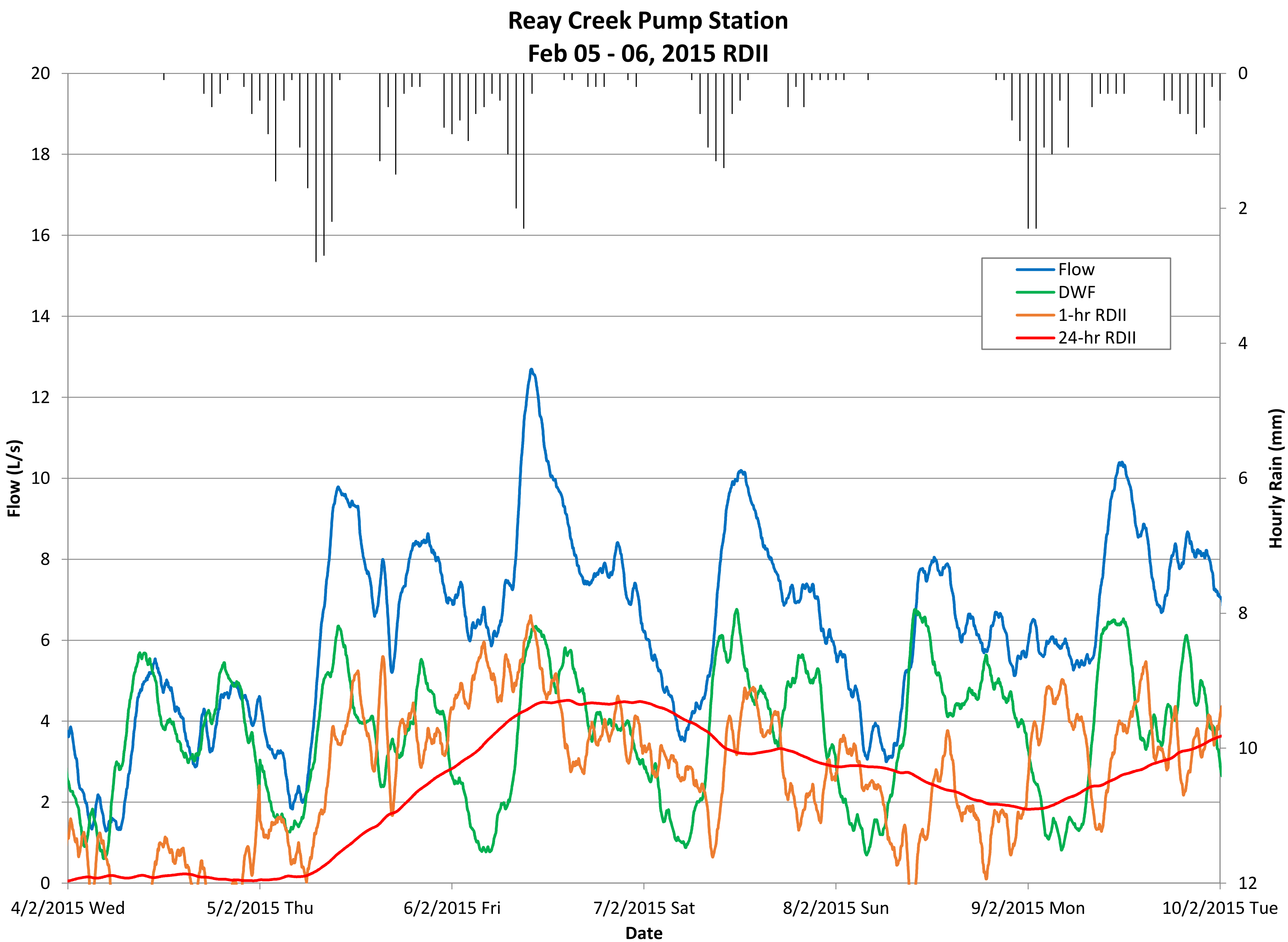
Reay Creek Pump Station
Dec 08 - 09, 2014 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
20
18
16
14
12
10
8
6
4
2
0
12
8/12/2014 Mon
9/12/2014 Tue
10/12/2014 Wed
11/12/2014 Thu
12/12/2014 Fri
13/12/2014 Sat
14/12/2014 Sun



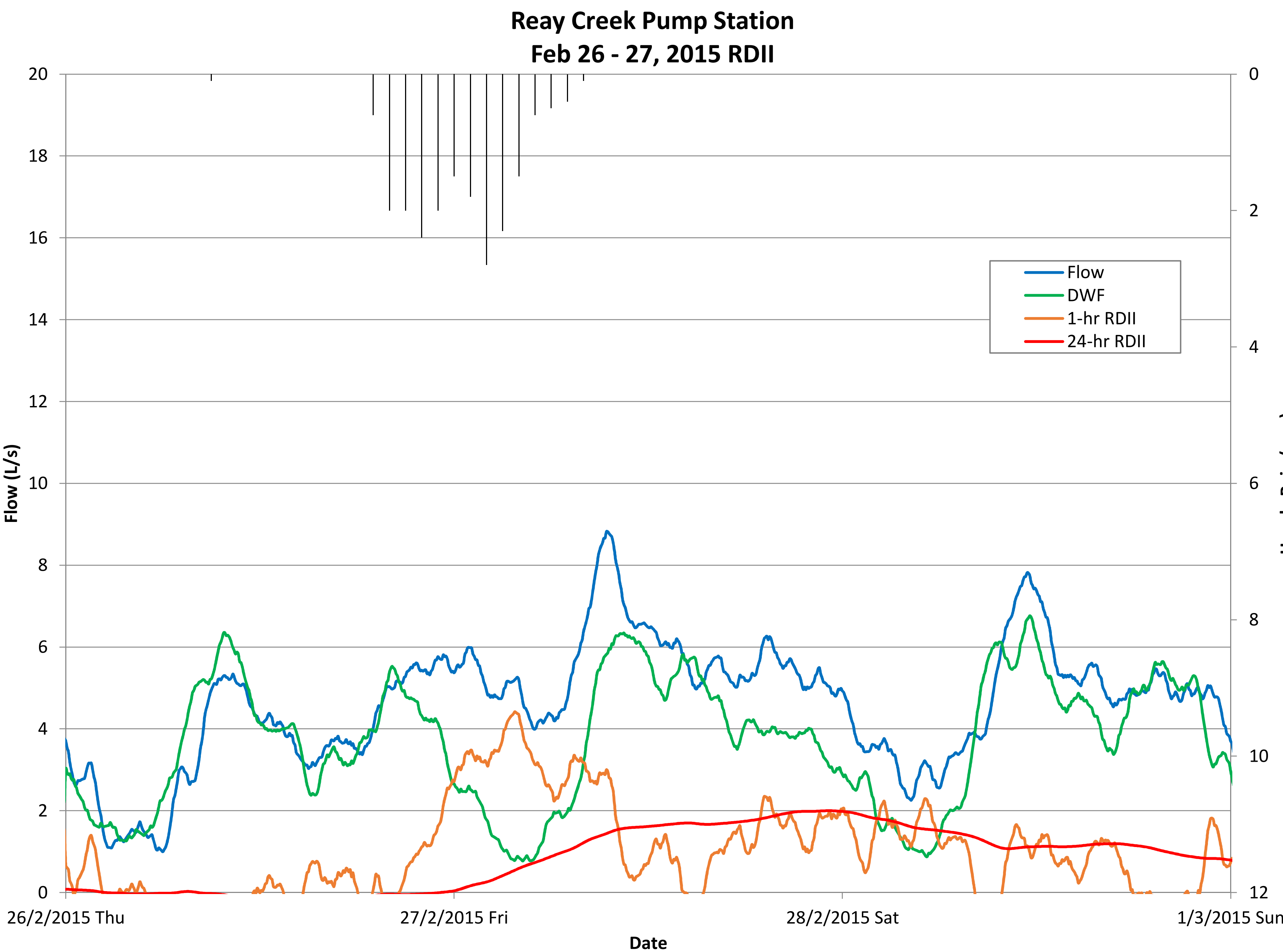
Reay Creek Pump Station
Jan 04 - 05, 2015 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
30
25
20
15
10
5
0
0
2
4
6
8
10
12
3/1/2015 Sat
4/1/2015 Sun
5/1/2015 Mon
6/1/2015 Tue
7/1/2015 Wed



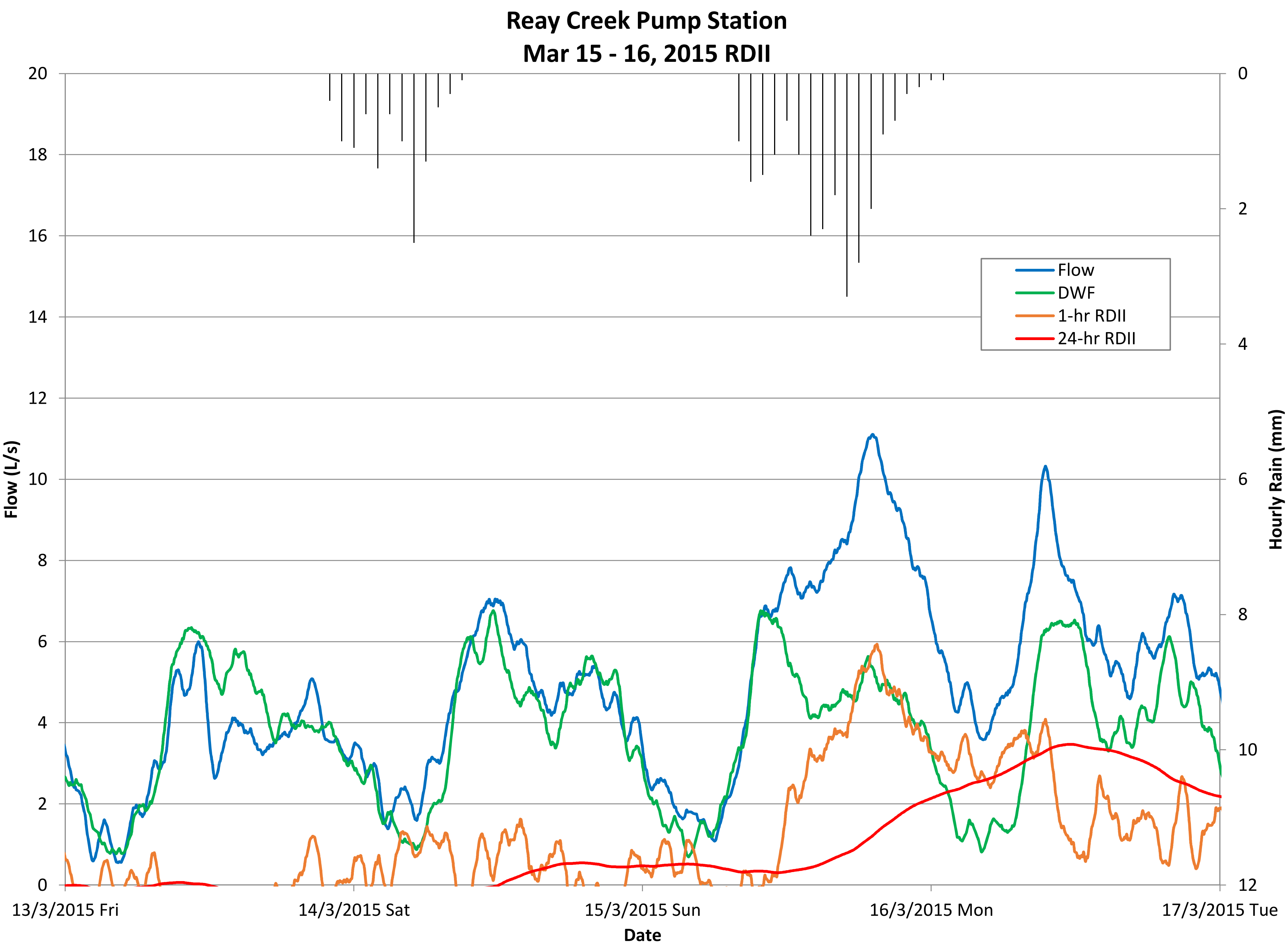
Reay Creek Pump Station
Jan 17 - 18, 2015 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
0
2
4
6
8
10
12
14
16
18
20
0
2
4
6
8
10
12
16/1/2015 Fri
17/1/2015 Sat
18/1/2015 Sun
19/1/2015 Mon
20/1/2015 Tue
21/1/2015 Wed

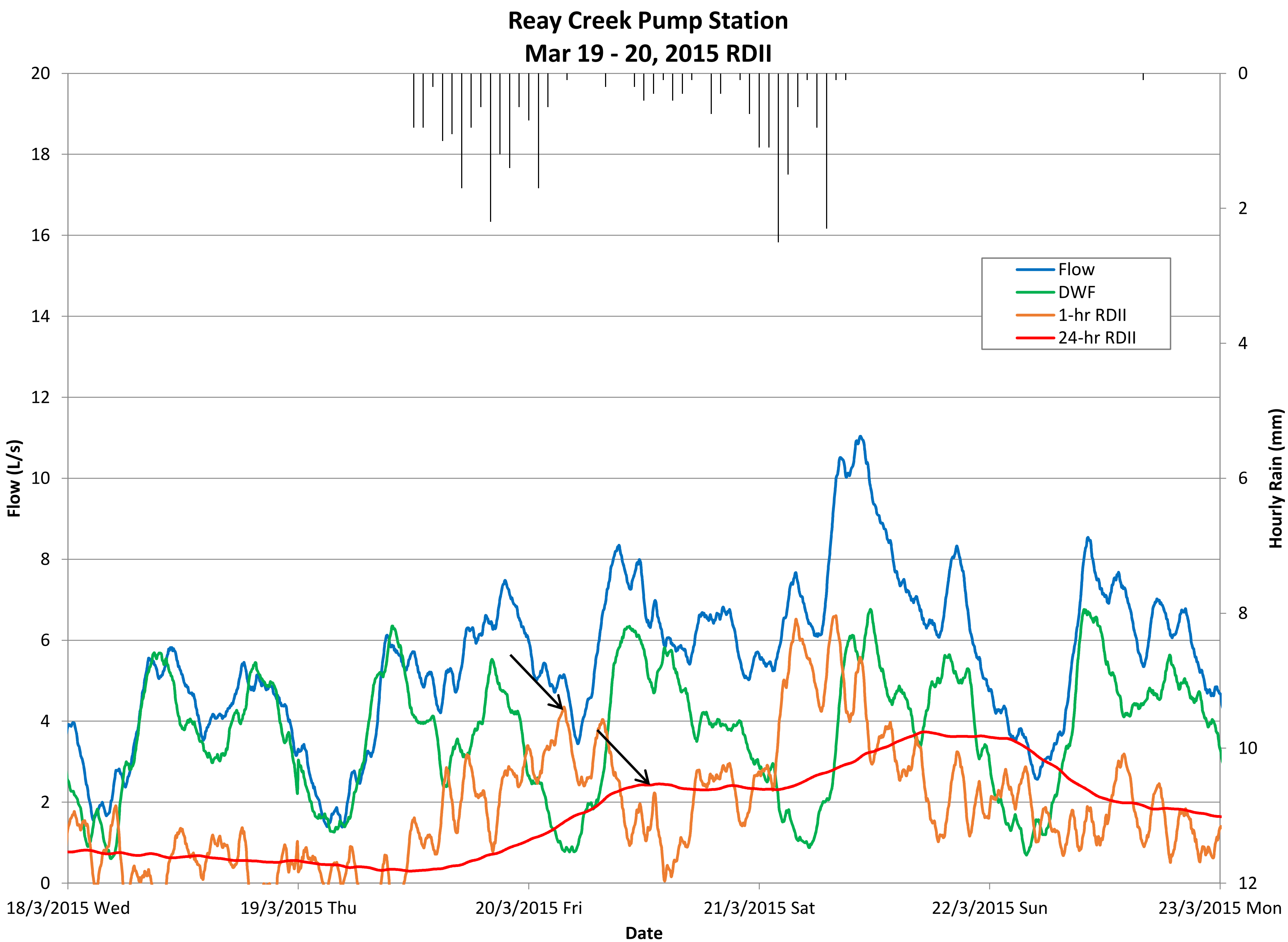


Reay Creek Pump Station
Feb 05 - 06, 2015 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
20
18
16
14
12
10
8
6
4
2
0
12
4/2/2015 Wed
5/2/2015 Thu
6/2/2015 Fri
7/2/2015 Sat
8/2/2015 Sun
9/2/2015 Mon
10/2/2015 Tue



Reay Creek Pump Station
Feb 26 - 27, 2015 RDII
Flow (L/s)
Hourly Rain (mm)
Date
26/2/2015 Thu
27/2/2015 Fri
28/2/2015 Sat
1/3/2015 Sun
Flow
DWF
1-hr RDII
24-hr RDII
0
2
4
6
8
10
12
14
16
18
20

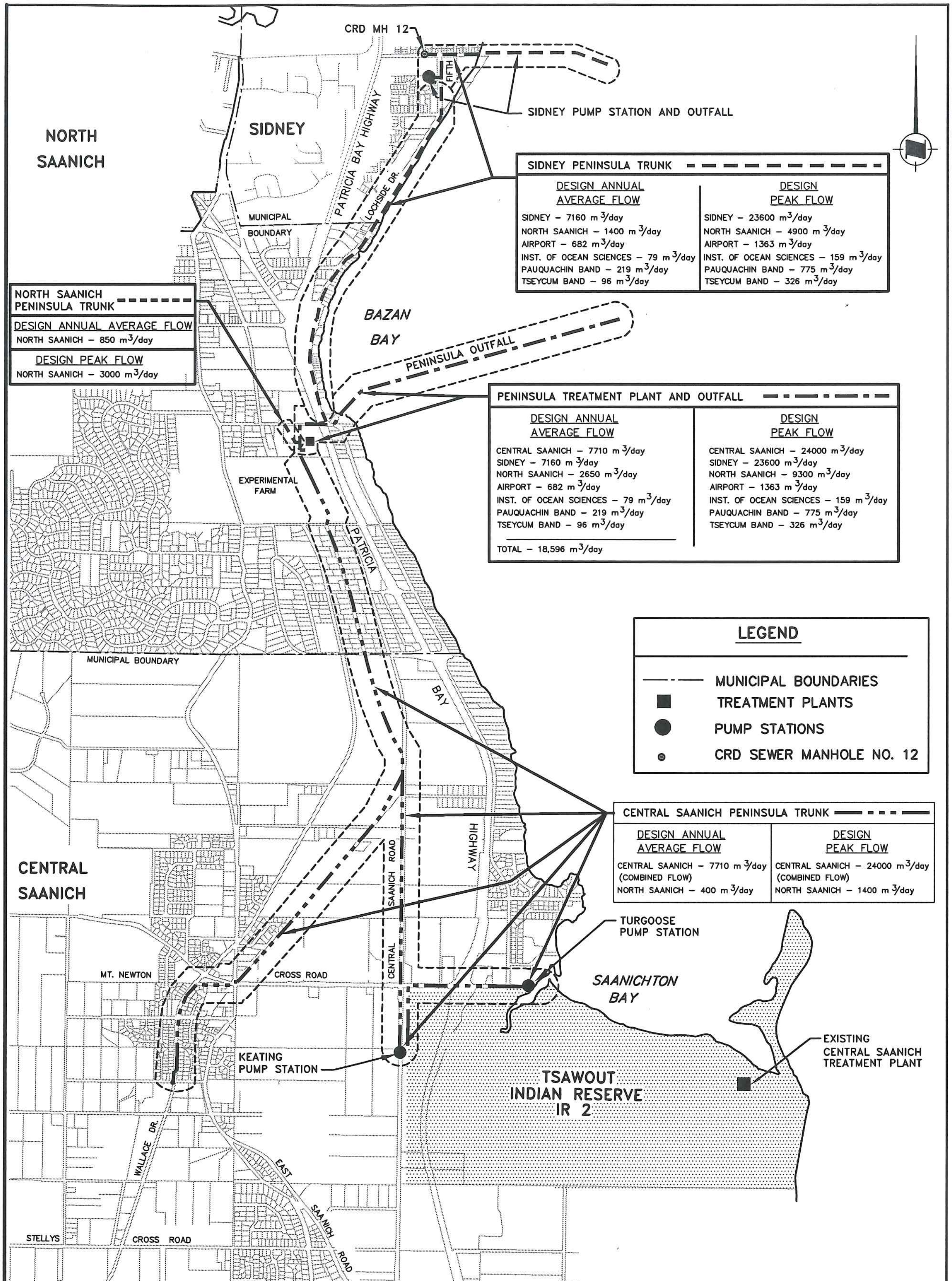

Reay Creek Pump Station
Mar 15 - 16, 2015 RDII
Flow (L/s)
Hourly Rain (mm)
Date
Flow
DWF
1-hr RDII
24-hr RDII
13/3/2015 Fri
14/3/2015 Sat
15/3/2015 Sun
16/3/2015 Mon
17/3/2015 Tue
0
2
4
6
8
10
12
14
16
18
20


Reay Creek Pump Station
Mar 19 - 20, 2015 RDII
Flow
DWF
1-hr RDII
24-hr RDII
Flow (L/s)
Hourly Rain (mm)
Date
0
2
4
6
8
10
12
14
16
18
20
18/3/2015 Wed
19/3/2015 Thu
20/3/2015 Fri
21/3/2015 Sat
22/3/2015 Sun
23/3/2015 Mon

KERR WOOD LEIDAL
consulting engineers

Appendix C

CRD Bylaw 2439 – Schedule A



CAPITAL REGIONAL DISTRICT ENGINEERING

DESIGNED	M.C.W	SCHEDULE 'A' CRD PENINSULA SYSTEM AND DESIGN FLOW ALLOCATIONS					
DRAWN	L.N.						
CHECKED							
APPROVED		DATE AUG 3/01	SCALE N.T.S.	DWG. NO. 9–S121–1		REV. 1	SHT 1 OF 1