City of Maple Ridge

Sanitary Master Plan

Final Report

Prepared by:

AECOM
3292 Production Way, Floor 4 | 604 444 6400 tel
Burnaby, BC, Canada V5A 4R4 | 604 294 8597 fax
www.aecom.com

Project Number:

60285153

Date:

November 7th, 2016

AECOM
3292 Production Way, Floor 4
Burnaby, BC, Canada V5A 4R4
www.aecom.com

604 444 6400 tel
604 294 8597 fax

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City of Maple Ridge
Engineering Department
11995 Haney Place
Maple Ridge, BC
V2X 6A9

Attention: Joe Dingwall, P.Eng.
Manager of Utility Engineering

Regarding: City of Maple Ridge Sanitary Master Plan
Final Report

Please find attached our Final Report for the Sewer Master Plan. This report summarizes the following:

- Approach to model development;
- Model calibration results and parameters used;
- Future growth projections and flow estimates (2018, 2023 and OCP);
- Hydraulic assessment and results for various time horizons; and
- Recommendations on capital upgrades

We look forward to discussing the report with you at your earliest convenience. In the meantime, if you have any questions please don't hesitate to contact me at 604.444.6400.

Sincerely,
AECOM Canada Ltd.

Distribution List

# of Hard Copies	PDF Required	Name

Revision Log

Revision #	Revised By	Date	Issue / Revision Description
1	Suman Shergill	July 20, 2014	Draft Final Report
2	Chris ODonnell	November 6, 2014	Final Report
3	Chris ODonnell	December 8, 2014	Revised Final Report
4	Chris ODonnell	April 23, 2015	Final Report
5	David Lee	November 7, 2016	Revised Final Report

AECOM Signatures

Prepared by:

Chris O'Donnell, P.Eng
Project Engineer

Reviewed by:

Jason Colenutt, P.Eng.
Project Manager

Revised by:

David Lee, P.Eng
Senior Infrastructure Planning Engineer

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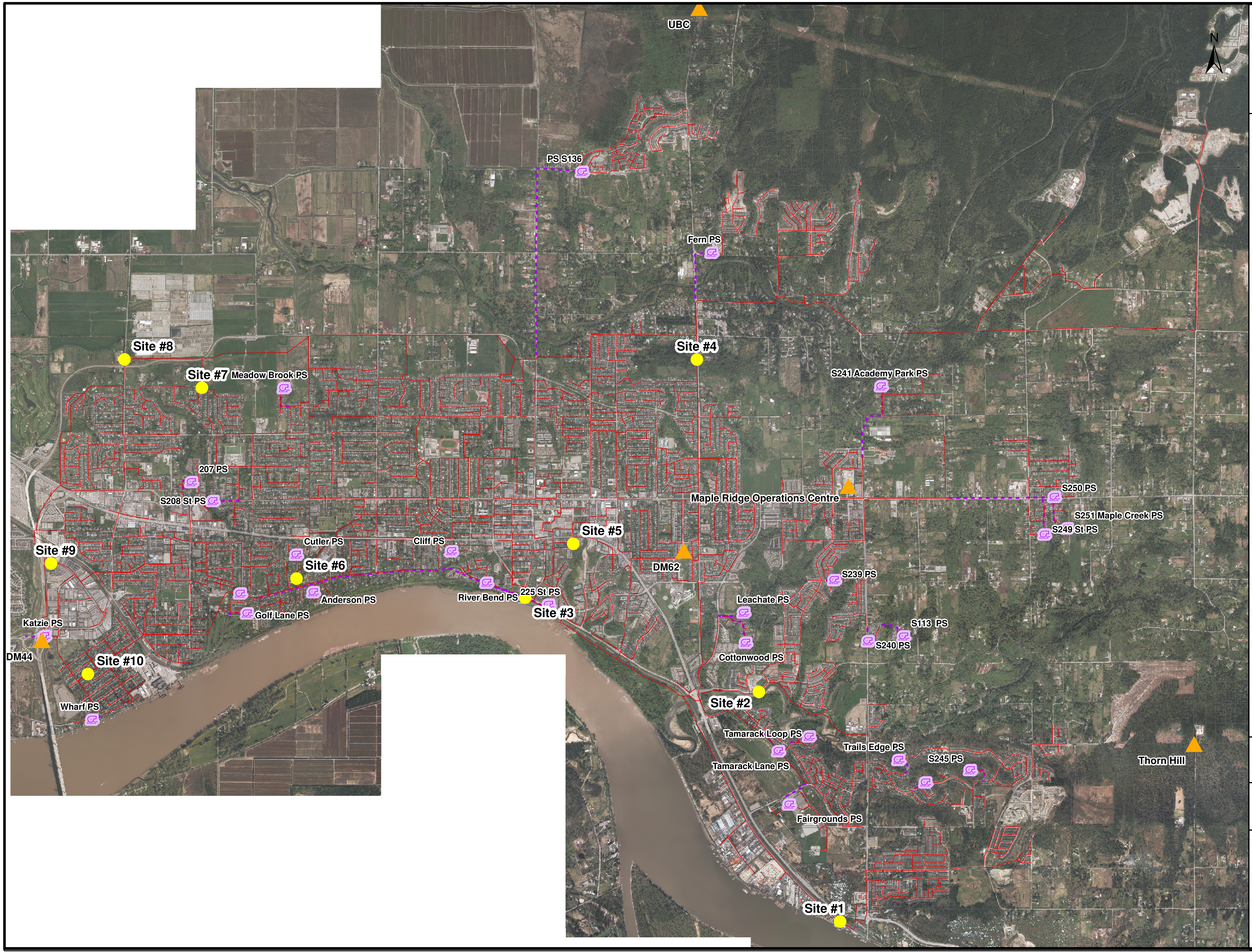
1 Introduction

1.1 Overview

The City of Maple Ridge has an estimated population of 79,142 residents according to 2013 BC Stats. The City's sanitary sewage collection system, which serves the majority of the City, is composed of 22 catchments, with 31 pump stations and over 290 km of pipes. In 2002, AECOM developed a Sanitary Master Plan for the City that focused on assessing the capacity of the sanitary trunk sewers. The model for the 2002 Master Plan was developed using HYDRA software, which was widely used at the time for sewer assessment. The City has experienced significant growth since the last master plan and there was a need to develop a more comprehensive hydro-dynamic (flow hydrology and hydraulics) model of the sewer system as part of the continuing stewardship of these assets.

The City retained AECOM to develop a new hydro-dynamic using Inovyze's InfoSWMM software package. The new model includes the City's entire sanitary sewer system excluding service laterals and private systems, up to Metro Vancouver's Katzie Pump Station. ***Figure 1.1*** shows the City's existing sanitary sewer system up to the Katzie Pump Station. As illustrated in ***Figure 1.1***, a large percentage of the City's total land area is not currently serviced by a municipal sewer system; however, these areas are primarily agricultural properties that make up approximately 15% of the total population.

With exception to the lands within the Agricultural Land Reserve (ALR), most of the City is within Metro Vancouver's Fraser Sewerage Area (FSA); this implies that most properties in the City are permitted to connect to the municipal sewer system and drain to Metro Vancouver's regional system and treatment facilities. There will be an ongoing demand to expand the City's sewer system to service the area's ultimate development potential, mainly in the Albion, Silver Valley, and Thornhill areas.



MAPLE RIDGE British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Rain Gauge
- Monitoring Site (2013)
- PumpSations
- Sanitary Sewer
- Forcemains

AECOM

0 375 750 1,500 Meters

Project No.	Date
60285153	August 2013

Existing Sewer Network

Figure 1.1

1.2 Key Objectives

The report constitutes the Final Report for the development of the City's Sanitary Master Plan. The City's key objectives for this project and report are:

1. Develop a new hydraulic model of the City's entire sanitary sewer system;
2. Calibrate the model to "existing conditions/populations" using historical dry and wet weather flow data, then validate pump station inflow/outflows;
3. Develop sanitary flow projections for various planning level horizons including: (i) Current Zoning (ii) 2018 (iii) 2023; and (iv) OCP;
4. Characterize I&I (and RDII) in specific catchments;
5. CCTV database review and recommendations;
6. Assess the hydraulic capacity of the sewer system under various planning level time horizons; and
7. Provide City Staff with InfoSWMM model / software training(optional), focused on attributes of their new model.

1.3 Past Technical Memorandums

Over the course of this project, a series of Technical Memoranda have been prepared at the conclusion of milestone activities. Key information from each Technical Memorandum has been extracted, updated, summarized, and incorporated into this report; however a full copy of each Technical Memorandum and their associated tables, figures and appendices has not been included herein.

The following three Technical Memoranda are fundamental deliverables in the development and calibration of the model. These memoranda form part of our overall report and are included by reference. For each milestone activity, refer to the appropriate sections in this report for updated synopsis/results and then subsequently, any background information included in the following memoranda:

- Technical Memorandum #01 –Model Development and Calibration, dated August 30, 2013.
- Technical Memorandum #02 –River Road and Town Center Sewer Capacity Analysis, dated October 08, 2013.
- Technical Memorandum #03 –Albion Flats & North Albion Sanitary Servicing Strategy, dated January 27, 2014.
- Technical Memorandum #05 –Inflow & Infiltration Assessment, dated October 20, 2014.
- Technical Memorandum #06 –Thornhill Urban Reserve Assessment Sanitary Servicing Strategy, dated October 22, 2014.

1.4 Key Terms and Abbreviations

A list of key terms and abbreviations along with their definitions is presented in ***Table 1.1***.

Table 1.1 – Key Terms/Abbreviations

Term	Definition
Average Dry Weather Flow (ADWF)	The lowest 24-hour average sanitary flow value during a 7-day period of dry weather. The sanitary flow is composed of base sanitary flow plus groundwater infiltration (ADWF = BSF + GWI).
Base Sanitary Flow (BSF)	All wastewater flow from residential, commercial, industrial and institutional sources that the sanitary sewer system is intended to carry. (BSF = ADWF – GWI)
Diurnal Pattern	Pattern describing the variance in sewage flows over a day
Fraser Sewer Area (FSA)	Metro Vancouver's catchment area / boundary that identifies all properties that are permitted to discharge sewage to the Regional System. Areas outside the FSA are assumed to not be permitted to connect to a municipal sewer.
Groundwater Infiltration (GWI)	Groundwater infiltration that enters the sanitary sewer system during dry weather periods; through breaks, cracks, misaligned joints, tree root punctures and manhole joints and covers. In general, GWI = 70 - 85% of minimum night-time flow.
Greater Vancouver Regional City (GVRD)	Regional City whose trunk system collects all sewage from the City, and neighbouring municipalities, and conveys it to a treatment facility
Hydraulic Grade Line (HGL)	The maximum level of water in the pipe system, calculated as the height that liquid will rise in a piezometer using the Bernoulli's Equation
Inflow	Stormwater that enters the sewer through direct connections (i.e. CB leads or roof drains connected to the sanitary sewer)
Inflow and Infiltration (I&I)	The total inflow and infiltration that enters the sanitary sewer system from all sources, equal to GWI + RDII
Metro Vancouver (MV)	Same entity as GVRD (Regional City)
Peak Dry Weather Flow (PDWF)	Peak instantaneous sanitary flow value during dry weather conditions (peak of the diurnally varying BSF plus normal GWI).
Peak Wet Weather Flow (PWWF)	Maximum instantaneous sanitary flow value. It represents all flow contributions carried by the sanitary sewer system (equals PDWF + RDII).
Rain Dependent Inflow and Infiltration (RDII)	All stormwater inflow (see above) into the sanitary sewer system plus increase in GWI that occurs directly due to the influence of rainfall
RTK	A synthetic unit hydrograph technique used by InfoSWMM and SSOAP to quantity and simulate RDII. The R parameter is the fraction of rainfall volume entering the sewer system as RDII, T is the time to peak, and K is the recession time/ratio
Sanitary Sewer Overflow (SSO)	Non-frequent occurrence when sewage backs-up, surcharges and overflows from the municipal sewer system.
Sanitary Sewer Overflow Analysis and Planning (SSOAP)	A software program / toolbox developed by the US EPA. The software is used to quantify RDII using a hydraulic approach and RTK method.

2 Model Development & Calibration

2.1 GIS Integration

Prior to importing the sewer data into the model, a review of the GIS network data was completed to identify all "data gaps". We were able to reconcile the majority of the GIS data gaps using information provided by the City.

Model connectivity gaps were reviewed using connectivity tools available in InfoSWMM to ensure all pipes connect to manholes. Pipe slopes were reviewed in GIS prior to importing into the model to make sure pipes have positive slopes, particularly at those manholes where inverts were originally missing in GIS. Common connectivity gaps found, and corrected, include the missing upstream or downstream pipe inverts, isolated manholes with no connecting pipes, pipes with no assigned upstream/downstream manhole ID, manhole inverts above the pipe invert, and the manhole missing invert or ground elevations.

In order to incorporate the data gaps found during model development, AECOM will provide shape files showing updated invert and rim elevations for all the manholes and pipes in the model. The shape files will only be for those elevations that were changed, and not a reiteration of the entire sewer system, so that the City can review the data.

2.2 Model Development

2.2.1 Sewer Network

Once the GIS data gaps were resolved, the sewer network attributes were imported into InfoSWMM. Data imported into the model included pipe ID, diameter, pipe inverts, length and material type (Manning's roughness coefficient), manhole ID, inverts, ground elevations and "X-Y" coordinates. Pipe offsets (i.e. inverts at drop manholes) were inserted in the model using the corresponding pipe and manhole inverts.

2.2.2 Pump Stations

In total, 28 City operated pump stations (out of total 31) were incorporated into the model. The S232, S236, and Nelson Peak pump stations were not included in the model because these stations are located in very new developments and/or are not in use yet (sewer network information was not available for these areas).

2.2.3 Boundary Conditions

Sewage from the City eventually discharges into Metro Vancouver's (MV) Katzie Pump Station. The Katzie Pump Station also receives wastewater from areas of Pitt Meadows. The station is comprised of 4 centrifugal pumps installed in a parallel configuration. Three of the pumps are powered by 250 hp electric motors, and one by a 100hp electric motor. The Katzie PS discharges to Metro Vancouver's Maple Ridge Forcemain (MRFM), which in turn, discharges to the North Surrey Interceptor. The *Katzie Pump Station Pump and Piping Upgrade Study* (AECOM, 2013) estimated the current firm capacity of the pump station to be 700 L/s. Basic Service Flow according to Metro Vancouver's criteria is the peak dry weather flow plus the standard I&I allowance of 11,200 L/ha/day. According to MV, Katzie's 2011 Basic Service Flow (BSF) is 977 L/s, and the 2061 BSF is projected to be 1660 L/s; therefore, a station capacity upgrade is already required. MV is in the process of upgrading the North Surrey Interceptor, however the timing of other future upgrades (including the pump station) is unknown at this time. Modelling of the Katzie Pump Station collection and discharge system is beyond the scope of this study.

Due to capacity constraints further downstream of Katzie, surcharging of the North Surrey Interceptor (Port Mann Section) can occur during wet weather. In order to avoid sanitary sewer overflows (SSO) to Bon Accord Creek, MV reduces the discharge from the Katzie Pump Station during significant storms (which can be less than the 1:5yr storm). This reduced pumping results in sanitary flows backing up in Maple Ridge's North Slope and South Slope Interceptors.

MV is currently designing plans for construction of an SSO containment tank, with the aim of preventing SSOs from rainfall events with a return period of less than 5 years, as well as allowing the 225 Street Pump Station to operate normally. However, currently the MV's Katzie Pump Station operating procedures dictate that flows at the station are controlled by operators during significant storm events. When the sewage level at Katzie reaches a depth 4.57m (- 0.66m elevation) MV operators at Katzie coordinate with operators at the City's 225 Street Pump Station to reduce flows into the Katzie Pump Station. This is done to prevent overflows to the Katzie Slough as well as overland SSOs to Bon Accord Creek in Surrey, which is accomplished by turning off some of the pumps at the 225 Street station. If the levels continue to rise at Katzie, then the 225 Street PS is shut down, resulting in an overflow to the Fraser River. Several parties are notified by Metro Vancouver via e-mail as soon as a sewage overflow begins and again when it ends. They are also provided a summary report which includes an environmental assessment. Notified parties include Emergency Management BC (EMBC), Environment Canada, BC Ministry of Environment, BC Ministry of Agriculture and the Fraser Health Authority.

For modelling purposes, three boundary conditions were used:

- **Scenario 1- Katzie Pump Station modelled at current capacity:** In this scenario the station outflow is set to the maximum rated capacity of the station (700 L/s), with on/off levels matching MV settings. Under this condition there is regular sewage backups into the upstream interceptor sewers. This scenario is used for the model capacity assessments under current, and interim scenarios (2013, 2018, 2023).

- **Scenario 2- Katzie Pump Station modelled as an Ideal Pump:** In this scenario modelled outflow from the pump station is the same as inflow. Under this condition there is no backup into the upstream network. This scenario is used to assess the ultimate OCP capacity of the upstream sewer network.
- **Scenario 3- Katzie/225 Street Pumps Station turned off, Sanitary Sewer Overflow (SSO):** To study the impact of a worst case scenario, upstream HGL will be reviewed during the design storm when the Katzie and 225 Street pump stations are shut down

2.3 Existing Sewage Flow Generation

This section describes the process used to generate sewage flows for the calibration scenarios, which was translated into the flow rates for the "existing land-use, actual flows" scenario. Planning level scenarios such as: 2018, 2023, and OCP are discussed further in this report under Section 3.

2.3.1 Flow Allocation

Sanitary flow allocation was determined on a parcel by parcel basis (i.e. each parcel/lot is its own sanitary catchment). The City provided the existing land use information at the lot level in the form of a shape file. Each parcel has a unique Identification number (Oracle PIN) which was used to identify & assign flows in the model. The shape file also contained information about the number of units within a particular parcel, and all units within a parcel have the same Oracle PIN. The existing residential population was based off the 2011 census data. The census population is available in the form of "dissemination blocks", for which the City is divided into 527 blocks. Population was assigned to each parcel/unit based on existing land use. To compare the AECOM assigned population with the census block data, parcels were grouped into the same boundaries. The difference between total estimated population and the census population was less than 1%.

Once flows were generated in GIS, the catchment (or parcel) was then automatically allocated to the nearest fronting sewer manhole using tools within InfoSWMM.

2.3.2 Residential Land-Use

Residential unit flow rates were then determined based on observed flows from the 2013 flow monitoring data supplied by SFE Global. Average DWF (ADWF) was first determined based on the dry weather week of May 4th to 11th, 2013. Residential flow component of ADWF was then determined by subtracting ICI and GWI flows from ADWF. Serviced residential population for each flow monitoring catchment was determined from the City's land use shape file using GIS tools. All houses with fronting sewer are assumed to be serviced.

Sites 8, 9 and 10 (see ***Figure 1.1***) are the three primary locations that represent 94% of the total serviced population in Maple Ridge (approximately 66,200 people). The total serviced population of Sites 8, 9 and 10 catchments is estimated at 62,000 people. ***Table 2.1*** shows the calculated residential sewage flow rates on a unit rate basis (L/cap/day) for the above three catchments. The weighted average base sanitary flow across the above flow monitoring sites is 191 L/capita/day.

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Table 2.1 – Observed Sewage Flow for Three Primary Sites

Site#	ADWF (Observed Data) (L/s)	GWI (L/s)	ICI flow (L/s)	Residential Flow (L/s)	Residential Pop.	Residential Rate (L/Capita/Day)
Site 8	83.0	20.0	7.6	55.4	25,761	186
Site 9	11.1	1.6	3.0	6.5	3,258	172
Site 10	118.2	13.6	29.0	75.6	33,010	198
Total Serviced Population					*62,029*	*191*
Site 1	7.6	2.2	0.7	4.6	2,716	147
Site 2	18.5	3.7	0.8	14	7,967	152
Site 3	12.2	1.6	4.6	6.0	3,135	166
Site 4	14.4	2.6	1.2	10.7	5,110	180
Site 5	4.6	0.1	2.7	1.7	596	252
Site 6	21.5	7.8	4.7	7.1	5,284	146
Site 7	11.3	3.4	0.7	7.1	3,224	191
Total Serviced Population					*28,031*	--

Table 2.1 also shows the summary of sanitary flows and populations for the remaining 7 flow meter catchments. These catchments are smaller in size as compared to the above three locations and are located upstream of them. Results show the residential base sanitary flow rate varies from 147 to 252 L/capita/day in these catchments. Variations in per capita flow rates are more noticeable in catchments with smaller populations since they are more sensitive to fluctuations in monitored flow. These 7 flow monitoring sites are captured in the three (3) primary flow monitoring sites (8, 9, and 10)

2.3.3 ICI Land Use

ICI flow rates were calculated from water consumption records provided by the City. The average sanitary flow was assumed to be 100% of average water consumption during the winter (based on January to March, 2012 water meter records). This period was selected to factor out irrigation, which is not typically done during winter months.

2.3.4 Groundwater Infiltration

Groundwater infiltration (GWI) is the non-rainfall dependent flow that enters the sewer system through holes in the sewers and manholes, misaligned joints and service laterals. Determining the rate of GWI can be complicated and highly variable across the City because GWI varies with:

- pipe age because as pipes age they deteriorate;
- pipe conditions such as cracks, joint dislocations etc;
- material type because some pipe materials are brittle (AC and concrete), and some pipes have gasketted joints to minimize infiltration;
- amount of pipe in the catchment (diameter and length), because the more pipe and surface area in contact with groundwater will result in more infiltration;
- depth of groundwater table relative to sewer pipe;
- number of service connections, with the more connections resulting in a higher rate of infiltration; and
- subsurface soil type and location of pipe relative to till/clay layers. If a pipe is at deeper depths and installed above an impermeable layer, the groundwater table is most likely elevated and can cause infiltration whereas a pipe installed at shallower depths in sandy soils will tend to have a lower groundwater table since the material is free draining.

In addition to the sanitary flow, Ground Water Infiltration (GWI) was added based on parcel areas and unit rates determined from each flow meter catchment. Groundwater infiltration (GWI) estimates were determined using the night-time flow monitoring data during dry weather period for each flow meter catchment. ***Table 2.2*** shows the observed GWI rate for each flow meter, which have been applied to the model. The location and the catchment area (sum of serviced lot areas excluding roads & green space) is also included. For catchments with no flow monitoring data, the GWI rate of the adjacent metered area was applied. ***Figure 2.1*** shows the observed dry weather GWI variation across the City, and the location of the 10 temporary flow monitoring sites.

Table 2.2 – Measured GWI Rates

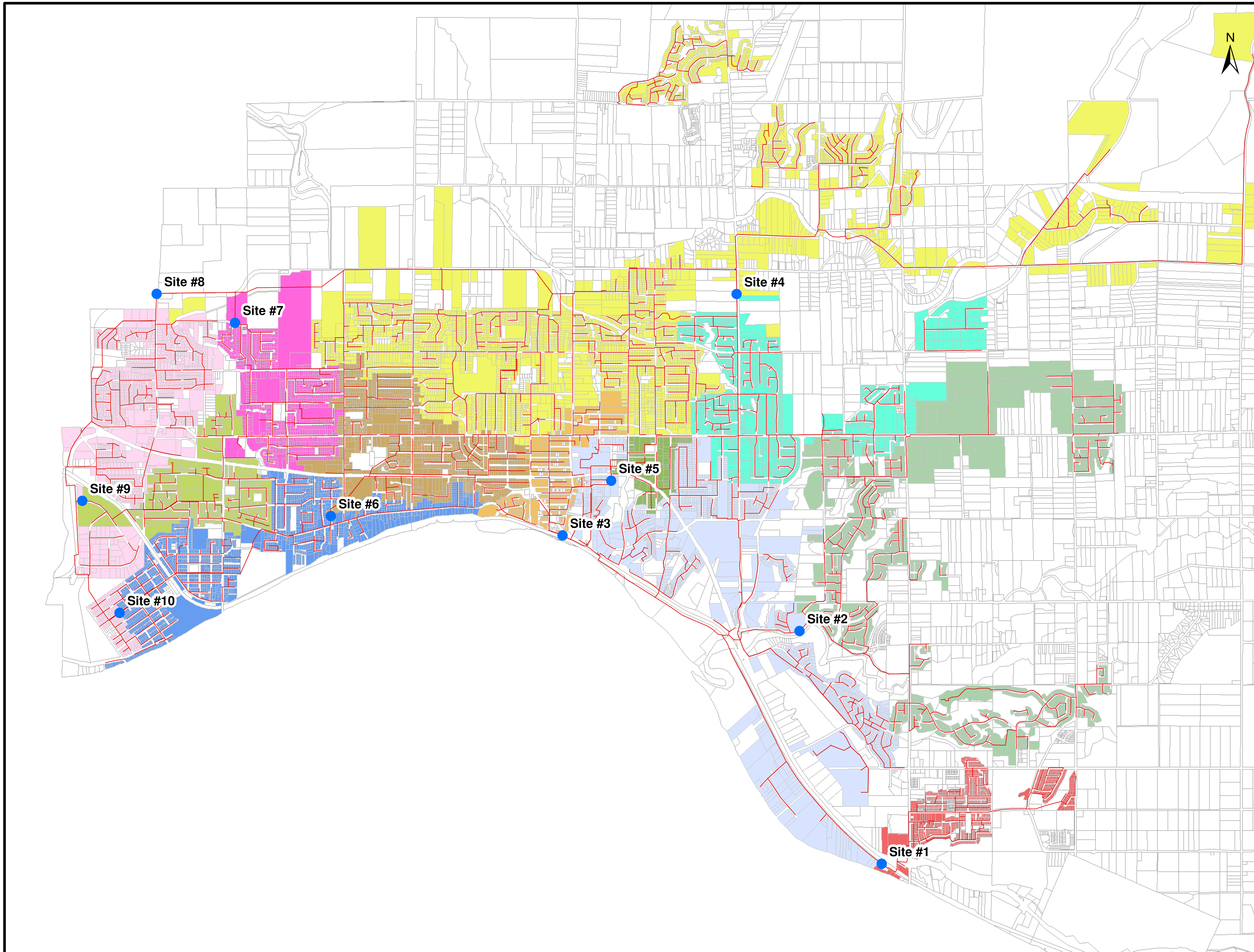
Site #	Location	Serviced Area (Ha)	GWI (L/s)	Dry Weather GWI (L/Ha/Day)	Wet Weather GWI (L/Ha/Day) (2)
Site 1	River Rd.	38.8	2.2	4,989	4,989
Site 2	Kanaka Way	276.2	3.7	1,151	1,151
Site 3	Haney Bypass	44.4	1.6	3,117	3,117
Site 4	232 St.	163.8	2.6	1,350	1,870
Site 5	Royal Cres.	22.4	0.1	555	555
Site 6	212 St.	127.9	7.8	5,294	6,649
Site 7	Powell Ave.	119.6	3.4	2,484	2,484
Site 8(1)	203 St.	668.5(1)	19.1	2,466	3,763
Site 9	Stewart Cres.	86.9	1.6	1,590	1,590
Site 10(1)	Lorne Ave.	(u/s of 225st PS) 285.8(1)	4.1	1,228	2,750
		(d/s of 225st PS) 142.3(1)	4.1	2,481	5,486
Not Metered	Varies	165.8	--	2,476	2,476

(1) Catchment area shown is downstream of other flow meter locations

(2) Refer to Wet Weather Calibration Section 4.2

GWI of metered areas range from 555 to 5,294 L/ha/day, with a weighted average of 2,342 L/ha/day. For the three major catchment areas (Sites 8, 9, 10) the average GWI was calibrated to be 2,201 L/ha/day. The detailed results of GWI rates for each metered catchment are presented in our **Technical Memorandum #01.**

During wet-weather, saturated soil conditions result in a shallower groundwater table, which in turn can increase GWI compared to dry weather. Therefore, we made further adjustments to GWI rates where required based on observed flow during wet weather weeks in March and April 2013. Analysis of observed flow showed that there was a significant increase in the ground water infiltration rate (GWI) from dry months to wet winter months at two monitoring sites (Site 6 and Site 8). As a result, a separate Wet Weather GWI data set was created for the sites affected during wet months and applied for wet weather calibration to simulate increased infiltration that results from saturated soil conditions. As listed in **Table 2.2**, Sites 4, 6, 8, & 10 have separate Wet Weather GWI values.



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Monitoring Site (2013)
- Sanitary Sewer

GWI (L/Ha/d)

- Site 1 : 4,989
- Site 2 : 1,151
- Site 3 : 3,117
- Site 4 : 1,350
- Site 5: 555
- Site 6 : 5,294
- Site 7 : 2,484
- Site 8 : 2,466 (Excluding 4 &7)
- Site 9 : 1,590
- Site 10 : 1,228(u/s of 225 PS)
- Site 10 : 2,481(d/s of 225 PS)
- Unmetered : 2,476

AECOM

0 375 750 1,500 Meters

Project No.	Date
60285153	August 2013

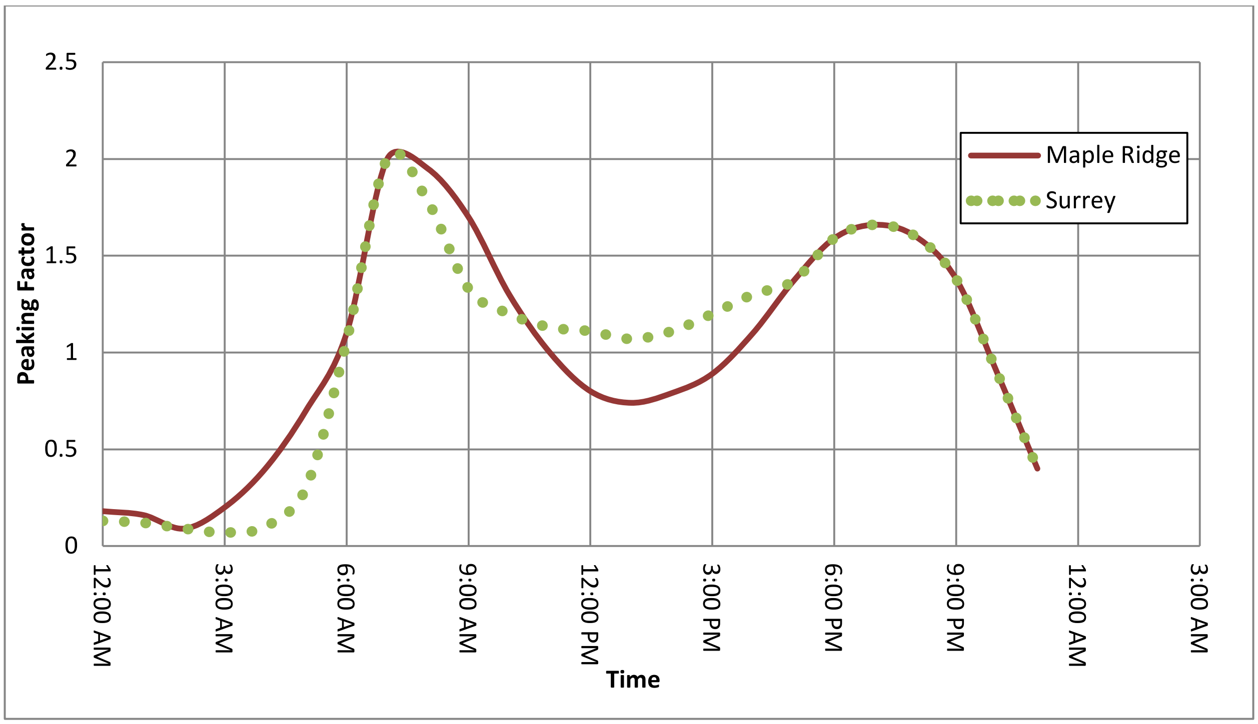
Dry Weather GWI Rates

Figure 2.1

2.3.5 Diurnal Curve

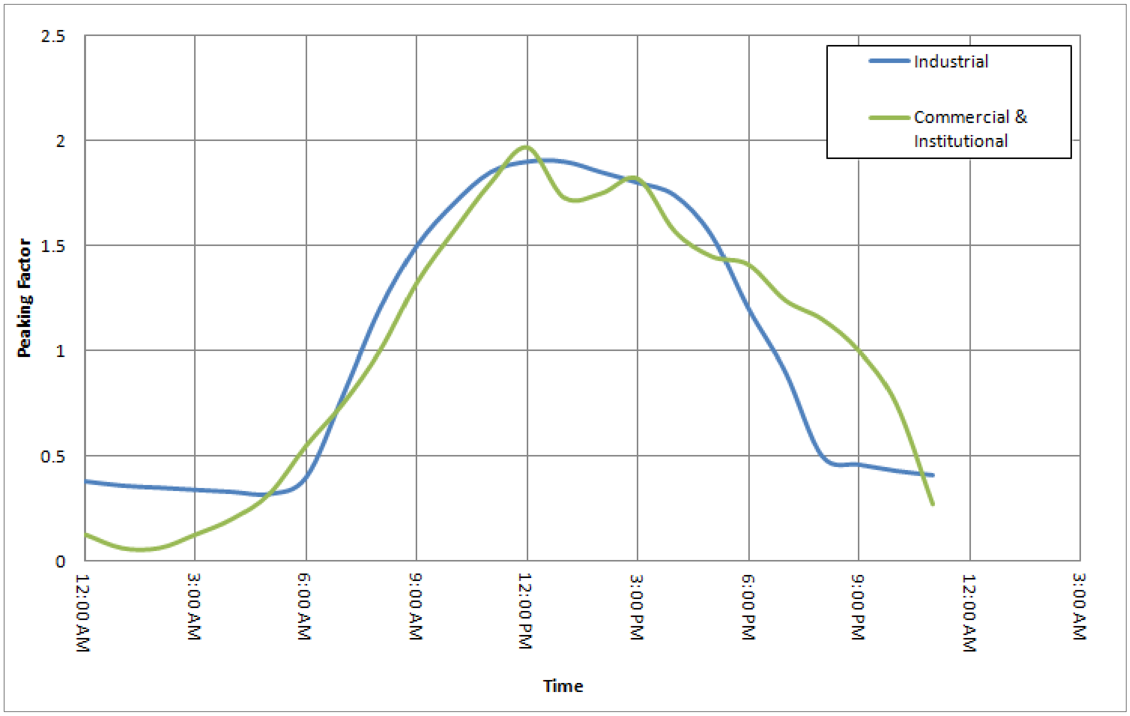
Diurnal curves for sanitary flows, for each land use, were determined from the 2013 flow monitoring data. For single family and multi-family land-use, the typical diurnal pattern was developed using a 5 day average of the flow monitoring at Site 2 which has a residential population of approximately 8,000(mostly single family), and limited flows from ICI. The 5 day period selected was in May, 2013 and no rainfall was observed for at least one week before this period. The residential diurnal curve shown in ***Figure 2.2*** was then incorporated into the model. The results revealed a peaking factor of 2.0 for base residential sanitary flows. Overall, this factor is slightly lower than, but in-line with residential sanitary peaking factors we have observed in other master planning assignments in Surrey, Township of Langley and Richmond. The peaking factor of 2.0 is reasonable for moderately sized catchments, but non-conservative for small catchments and local sewers. It is recommended that the peaking factor should be reviewed in greater detail for local sewers (defined as having a total capacity less than 40 L/s) as part of the development permitting process for future developments.

Figure 2.2 Residential Diurnal Curves



For commercial land-use, the typical diurnal pattern was developed using a 5-day average of the flow monitoring at Site 5. There were no flow meters available to represent 100% institutional land-use; therefore, the institutional pattern was assumed to be the same as the commercial pattern and was applied to all schools, government buildings, senior care facilities, and hospitals. The temporary flow monitoring program in the City did not isolate any catchments that were solely industrial land-use. For industrial areas in Maple Ridge, we applied the diurnal patterns that we recently developed for the City of Surrey. For industrial land-use, the curve was developed from 10 days of flow monitoring data from a meter in North-Surrey. The monitored area was of primarily industrial land use, similar to the Hammond industrial park in south-western Maple Ridge. ***Figure 2.3*** shows the industrial, commercial & institutional diurnal curves incorporated into the model.

Figure 2.3 ICI Diurnal Curve



2.4 Dry Weather Calibration Results

This section summarizes the calibration and validation results for dry-weather conditions. The model was calibrated at 10 flow monitoring sites throughout Maple Ridge. The site locations are shown in ***Figure 1.1***. Calibration was completed using a full hydrograph approach (i.e. not just peak flows). Model parameters such as pump discharge, diversion structures, diurnal factors and groundwater rate were adjusted to match night time flows, peak flows, hydrograph shape and time to peak.

A comparison of model predicted flows against the observed flow for dry weather periods was completed at all flow monitoring sites for a 7 day period (May 4 to May 11, 2013). Sites 8, 9 and 10 are the three major flow monitoring locations that represent approximately 94% of the total serviced population in Maple Ridge. Hydrograph plots comparing the model predictions with the measured flows from the above 3 benchmark locations are shown in ***Figure 2.4*** through ***Figure 2.6***.

Please refer to *Technical Memorandum #01 for d*ry weather calibration plots for the remaining 7 sites and tables summarizing the peak flows and volumes . A good correlation was achieved between the observed flow hydrographs and model predicted results, in terms of peak flows, night time flows and daily volumes.

Figure 2.4 Dry Weather Calibration Results – Site 8

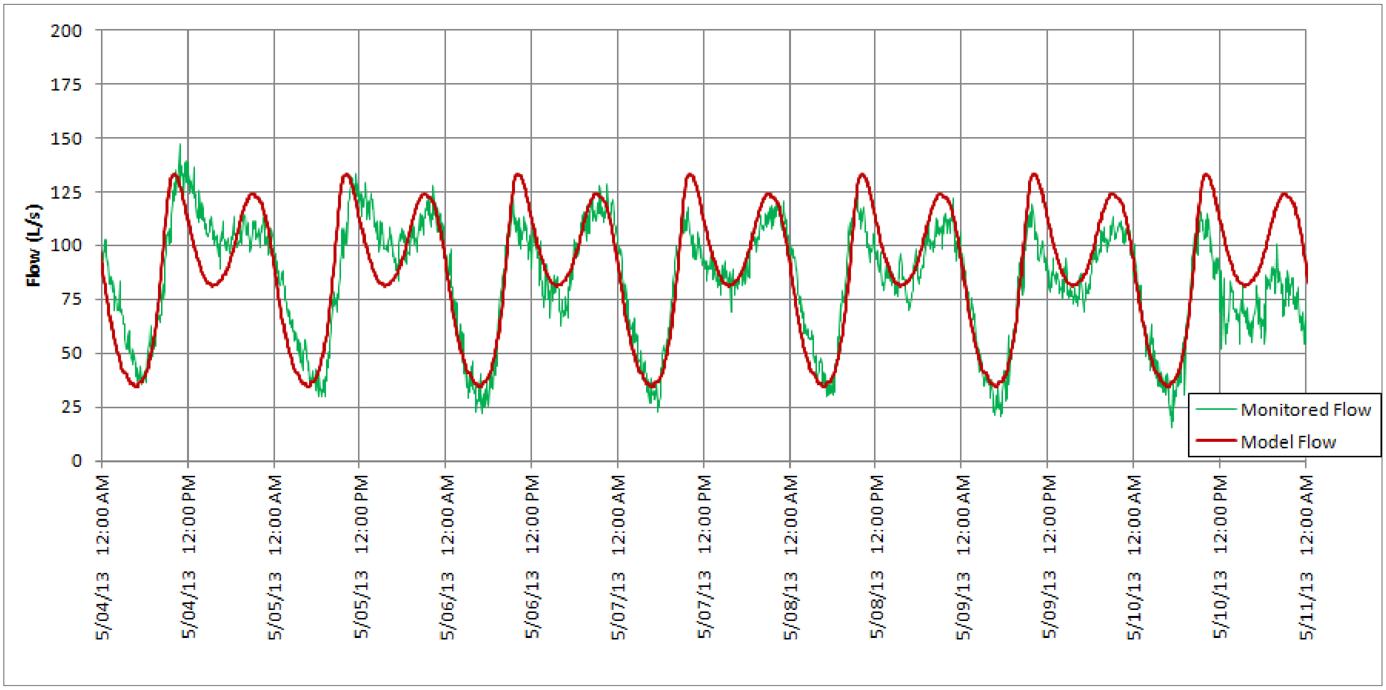


Figure 2.5 Dry Weather Calibration Results – Site 9

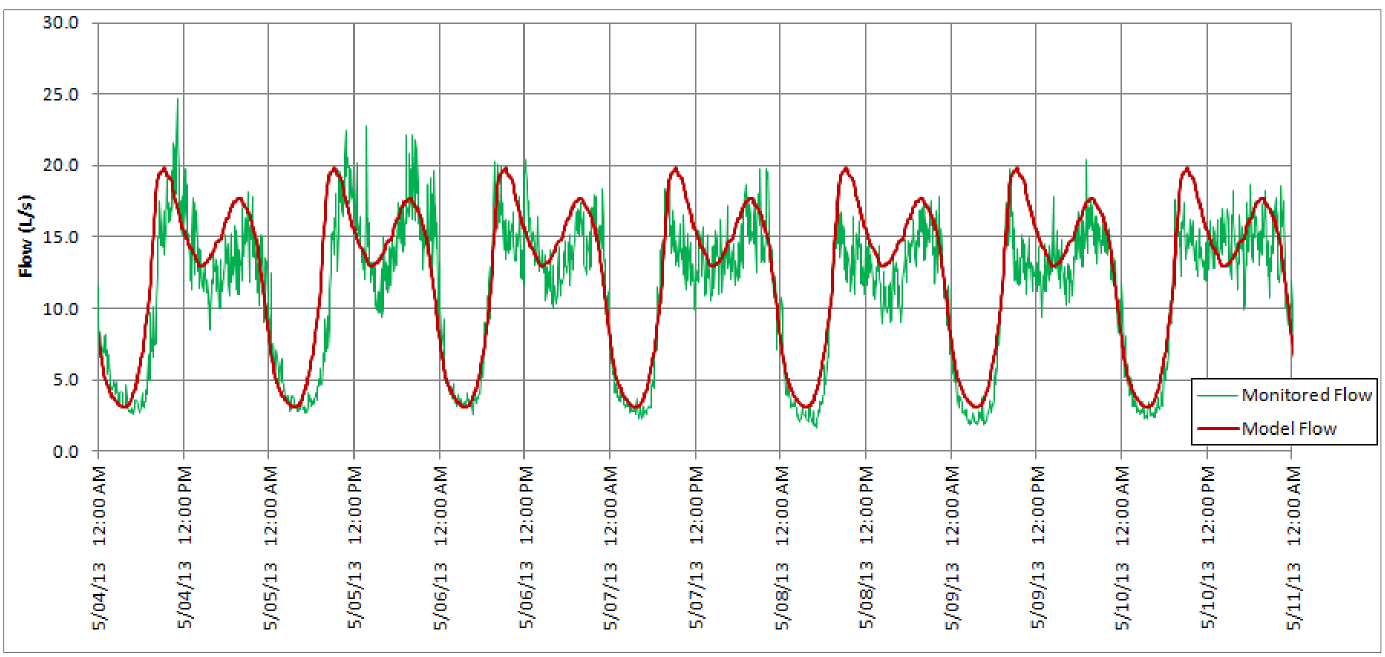
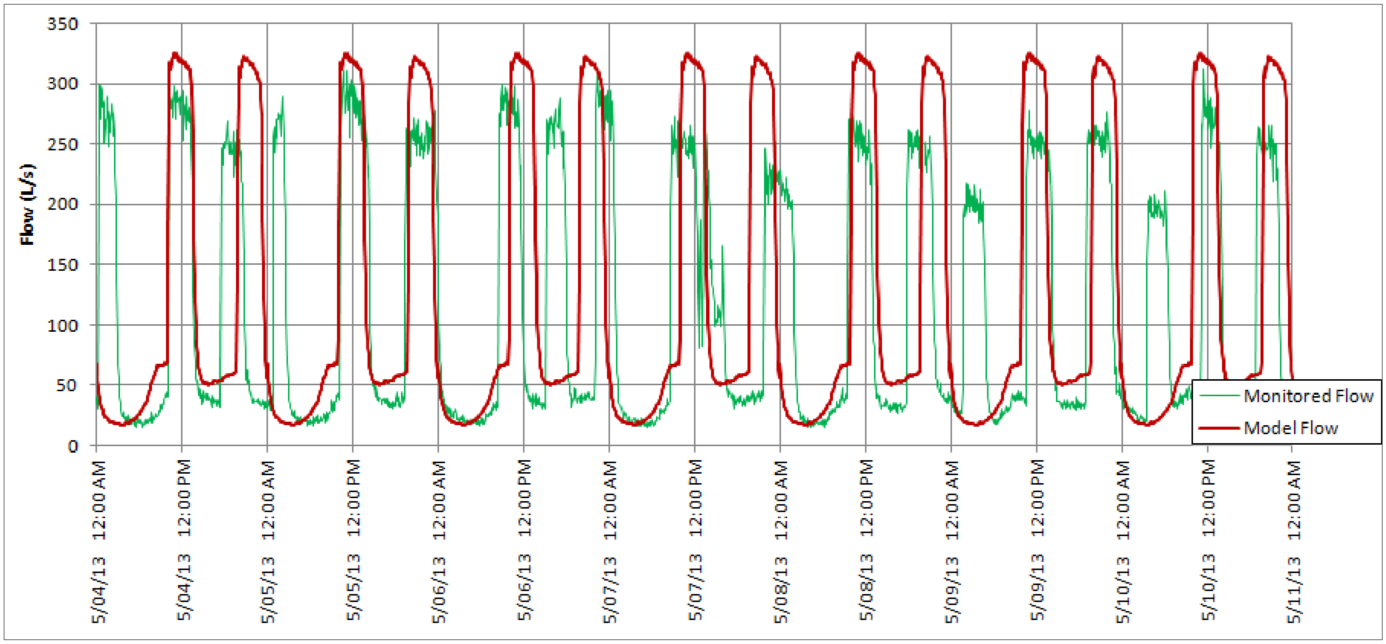


Figure 2.6 Dry Weather Calibration Results – Site 10



***Note:** Measured peak flows are affected by 225 Street PS which has four different sized pumps cycling, where model flow cannot vary the pump capacity for each cycle.*

2.5 Pump Station Inflow and Outflow

2.5.1 225 Street Pump Station

The 225 Street Pump Station is the City's largest pumping station, and was the most difficult to model. The station conveys flow from the River Road and Cotton Wood trunk sewers , along with the majority of the Town Centre area. The 225 Street PS discharges into a 500mm diameter forcemain which is approximately 1.2km long. This forcemain then ties into a 1050mm diameter gravity sewer on River Road. The station consists of 4 different sized pumps ranging from 100 hp to 250 hp. Pump curves were obtained from Flygt and input into the model. As-Built drawings were provided by the City, and used to generate the wet-well volume.

A pump test was conducted on December 9, 2013 to observe actual flow rates from the stations duty pumps P1, P3, and P4; the results are summarized in **Table 2.3**. The maximum flow of 509.5 L/s was observed with all three pumps in operation. The observed firm capacity of the station, measured with largest pump (P4) not in operation, was 505 L/s.

Table 2.3 – Observed 225 Street Pump Station Flow

Pump	Motor Rating (hp)	Impeller Diameter (mm)	Test Duration (mins)	Observed Average Flow (L/s)
P1	214	380	11	298.5
P2	100	450	--	--
P3	250	425	13	301.2
P4	250	425	12	442.8
P1 & P3	--	--	11	505.0
P1 & P4	--	--	11	510.9
P3 & P4	--	--	12	499.4
P1, P3 & P4	--	--	13	509.5

The 225 Street pumps are operated in rotating cycles in order to distribute the runtime more evenly over the duty pumps. The InfoSWMM model simulates pump on-off cycles based on simple fixed water levels; activating each pump when the wet-well water level reaches the 'start-up depth', and shutting down when it lowers to the 'shutoff depth'. Due to the variations in pump usage, actual and simulated pump cycles were different. To validate the results of the model calibration a volume comparison of the downstream Site 10 was used. The total difference during the dry weather calibration period was within 6%, and 1%for the wet weather validation period. The wet weather calibration period had a volume difference of 8%, which was affected by a sewage overflow at the station.

2.5.2 Minor Pump Stations

Out of 28 City operated pump stations modelled- three pump stations had no as built drawings; four pump stations had only pump on-off data, but no level data; and one pump station was missing both level and as-built information. For pump stations with missing as-builts, the wet well size was assumed. For pump stations with missing level data, the as-builts were used (if available) for determining the on-off levels.

We validated the model using average inflows and pump discharge rates developed using SCADA records provided by the City. All of the pump stations have average dry weather inflow of less than 5 l/s, with the exception of the 225 Street Pump Station. Most of the stations have pump run times in the range of 1- 5 minutes during dry weather. One minute interval SCADA data was available for all pump stations. One station had varying pump run times of 2 to 4 minutes which affected the calculated outflow. In this case the pump was modeled to the middle range , and the pump station and downstream pipe capacity were reviewed to ensure both the low and high estimate could be serviced . **Table 2.4** summarizes the observed and modelled outflow at each pump station.

Table 2.4 - Pump Station Calibration Summary

Name	Location	Model ID	Calculated SCADA Pump Outflow (L/s)	Model Outflow (L/s)	ADW Inflow from SCADA (L/s)	Model ADW. Inflow (L/s)	Comments
Katzie		PS01	--	--	--	--	PS not modelled
Wharf St	20208 Wharf St	PS02	4	4	0.1	0.1	
S207 St	20686 - 120B Ave	PS03	8 to 18 [1]	14	2.0	2.0	No SCADA Level data
S208 St	20810 Dewdney Trunk Rd	PS04	14	15	2.2	2.2	Missing As-Built Drawing
Steeves St	20937 River Rd	PS05	--	38	--	1.8	No SCADA level data, Missing As-Built Drawing. Assumed 1800mm dia. wet well
Golf Lane	20975 Golf Lane	PS06	3	5.5	2.3	2.2	
Cutler Pl	21198 Cutler Pl	PS07	9	12	0.2	0.3	
Anderson Pl	11583 Anderson Pl	PS09	7	11	0.3	0.2	
Meadow Brook Pl	12463 Meadow Brook Pl	PS10	10	9	1.0	0.9	No SCADA Level data
Cliff Ave	22010 Cliff Ave	PS11	6	6	0.2	0.2	
River Bend	22197 River Rd	PS12	12	11	1.8	1.5	No SCADA Level data, cycle depth of 700mm is assumed
S225	11555 - 225 St	PS13	299	266 [2]	--	--	Observed Firm Capacity = 505 L/s (P1&P3)
Leachate	11589 Cottonwood Dr	PS14	16	15	1.4	1.4	Model Inflow set from ADW Inflow from SCADA.
Cottonwood	11335 - 234A St	PS15	10	8	0.7	0.6	
Tamarack Lane	23527 Tamarack Lane	PS16	26	30	5.0	4.0	
Fairgrounds	23588 - 105 Ave	PS17	11	15	0.8	0.9	
Tamarack Loop	23680 - 108 Loop	PS18	8	5	0.3	0.4	
S239 St	11600 - 238A St	PS20	16	18	1.6	1.5	
S240	11320 - 240 St	PS21	4	6	0.6	0.4	
S113	24195 - 113 Ave	PS22	6	7	0.2	0.2	
S249 St	11804 - 249 St	PS23	8	8	0.8	0.7	
S250	24927 Dewdney Trunk Rd	PS24	30	29	3.8	3.1	
S251St / Maple Creek	11881 - 250 St	PS25	7	8	0.5	0.4	
S243 St / McClure	24300 McClure Drive	PS26a	9	10	0.9	0.8	Very small catchment Area
S245	24483 - 106B Ave	PS27A	9	9	0.3	0.2	
Fern Crescent	23291 - 132 Ave	PS29A	25	25	1.8	2.3	
S136 Ave	22620 - 136 Avenue	PS30	67	71	3.9	5.0	
Trails Edge	24185 - 106 B Ave	PS31A	11	17	1.0	0.7	
S241	12510 - 241 St (Academy Pk.)	PS32	12	11	1.0	0.9	

[1] SCADA data resolution of +/- 1min coupled with short pump cycle affect calculated pump discharge rate
[2] Average model outflow of P1. Pump flow ranges from 250 - 281 L/s based on pump curve. Volume of flow is within 6% of observed

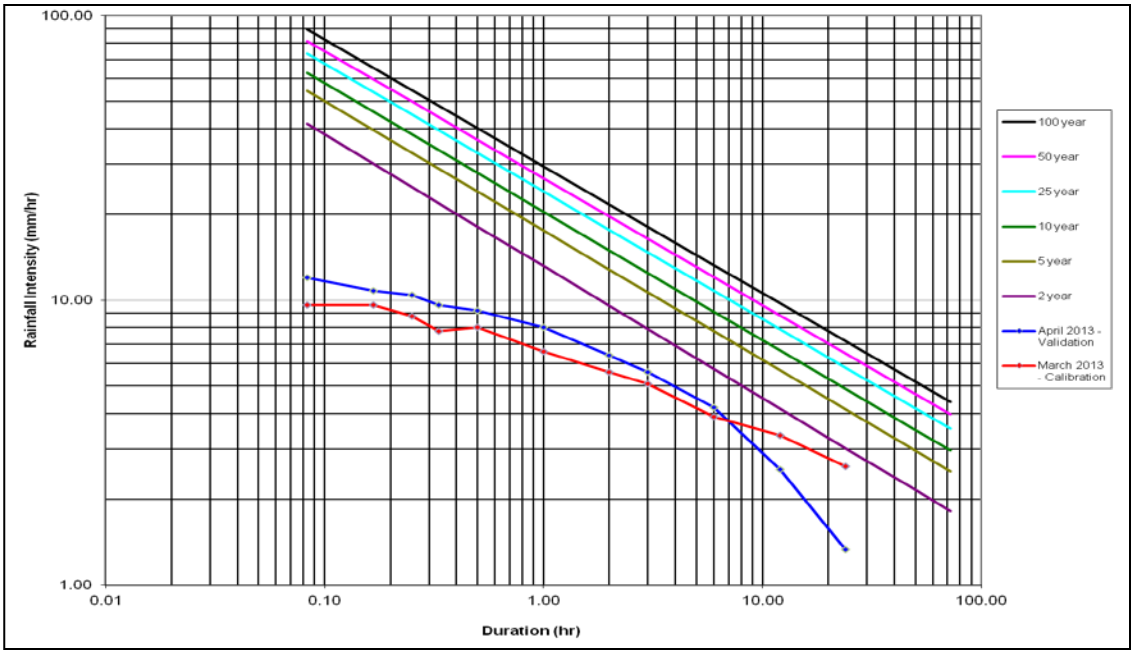
2.6 Wet Weather Calibration Results

The first steps for wet weather calibration were to analyze the rainfall data, compare periods of rain with flow data available, and then select the appropriate calibration sites and events. Rainfall data from February to May 2013 was analyzed for long-duration events proceeded by rainfall to ensure wet antecedent conditions existed. As a result, the two most significant rainfall events during the monitoring period were selected for model calibration/validation:

- March 11-15, 2013
- April 9-11, 2013

Figure 2.7 shows the IDF curves for above two rainfall events overlaid on the IDF curve of DM44 rain gauge (Katzie Pump Station). The storm events were close to the order of a 6-month return period storm in the 1-6 hour duration, and the March storm approached a 1 year event for 12 hour durations. It is preferred to calibrate to events on the order of a 2 to 10-year return period because RDII analysis are often simulated for a 5-year return period. However, these events were the largest storms observed during the monitoring period.

Figure 2.7 Katzie IDF Curve with Calibration Events Overlaid



Wet weather calibration was completed using a full hydrograph approach (i.e. not just peak flows) for a minimum 5-day period including at least 2 days before and after the major storm event. The rainfall-derived infiltration/inflow (RDII) into a sewer system was modelled using the Unit Hydrographs (UHs) approach. A UH set contains up to three such hydrographs, one for a short-term response, one for an intermediate-term response, and one for a long-term response. Each UH group is considered as a separate object by InfoSWMM and is assigned its own unique name along with the name of the rain gauge that supplies the rainfall data. The rainfall data for each inflow node was supplied by the nearest gauge to the node.

Each unit hydrograph is defined by three parameters :

- R: the fraction of rainfall volume that enters the sewer system
- T: the time from the onset of rainfall to the peak of the UH in hours
- K: the ratio of time to recession of the UH to the time to peak

Each inflow node in the model is assigned a UH group based on the flow meter catchment in which it is located. To generate RDII into a node, the model applies the assigned UH to the sewershed area draining to that node. Separate RTK parameters for each flow meter were applied based on observed flows during the calibration events. ***Table 2.5*** summarizes the calibrated RTK values used for each location. **Appendix A** provides hydrograph plots of the wet weather calibration results, comparing model vs observed flows over during the calibration event (March 2013).

Table 2.5 – RTK Parameters Used for Calibration

Catchment	R1	T1	K1	R2	T2	K2	R3	T3	K3	**R Sum**
Site 1	0.4%	2	1	0.4%	5	2.5	0.5%	10	4	**1.3%**
Site 2	0.4%	2	1	0.4%	5	2.5	0.4%	10	4	**1.2%**
Site 3	6.0%	2	1	1.0%	5	2.5	1.0%	10	4	**8.0%**
Site 4	1.3%	2	1	1.0%	5	2.5	0.5%	10	4	**3.1%**
Site 5	1.5%	2	1	3.0%	5	2.5	1.0%	10	4	**5.5%**
Site 6	0.4%	2	1	0.5%	5	2.5	2.0%	10	4	**2.9%**
Site 7	1.6%	2	1	1.0%	5	2.5	0.5%	10	4	**3.1%**
Site 8	0.3%	2	1	0.4%	5	2.5	0.9%	10	4	**1.6%**
Site 9	0.6%	2	1	0.4%	5	2.5	0.5%	10	4	**1.5%**
Site 10	0.8%	2	1	0.6%	5	2.5	0.5%	10	4	**1.9%**

2.7 Model Validation

In addition to calibrating the model at above mentioned sites, the model was also validated using the April 2013 storm. **Appendix B** shows the detailed plots for all validation results at temporary monitoring sites. The model was also validated using average inflows and pump discharge rates developed using SCADA records provided by the City.

Please refer to *Technical Memorandum #01* for a more detailed analysis of the wet weather calibration and validation.

3 Future Scenarios

3.1 Land-Use / Planning Scenario Overview

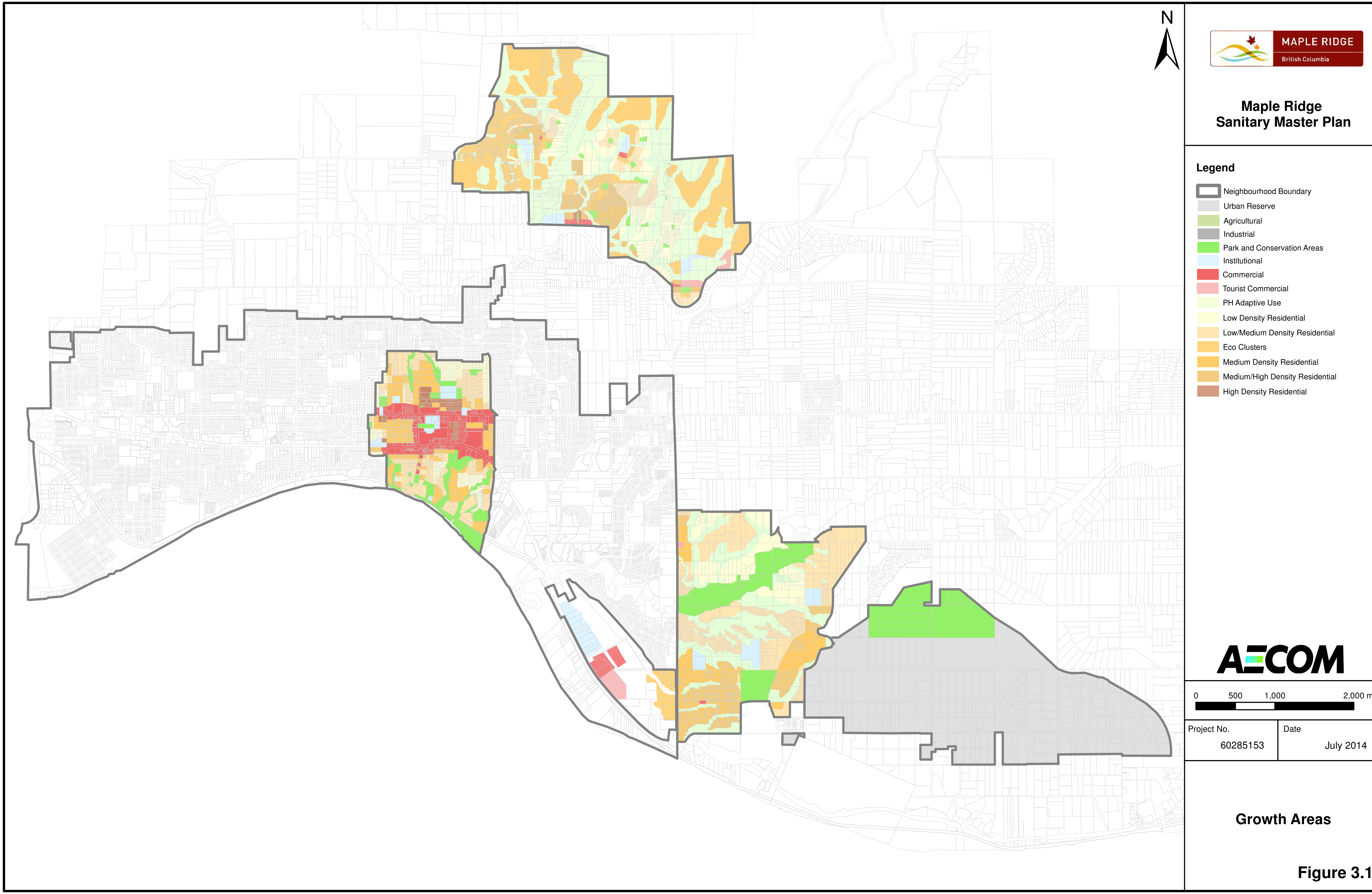
Land-use, zoning, and OCP information was collected from the City in GIS/hardcopy format. The following is a summary of key information obtained from the City:

- GVRD's Traffic Zone population projections in shapefile format. The shapefile updated by City's planning staff was used as a basis for the OCP population projections for most areas except Albion and Albion Flats. Please refer to *Technical Memorandum #03* for more details regarding future population projections for the Albion and Albion Flats area;
- Existing zoning and existing land-use shape files;
- Land use shape file of the various growth areas. ***Figure 3.1*** shows the location for each area within the City; and
- OCP shape file.

Various land-use and/or zoning scenarios were developed in the InfoSWMM model. Each scenarios include different flow generation data sets. The scenarios include:

- **"Existing Actual"** – Represents current land-use and flow conditions based on calibration results. Only includes lots currently serviced by the City. Populations based on the latest census data, sewage flows based on population, water meter records and observed GWI rates. Infrastructure is existing infrastructure as per the City's GIS;
- **"Intermediate 2018 and 2023"** – Population for the interim scenarios (2018 & 2023) were interpolated from current and OCP scenarios, with emphasis from the Planning department on what is expected in the next 5 to 10 years in terms of growth. Two meetings were held with the City's planning staff to better understand the staging of expected growth in the areas outlined in **Figure 3.1**.
- '**"OCP Scenario"** – Represents the ultimate future development scenario (2041).

There is significant growth planned for the Albion area, Town Center, Silver Valley, Thornhill, and Albion Flats. Limited growth is also expected along major transportation corridors. AECOM met with the City's Planning department to develop future population estimates that accurately reflect the OCP and planned developments.


N
MAPLE RIDGE
British Columbia
Maple Ridge
Sanitary Master Plan
Legend
Neighbourhood Boundary
Urban Reserve
Agricultural
Industrial
Park and Conservation Areas
Institutional
Commercial
Tourist Commercial
PH Adaptive Use
Low Density Residential
Low/Medium Density Residential
Eco Clusters
Medium Density Residential
Medium/High Density Residential
High Density Residential
AECOM
0 500 1,000 2,000 m
Project No.
60285153
Date
July 2014
Growth Areas
Figure 3.1

The OCP scenario also reflects the amendment approved by Council in July 2013 which permits an increase in development density in the Albion area. Finally, land use assumptions for the Albion Flats neighbourhood were based on November 2013 plans made available by the City. The OCP Scenario does not include development in the urban reserve, however that area is reviewed in Section 4.7.

The population increases were estimated using community growth areas instead of a “City-wide” blanket approach; this better captures the different rates of growth expected throughout Maple Ridge. The Traffic Zone populations maintained by the City were used as the starting base, but in several instances did not reflect the full amount of growth expected from the Area Plans. Future populations for the Town Centre and Silver Valley were developed using the OCP land use and estimated density values. Intermediate (2018 and 2023) year populations were interpolated between the current and ultimate population estimates. This was accomplished using a linear regression from the expected build-out year to the desired scenario year.

3.2 Future Flow Generation

In order to input dry weather flows in model on a parcel-by-parcel basis, a separate shape file was created for each scenario containing land-use, residential populations, and industrial, commercial and institutional (ICI) flows for each property in the City. AECOM consulted with City staff to develop the unit-rates that were used to generate future flows for assessment. The average dry weather flow (ADWF) was calculated using a unit rate of *300 L/Capita/Day*. This rate was applied to all residential and ICI equivalent populations (PE).

3.2.1 Commercial Flows

The average commercial rate of 80 PE/ha was determined using winter water meter records from a sample of existing neighbourhood and community commercial lots in the calibrated model. This rate was used for all future neighbourhood and community commercial areas. For future big box commercial developments 120 PE/ha was used as per MMCD design criteria. To account for re-development of existing neighbourhood and community commercial lots in the future, the following criteria was used:

- If existing flows were less than or equal to average plus one standard deviation: the average flow rate of 80 PE/ha was used
- If existing flows were greater than the average plus one standard deviation: flow rate was based on the existing winter water meter data.

3.2.2 Industrial and Institutional Flows

Flows from existing industrial and institutional lots were calculated using the winter water meter data. For future industrial areas, 200PE/ha was used as per MMCD design criteria.

3.2.3 GWI and I&I

The ground water infiltration (GWI) and RDII rates for existing areas are based on the calibrated model. For details please refer to AECOM's *Technical Memorandum #01 –Model Development and Calibration*. For future developments (greenfields), Inflow & Infiltration (I&I) was estimated using the Metro Vancouver unit rate of 11,200 L/ha/day (equivalent to 5-year I&I level).

For future developments (greenfields) in Albion Flats and North Albion, average dry weather flows (ADWF) was input into the model along with the diurnal pattern to model the peak dry weather flow. The diurnal pattern established in the model was used to create the peak flows. After discussing with City staff we decided not to use the Harmon Peaking Factor since the diurnal pattern is based on a calibrated model, and is more accurate for estimating peak flows in trunk sewers and inflows to the pump stations.

4 Sewer System Assessment

4.1 Hydraulic Capacity Criteria

Sections 2 and 3 of this report described the approach to developing the sewer model, and the subsequent scenarios that were developed to assist the City with planning for capital works required to service increased development. The model scenarios were simulated to identify hydraulic constraints such as undersized sewers and pump stations. The OCP boundary condition does not reflect any backwater conditions at the Katzie Pump Station, as discussed in Section 2. The capacity assessment was completed for the four scenarios including current, 2018, 2023 and OCP. All scenarios were simulated under a 5-year 24-hour design storm condition.

The following criteria were used to assess the sanitary sewer system:

- Local sewers (PWWF<40 L/s) running more than 70% full (Qpeak/Qfull > 0.7) were recommended for upgrade.
- Trunk sewers (PWWF>40 L/s) running more than 83.5% full (Qpeak/Qfull > 0.835) were recommended for upgrade. This is equivalent to approximately 70% of the full pipe depth.
- Pump stations should have capacity to convey the PWWF using only the duty pump. Stand-by pumps were deactivated for the assessment.

4.2 Gravity Sewer Assessment

The results of each scenario are summarized in **Figures 4.1 to 4.4.** Sewers with a Qpeak/Qfull greater than the threshold criteria have been flagged, and are shown in bold red. Manholes are colour-coded based on the maximum surcharge level experienced in the simulation. Surcharging is defined when the HGL is above the pipe crown, but below the ground surface, where flooding is when the HGL exceeds the ground elevation.

As an important reminder, the calibration of model results was based on a tolerance of +/- 15% and some of the highlighted sewers may fluctuate on the design criteria threshold. The sewers highlighted as exceeding the design criteria and/or surcharging are not necessarily considered undersized sewers and in need of immediate upsizing because they are based on theoretical population horizons. These highlighted sewers should be examined in more detail on a case by case basis in order to confirm if and when capital upgrades and upsizing are warranted. The detailed examination should consider diversion options to alleviate peak flows, alternate routing / twinning options, risk of property damage upstream and potentially flow/level monitoring to confirm model results in these localized areas.

Tables 4.1 and 4.2 provide a summary of the total length of sewers in each category of capacity (PWWF vs full) for the various population scenarios The tables are separated respectively for local and trunk sewers.

Table 4.1 – Local / Collector Sewer Capacity Summary (Q < 40 L/s)

Qpeak / Qfull Ratio	Length of Undersized sanitary sewers (km)			
	Current	2018	2023	OCP
0 – 0.5	248.7	241.8	240.0	236.5
0.5 – 0.70	4.7	3.5	3.6	4.3
0.70 – 1.00	1.7	2.4	3.1	3.0
>=1.0	0.6	0.4	0.5	0.9

Table 4.2 – Trunk Sewer Capacity Summary (Q > 40 L/s)

Qpeak / Qfull Ratio	Length of Undersized sanitary sewers (km)			
	Current	2018	2023	OCP
0 – 0.5	20.3	18.7	17.6	16.7
0.5 – 0.835	2.2	8.7	11.2	14.1
0.835 – 1.00	0.1	0.6	0.9	1.1
>=1.0	0.3	2.2	2.5	2.8

Full size hardcopies of **Figures 4.1** through **4.4** showing model results of the various scenarios are provided in ***Appendix C***. Plots of Maximum HGL in the major trunk sewers are provided in **Appendix D**.

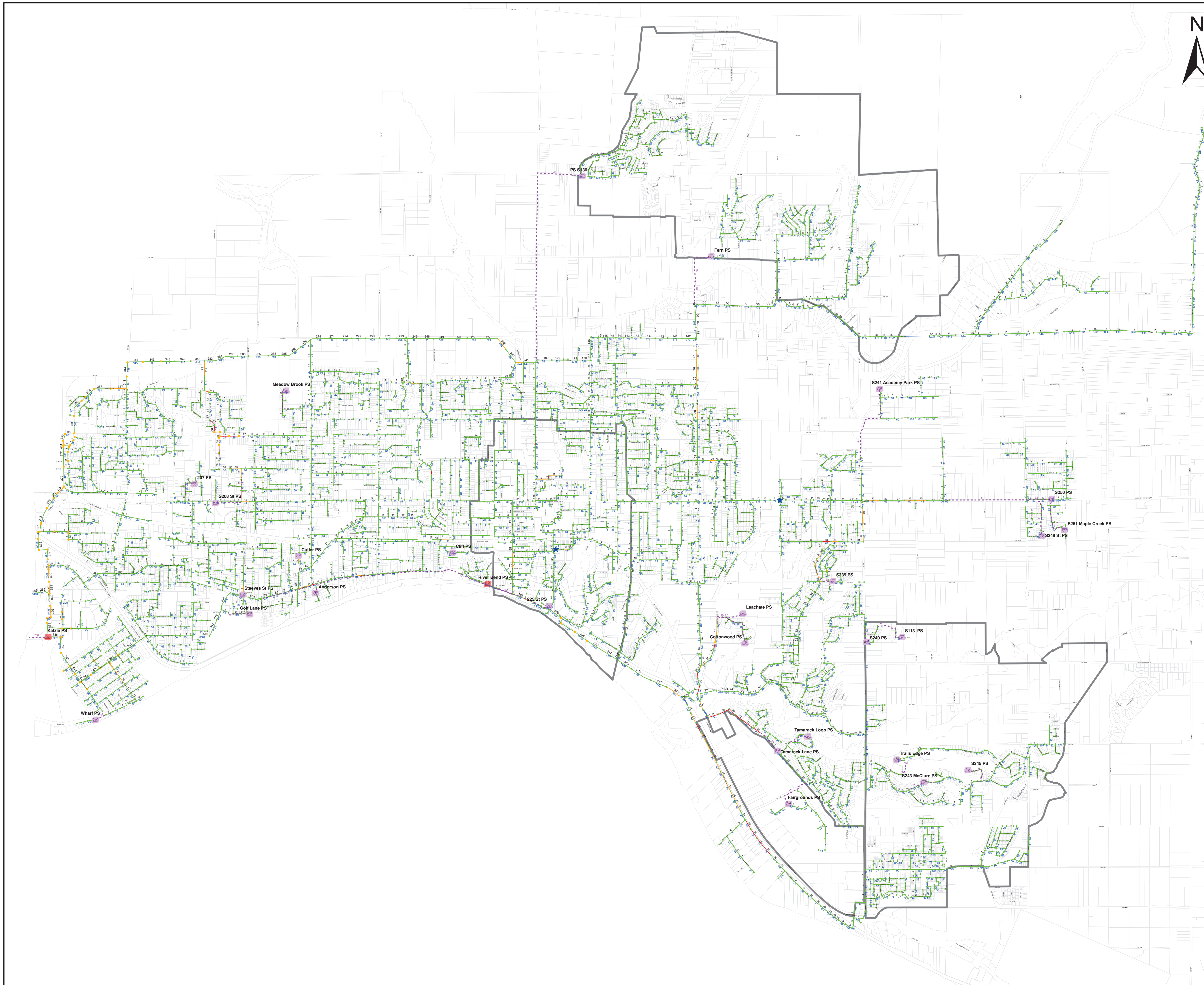
Under the current scenario several manholes are flagged as surcharging during the design storm, but in all instances the HGL remains within the MH depth. The surcharging MH's are mainly limited to the interceptor sewers adjacent to the Katzie PS, and the River Road trunk sewer immediately upstream of the 225 Street PS. Along with the model results, corroborating evidence from City staff indicates that the River Road trunk does not cause any flooding under existing surcharged conditions, which are present when the 225 Street Pump Station is shut down. The area of Skillen Street and 123 Avenue has several pipes flagged as under capacity, with the connecting MHs experiencing some surcharging.

In the interim and OCP scenarios, the list of surcharging manholes increases; mostly in the areas upstream of the 225 Street PS. Several MH's on the River Road trunk sewer are surcharged, and approximately 8 are shown to flood. The flooding is related to the 225 Street PS, which is shown to be under capacity beginning in the 2018 scenario (see Section 4.4). The inadequate capacity at the pump station leads to HGL rising in the River Road trunk sewer, which is also under capacity for the scenario.

When the River Road trunk sewer is surcharged, several upstream MHs flood under peak flow conditions in the area of Makay Avenue and Fisherman Road. While the River Road trunk sewer is estimated to be over capacity (Qpeak/Qfull > 0.835) for the 2018 scenario, actual peak flows will be dependent on the rate of upstream development. The 2018 scenario has assumed rapid development in Albion Flats, including the diversion of flows from the Fairgrounds PS to River Road, and further development in south Albion. As such, recommended improvements on the trunk sewer could likely be delayed beyond 2018, and the area is recommended for further study as development progresses.

Two MHs located at the rear of residential lots on Tamarack Lane, R044 and R045, were also flagged as flooding in the interim and OCP scenarios. The simulations showed the MH's flooded when the upstream Tamarack Lane PS was active. Downstream sewer capacity at River Road was reduced such that the pumped flow caused a sharp but brief rise in HGL. It is recommended that these manholes be bolted to reduce the potential of flooding due to short periods of high HGL. Property damage near these MHs would not be likely due to the topography; the adjacent homes are several meters above the manhole rim elevations.

Two MHs located on 232 Street (L078 & L076) were identified as surcharging for the Interim and OCP scenarios with several pipes identified as under capacity. The residential neighbourhood located immediately west of the surcharged manholes is built above the road elevation and will not be affected by the surcharging manholes. The modeled HGL through this sewer remains well below ground level for all of the scenarios. It is recommend that the City upgrade these sewers prior to permitting any new service connections at this location.



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 — Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 — Q<40 L/s; q/Q>0.70

Peak Flow (L/s)
Full Flow Capacity (L/s)

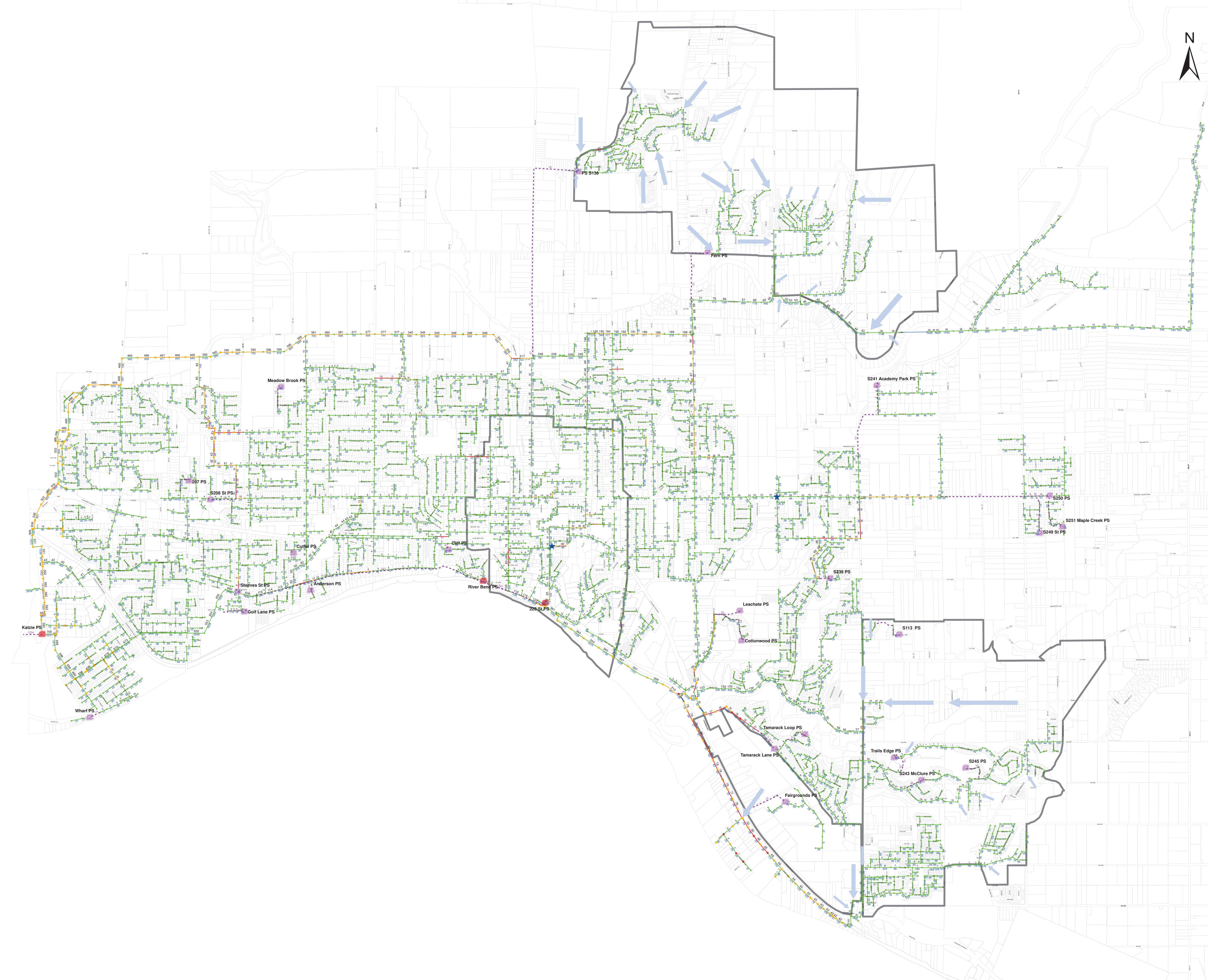
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

Current Scenario 5 yr-24hr Design Storm Results

Figure 4.1



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 — Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 — Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)

Full Flow Capacity (L/s)

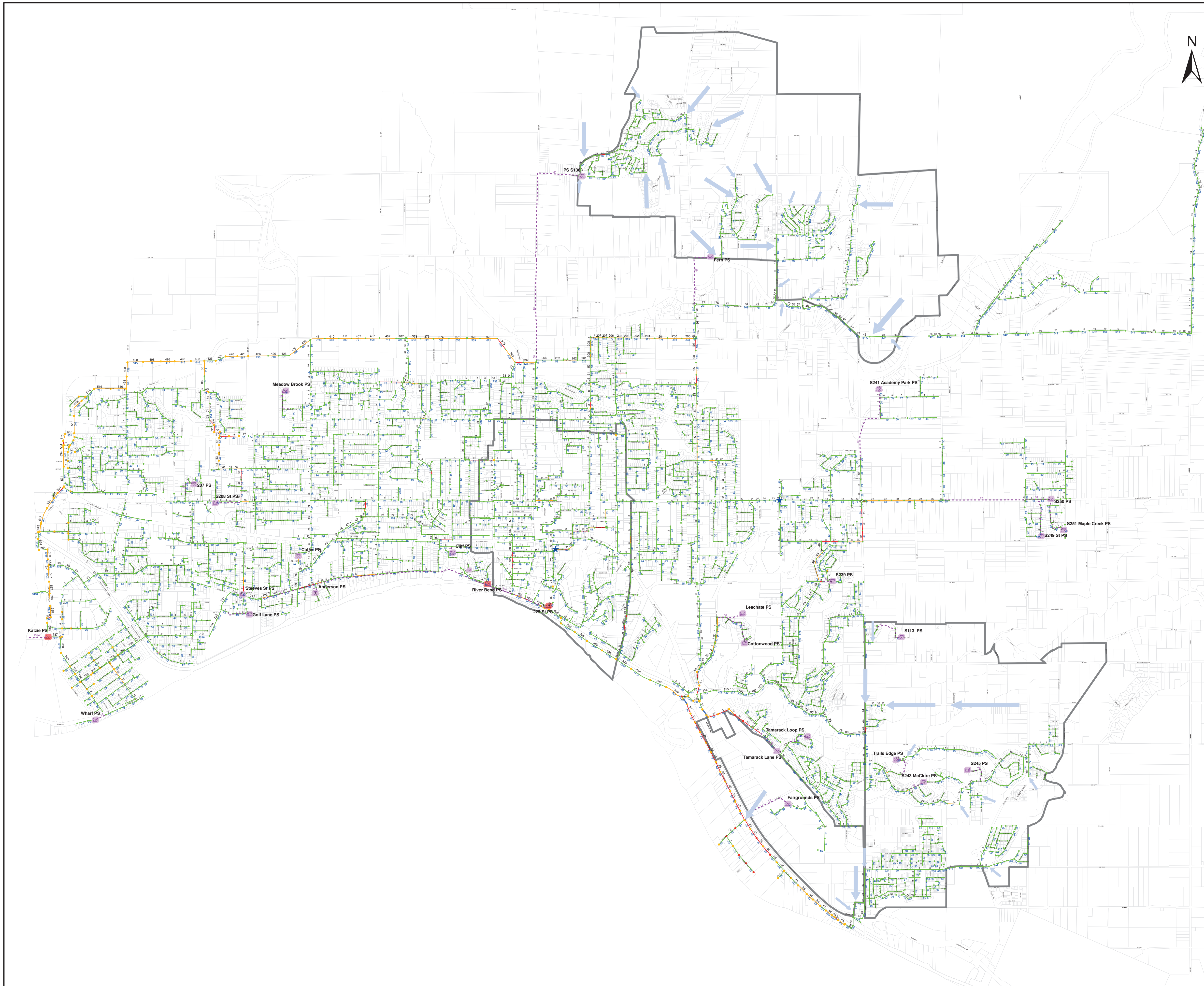
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

2018 Scenario 5 yr-24hr Design Storm Results

Figure 4.2



MAPLE RIDGE British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 | Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 | Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)
Full Flow Capacity (L/s)

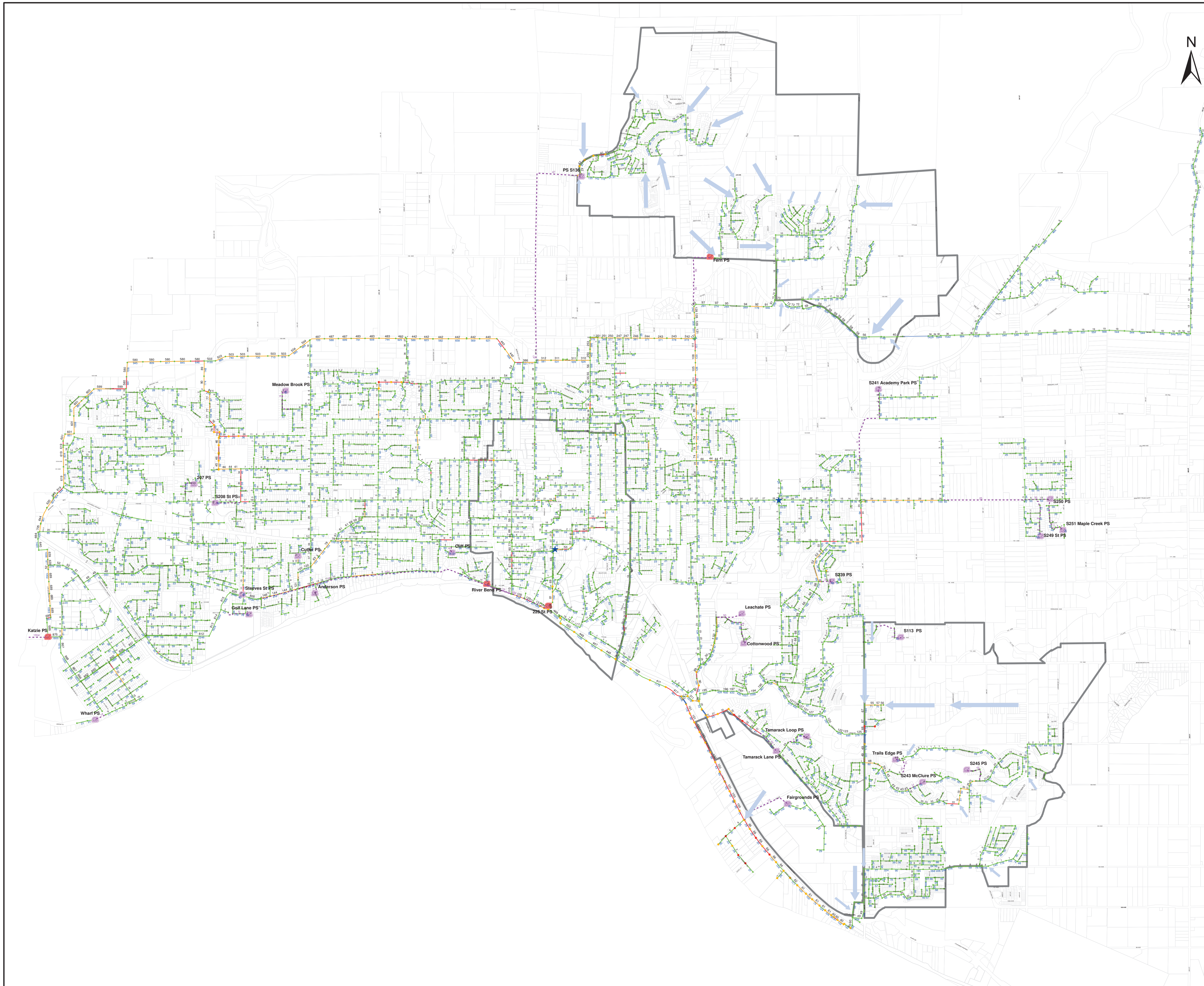
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

2023 Scenario
5 yr-24hr Design Storm Results

Figure 4.3



Maple Ridge Sanitary Master Plan

MAPLE RIDGE
British Columbia

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835   Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835   Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)
Full Flow Capacity (L/s)

AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

OCP 2041 Scenario 5 yr-24hr Design Storm Results

Figure 4.4

4.3 Inverted Siphon Sewer Assessment

The City operates several inverted siphons to convey sanitary flows across creeks and rivers. Due to the complexity involved in their modelling, siphon capacities were also assessed using the manning's equation. The theoretical capacity was compared to the peak OCP flows to identify any gaps in conveyance capacity. The results of the assessment are summarized in **Table 4.3**. Of the five siphon crossing areas assessed, only one location has an OCP flow exceeding the capacity; 240 Street at Kanaka Creek. With a total a hydraulic capacity of 37 L/s, the twin siphon is adequate to convey the current and 2018 peak flows, but will be exceeded by a 2023 flow of 38 L/s, and an OCP flow of 49 L/s. The City has indicated the 240 Street bridge crossing the creek has an opening to install a future sanitary pipe up to 650mm diameter. At an estimated slope of 1%, a 300mm diameter sewer would be the minimum size required to provide sufficient capacity at this crossing.

Table 4.3 – Inverted Siphon Capacity Assessment

Name	From MH	to MH	u/s Inv. (m)	d/s Inv. (m)	L (m)	Pipe Dia. (mm)	Mann-ing's n	Elev. Diff. (m)	S (m/m)	Vel. (m/s)	Full Flow (L/s)	OCP PWWF (L/s)
River Road	TS15	TS14	-0.4	-0.54	60.0	750	0.013	0.14	0.002	1.2	538	--
River Road	TS15	TS14	-0.4	-0.54	60.0	450	0.013	0.14	0.002	0.9	138	--
River Road	TS15	TS14	-0.4	-0.54	60.0	250	0.013	0.14	0.002	0.6	29	--
Total Capacity - River Road @ Kanaka A											**704**	101.2
Tamarack Lane	R054	R053	1.09	0.33	61.4	200	0.013	0.76	0.012	1.2	36	--
Tamarack Lane	R054	R053	1.09	0.22	61.4	150	0.013	0.87	0.014	1.0	18	--
Tamarack Lane	R054	R053	1.09	0.22	65.2	150	0.013	0.87	0.013	1.0	18	--
Total Capacity - Tamarack Lane @ Kanaka Creek											**72**	27.4
Lane west of 243A St	R340	R163	21.92	21.55	47.2	150	0.013	0.37	0.008	0.8	13	--
Lane west of 243A St	R340	R163	21.92	21.55	47.2	150	0.013	0.37	0.008	0.8	13	--
Total Capacity - Lane west of 243A Street											**27**	0.4
240 Street	W004	W001	6.25	5.3	98.0	200	0.013	0.95	0.010	1.0	32	--
240 Street	W004	W001	6.25	5.3	98.0	100	0.013	0.95	0.010	0.6	5	--
Total Capacity - 240 Street @ Kanaka Creek											**37**	**49.1**
128 Avenue	Y018	Y017	38.88	36.06	253.0	200	0.013	2.82	0.011	1.1	35	--
128 Avenue	Y018	Y017	38.88	36.03	253.0	375	0.013	2.85	0.011	1.7	186	--
Total Capacity 128 Ave @ Alouette River											**221**	36.4

4.4 Pump Station and Forcemain Assessment

In conjunction with the gravity sewer evaluation, the capacity of the existing sanitary pump stations was reviewed. ***Table 4.4*** summarizes the existing station capacity versus the predicted PWWF flows for each scenario. Note the station capacity is calculated based on the SCADA data and assumes one pump is held out of service as a back-up only. PWWF is measured as the maximum inflow at the station. For pump stations with multiple inlet sewers (e.g. 225 Street) the peak flow in each pipe may occur at different times, therefore peak inflow is not necessarily the sum of the inlet PWWFs.

As noted in ***Table 4.4***, 3 of 28 pump stations are undersized for the "OCP Scenario" peak flows. These stations are highlighted in orange in table below. In conjunction with pump station upgrades, some of the discharge forcemains may require upsizing. Forcemain upgrades will most likely be required for the 225 Street Pump Station. It is important to re-iterate that the peak inflows predicted at the pump stations may vary depending on the model calibration, and therefore any pump stations within a 10% +/- tolerance of the predicted peak inflows should be monitored regularly for longer run times and increased pump cycles as these are signs of potential under-sizing. Stations that fall into this category include those highlighted in "green" in ***Table 4.4***.

The 225 Street Pump Station is the most critical in the list. The station has a firm capacity of 505 L/s, which is estimated to be reached after 2018. Options to attenuate pumping rates by using more storage capacity in the interceptor sewers feeding the station are minimal as these sewers are already surcharged during the design storm. The capacity assessment is based on a conservative estimate of total peak flows conveyed by the station, however the timing of when peak flows will exceed the current capacity will depend on actual L per capita flows, and I&I. Our assessment assumes 300 L/c/d along with an allowance of 50 L/s inflow from the Leachate pump station. If the leachate was not included in the assessment, the capacity would not be exceeded until the next planning horizon, 2023. Similarly, if the station is assessed using a residential rate of 250 L/c/d or lower, the capacity would be sufficient in 2018, but exceeded in 2023 or later.

When the 225 Street Pump Station and forcemain are upgraded, they should be sized to accommodate flow from Thornhill. As described in the boundary conditions, Metro Vancouver is not expected to be able to accept PWWF from significant storms until after 2021 due to limitations in their system capacity; therefore there will be no benefit to upgrading pumping capacity before then.

The River Bend Pump Station is estimated to be out of capacity under the current scenario. The Pump Station catchment is located adjacent to an area identified with high rates of I&I. In the model simulation wet weather inflow accounts for the majority of pumped flow during the design storm, therefore station capacity may be improved through better I&I management. It is important to note the SCADA data used to calibrate this station did not include water level information, therefore the capacity was based on an estimated pump cycle depth. This station should be further investigated to confirm capacity estimates and determine if high inflows are occurring during wet weather prior to planning any upgrade.

The Fern Pump Station may require an upgrade to service future flows from development in Silver Valley. The station is estimated to be out of capacity for the OCP planning horizon. Inflows should be reviewed prior to any upgrades since the nature of development in Silver Valley will affect the loading at this pump station.

Table 4.4 – Pump Station Capacity Assessment

NAME	LOCATION	Model ID	Approximate Firm Capacity (L/s)	PWWF (L/s)				Comments
				Current Scenario	2018	2023	OCP	
Wharf St	20208 Wharf St	PS02	4	0.4	0.5	0.6	0.6	--
S207 St	20686 - 120B Ave	PS03	14	4.4	4.5	4.9	6.2	--
S208 St	20810 Dewdney Trunk Rd	PS04	15	8.7	9.4	9.5	9.9	--
Steeves St	20937 River Rd	PS05	38	6.4	8.9	9.5	11.0	No SCADA level data and as-builts available for this pump station. Firm capacity based on pump curve
Golf Lane	20975 Golf Lane	PS06	6	5.5	5.6	5.7	5.7	High GWI rate was observed at this pump station
Cutler Pl	21198 Cutler Pl	PS07	9	1.1	1.6	1.8	2.7	--
Anderson Pl	11583 Anderson Pl	PS09	7	0.7	0.8	0.8	0.8	--
Meadow Brook Pl	12463 Meadow Brook Pl	PS10	10	1.8	1.8	1.8	1.2	--
Cliff Ave	22010 Cliff Ave	PS11	6	1.4	1.9	1.9	2.1	--
River Bend	22197 River Rd	PS12	12	14.1	14.5	14.6	15.3	No SCADA level data available for this pump station. Cycle depth of 700mm assumed. High RDII rates observed in this catchment
S225	11555 - 225 St	PS13	505	400.7	521.5	593.8	655.7	Future scenarios include 50 L/s Inflow from the Leachate PS
Leachate	11589 Cottonwood Dr	PS14	50	1.1	50.0	50.0	50.0	Peak Wet Weather Flow assumed to be maximum design flow with both pumps operating.
Cottonwood	11335 - 234A St	PS15	10	2.3	2.3	2.3	2.3	--
Tamarack Lane	23527 Tamarack Lane	PS16	26	24.6	17.4	17.4	17.4	Flows from Fairground PS assumed to be diverted to River Road Trunk in future
Fairgrounds	23588 - 105 Ave	PS17	11	9.2	10.6	10.6	10.6	Flows from future development in Albion Flats loaded directly to River Road Trunk in Model
Tamarack Loop	23680 - 108 Loop	PS18	5	1.3	1.5	1.5	1.5	--
S239 St	11600 - 238A St	PS20	16	3.2	3.2	3.2	3.8	--
S240	11320 - 240 St	PS21	4	6.5	7.4	7.7	8.2	Inflow to this pump station to be diverted to south by gravity in future.
S113	24195 - 113 Ave	PS22	6	0.6	0.7	0.7	0.7	--
S249 St	11804 - 249 St	PS23	8	2.1	2.5	2.5	2.5	--
S250	24927 Dewdney Trunk Rd	PS24	29	23.6	27.3	27.4	27.2	Peak Inflows to this pump station are influenced by flow from Maple Creek PS. Inflows to this PS should be monitored in future
S251St / Maple Cre	11881 - 250 St	PS25	7	1.1	1.3	1.3	1.3	--
S243 St / McClure	24300 McClure Drive	PS26a	9	1.9	2.0	2.0	2.0	--
S245	24483 - 106B Ave	PS27A	9	0.5	0.5	0.5	0.5	--
Fern Crescent	23291 - 132 Ave	PS29A	25	6.7	12.9	15.9	27.4	Part of future development in Silver Valley is assumed to be serviced by this PS
S136 Ave	22620 - 136 Avenue	PS30	75	13.6	26.3	31.0	47.4	Part of future development in Silver Valley is assumed to be serviced by this PS
Trails Edge	24185 - 106 B Ave	PS31A	11	2.5	4.6	5.1	5.8	--
S241	12510 - 241 St (Academy Pk.)	PS32	12	5.5	7.8	8.1	8.8	--

4.5 Recommended Upgrades

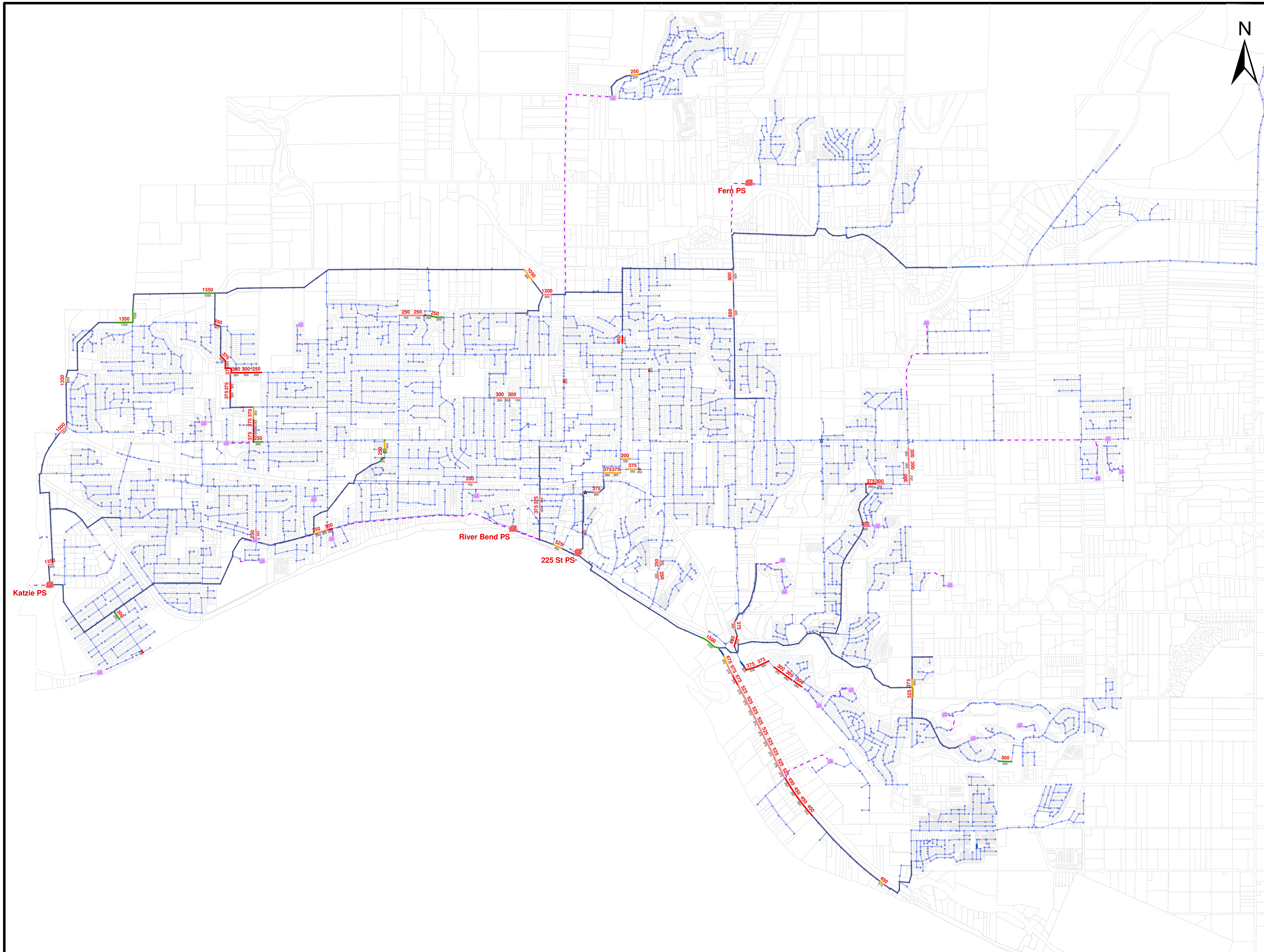
An analysis of the existing sewer capacities vs. future peak flows was completed. In general, the OCP scenario results in the highest peak flows of all the scenarios. For the purposes of sewer assessment, the OCP 5-yr 24hr peak flows were used to assess sewer capacity and determine the recommended pipe diameters.

Figure 4.5 shows the location of proposed pump station and pipe upgrades as well as the existing and ultimate required pipe diameters. **Figure 4.5** only shows the pipes that do not meet the design criteria threshold. At a few locations the pipes downstream of proposed upgrade are smaller in size than the proposed diameter but have sufficient capacity to convey the design flows because of steep slopes. The City can consider upgrading these sections during the detailed design stage to avoid downsizing diameters as flow travels downstream, even if the pipe has sufficient capacity. Also there are some sewers that currently have capacity for OCP flows because of steep pipe slopes; upgrades are still recommended on some of these sewers, where the upstream and downstream pipes are undersized, in order to avoid having smaller diameters as flows travel downstream.

Appendix E provides a tabular summary of the recommended pipe upgrades, including pipe ID, existing diameter/capacity, Peak Wet Weather Flow for each scenario and the proposed diameter. A total of 110 upgrades have been recommended for a total length of 8.2 km of sewers. Pipe upgrades have been grouped into 36 projects listed in ascending order of priority. Trunk sewers that lack capacity under current PWWF were given the highest priority, followed by trunk sewers needed to support ongoing developments, particularly in Albion, but also including Silver Valley and Town Centre. Segments of trunk sewer on the North Slope Interceptor (adjacent to the Katzie PS) flagged as under capacity should undergo a backwater analysis before upgrading.

Local sewers with insufficient capacity under current conditions were given the next priority, followed by trunk and then local sewers for the later future scenarios up to the OCP. Pipes that require future upgrades but are adjacent to those requiring current upgrades were grouped into the same project in order to achieve possible cost savings in mobilization and to avoid disturbing the same street several times.

High level cost estimates have been included for each project in **Table E.1**, along with an estimated percent of cost eligible for DCC funding. All Infrastructure requiring future upgrades due to increased population are assumed eligible for DCC funding. Pump Station and related forcemain cost estimates are summarized in **Table E.2** while the unit cost rates used in the estimates are included in **Table E.3**.



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Undersized Pump Station
- Pump Station

Upgrade Required for Scenario:

- Current
- 2018
- 2023
- 2041 (OCP)

Sewers

- Local Sewer
- Trunk Sewer
- Forcemain
- Siphon
- Diversion MH
- Sanitary MH

Proposed Diameter for OCP (mm)
Existing Diameter (mm)

AECOM

0 400 800 1,600 m

Project No.	Date
60285153	October 2014

Recommended Upgrades

Figure 4.5

4.6 I&I Assessment

As part of the assessment of the sanitary system, a characterization of Inflow & Infiltration (I&I) in catchments A & K (which were previously noted in in the 2002 Master Plan to have the highest I&I rates) as well as a general review of system wide I&I was completed.

Inflow and Infiltration (I&I) includes the Groundwater infiltration (GWI) representing groundwater that enters the sewer system during dry weather conditions, and Rainfall Dependent Inflow & infiltration (RDII) which represents both inflow + rainfall induced groundwater infiltration that enters the sewer system. AECOM developed RDII values for the 5-year (24-hour) events in using MV's envelope method in accordance with the I&I Management Plan Template. The purpose of the envelope method is to use a collection of recorded storm events to create a correlation between the amount of rain that falls in a catchment and the amount of RDII it generates. Knowing the return period of the rainfall allows the correlation to be used to produce estimates for return period based RDII.

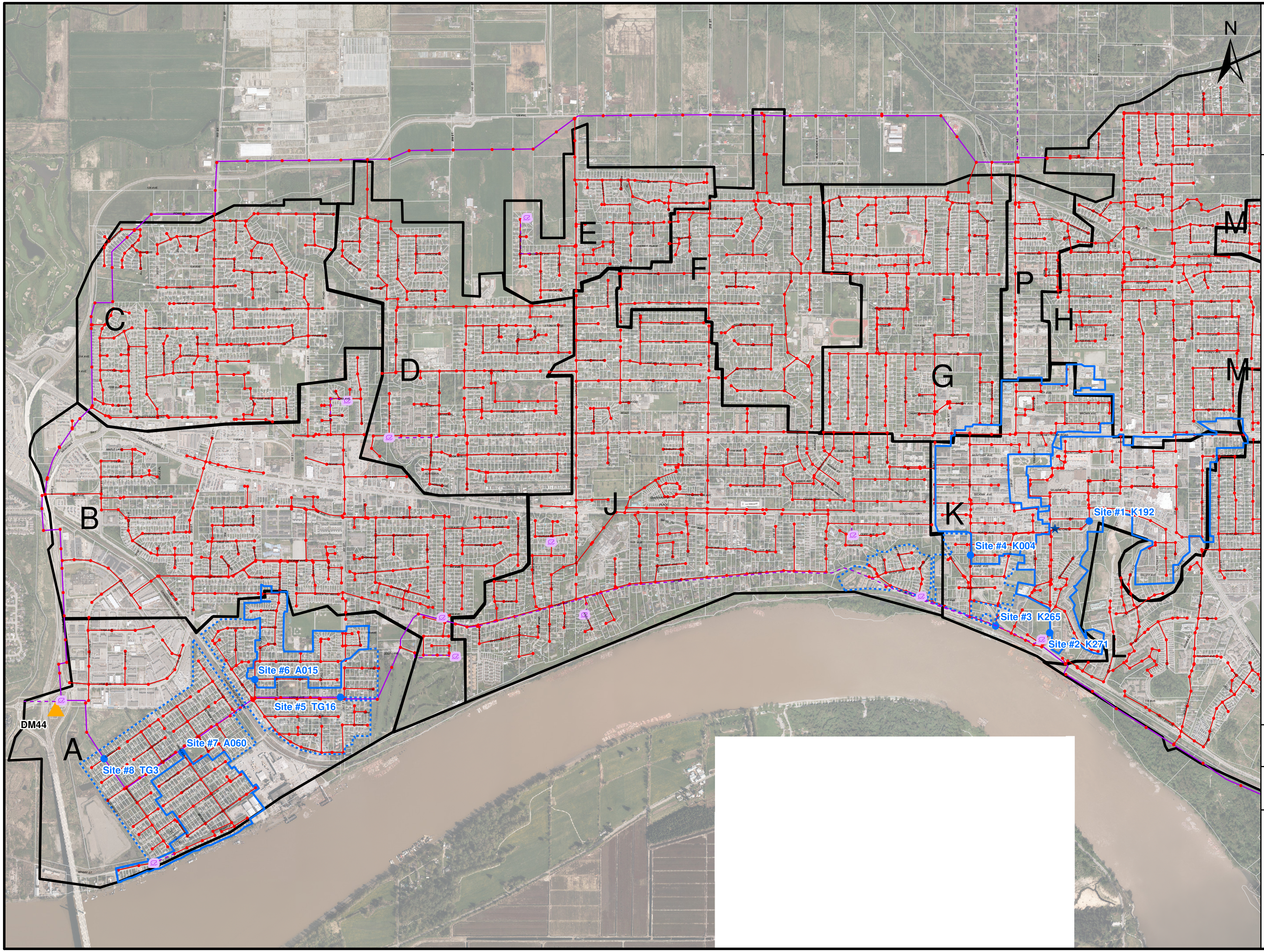
In February 2013 AECOM developed a flow monitoring plan for the purpose of I&I determination of the selected catchments. Nine (9) sites were selected for a monitoring period between December 2013 and May 21, 2014. The monitoring sites are depicted in **Figure 4.6**, and the results of the analysis are summarized in **Table 4.5** below;

Sites 1-4 and Site 9 are located near rain gauge DM62 at Golden Ears Elementary. Sites 5-8 are located close to gauge DM44 at the Katzie Pump Station. During the monitoring period a total of 10 rainfall events were identified and isolated for analysis; however only one rainfall event exceeded 55mm in 24 hours, which is comparable to a 6-month storm at the Maple Ridge rain gauges. Typically storms that are less than the 6-month return period are not used to extrapolate 5-year I&I rates. We used the available data to generate the RDII envelope, but caution that this extrapolation to a 5-year return period may be further off than if this exercise were to be repeated with a longer duration of monitoring data. Due to the varying dates of commissioning for the monitoring sites, and maintenance issues encountered at each, the number of storms used to extrapolate the RDII ranges from 4 to 9.

Table 4.5 – 5yr:24hr I&I Flow Rate Estimates

Location	Area (ha)	ADWF (L/s)	GWI (L/s)	5yr-24hr RDII (L/s)	5yr-24hr I&I (L/ha/day)
Site 2	63	7.4	2.9	13.1	21,883
Site 3-4	20	35.4	9.3	26.5	154,762
Site 4	44	12.4	5.7	31.7	72,686
Site 6	15	1.4	0.4	0.5	5,006
Site 7	20	1.6	0.8	9.2	43,667
Site 8-5	98	35.4	12.8	41.0	47,451
Site 8 (South Slope Interceptor)	1020	160.3	47.0	245.0	24,732
North Slope Interceptor (1)	960	96.2	19.10	80.0	8,919

(1) Interceptor was not monitored during the same period. Data used to develop I&I was taken from the 2013 calibration flow monitoring data.



MAPLE RIDGE
British Columbia

**Maple Ridge
Sanitary Master Plan**

Legend

- Diversion MH
- PumpSations
- Forcemains
- Monitoring Site and MH ID
- I&I Monitor Site Area
- Composite Site Area
- Rain Gauge

Sanitary MH

- Sanitary MH

Sewer

- Local
- Trunk
- Sanitary Subcatchment

AECOM

0 200 400 800 m

Project No.	Date
60285153	October 2014

**I&I Flow
Monitoring Sites**

Figure 4.6

For a more detailed description of the I&I assessment, please refer to **Technical Memorandum #05**. The results of the I&I assessment are summarized below:

- Catchment A represents predominantly single family residential areas. The only major exception is the large industrial lot to the south in Site 7, and a community centre located in the Site 6 catchment.
- Catchment A – The south west portion characterized by Site 7 has a larger than typical I&I, and likely includes direct connections to the sanitary sewer.
- Catchment A – the north east portion characterized by Site 6 indicated I&I did not contribute any significant amount of flow during the observed storms. Estimated I&I rates were lower than the MV target of 11,200 L/ha/d.
- Catchment A – In total was characterized by the composite Site 8-5. The I&I rate was larger than Site 6 and Site 7 indicating the unmonitored portions of the catchment may also be contributing a significant portion of I&I.
- Catchment K represents the Town Centre area of Maple Ridge, covered by monitoring Site 2, Site 3 and Site 4. All of the areas are a mix of multi-family and single family residential land uses, combined with a large portion of commercial and several institutional areas.
- Catchment K – the eastern half characterized by Site 2 indicates a significant portion of inflow, but I&I rates are only marginally above MMCD guidelines.
- Catchment K- the western portion characterized by Site 4 has high I&I, and significant inflows can be observed during wet weather. This area likely has many direct storm connections to the sanitary sewer.
- Catchment K – the southern portion characterized by the composite Site 3-4 had the highest I&I rates in this study. The catchment I&I is predominantly inflow and likely has direct storm connections to the sanitary sewer. The site area and upstream pump station (River Bend) should be studied further.

Due to high variability between the 5yr I&I rates, the City is advised to exercise caution when applying blanket rates to localized areas.

Overall the I&I rates estimated for the whole City are close to or below the Metro Vancouver target rate. There is a greater portion of I&I draining to the South Slope Interceptor, which may be explained by extraneous flows such as leachate that should not be included in the estimates. Potential leachate flow from the Cottonwood landfill would not impact I&I rates calculated for Catchment K, or Catchment A.

4.7 Thornhill Urban Reserve

As part of the assessment of the sanitary system, the City requires an estimate of potential future sanitary flows from the Thornhill Urban Reserve, a 630ha area located to the east of Albion. The City also requires conceptual servicing options for Thornhill. The Urban Reserve is within the urban area boundary but is not serviced by the water or sanitary system. According to the OCP, urban development in the Thornhill area will not occur until the City's population exceeds 100,000 and capacity in the urban area boundary is approaching build-out. Prior to urban development occurring, an area plan is to be developed that will specify land-use patterns, density, and servicing requirements.

In order to assess the impact of future development, the area was reviewed using GIS mapping and contours. The 81 ha park designated area was excluded from the assessment area, along with areas of steep terrain and allowances for creek-buffers. The net area considered developable was 521ha. In a high level nature AECOM estimated the population using a single type of land use density from values in the North Albion & Albion Flats Servicing Strategy. The total residential population estimated was 20,300 residents, which is similar to previous consultants' estimates of approximately 19,400 based on the City's 1996 OCP. An equivalent population was used to represent potential commercial and institutional areas in Thornhill based on the ratio of residential to ICI population in North Albion. A rate of 0.0185 Eq. Pop per Resident was used. The total population may be significantly less than estimated depending on the density agreed to as part of the future area plan.

To generate flow in the model an average dry weather flow (ADWF) of 300 L/cap/day was assumed for the population. An I&I rate of 11,000 L/ha/d was applied to the area, and the diurnal pattern was used in the model to generate peak flows. Thornhill flows were loaded to the model's OCP scenario for the assessment.

To service Thornhill, sanitary flows must be conveyed to the City's 225 Street Pump Station. The general topography of the urban reserve and the physical constraints of the existing sanitary network mean there are only two viable options to route the future flow to the pump station:

- The River Road trunk sewer
- The Cottonwood trunk sewer on Kanaka Way

River Road Trunk Sewer Option:

The Thornhill area can be internally serviced predominantly by gravity sewer by constructing a trunk sewer along 100 Ave/Jackson Rd. Some areas of Thornhill are located at a lower elevation and will require servicing by a pumping station.

For this scenario, the Thornhill design flow was loaded to MH R328 on Jackson Rd at 102 Ave, and a model simulation was run. The model predicted a PWWF of 211 L/s from Thornhill. When combined with the OCP flows, the total PWWF on River Rd was estimated to be up to 306 L/s. This is about double current pipe capacities, therefore in order to service Thornhill the City may require twinning the existing sewer on River Rd instead of replacing existing pipes with larger ones. This option assumes the existing pipes are in ok condition at that time, and will remain in service.

The cost of constructing the new trunk, and twinning of the River Rd. sewer was estimated at approximately **$6.2 M**. Note that costs mentioned are for offsite works only, and do not include purchase of land/ROW, or additional Thornhill pumping costs.

Cottonwood Trunk Sewer Option:

It is possible to service Thornhill via an outlet at 104 Ave to the Cottonwood trunk sewer. Due to the higher elevation at 104 Ave this option will increase the amount of internal area in Thornhill to be serviced by a pump station.

In this scenario the design flows from Thornhill were loaded at MH W213 located on 245B St. The simulation estimated a maximum PWWF of 387 L/s in the Cottonwood trunk sewer when combined with the previously modeled OCP flows. The trunk sewer is able to accommodate the flow predominantly with existing pipe sizes. Approximately 660m of the Cottonwood sewer would require an increased diameter. Most of the local sewers between 245B St and Kanaka Creek will also require upgrades to trunk sewers. As described in section 4.2, the inverted siphon crossing Kanaka Creek at 240 St is expected to be out of capacity by 2023. Replacement of the siphon is assumed to have already taken place for this assessment, and that the new pipe will have sufficient capacity to convey Thornhill flows.

The costs of the sewer upgrades and construction of the 800m sewer from 104 Ave to 245B St are estimated to be **$4.0 M**. This is the more cost effective option and requires the least amount of trunk sewer upgrades. Note that costs mentioned do are for offsite works only, and do not include the purchase of land for ROWs, or any replacement costs for the 240 Street inverted siphon.

Beyond the 225 Street Pump Station, it is estimated the current pipe diameters have sufficient capacity to convey the peak flows from Thornhill to the Katzie Pump Station. See **Technical Memorandum #06** for the full analysis of the Thornhill Urban Reserve and alternative routing options conducted for this assessment.

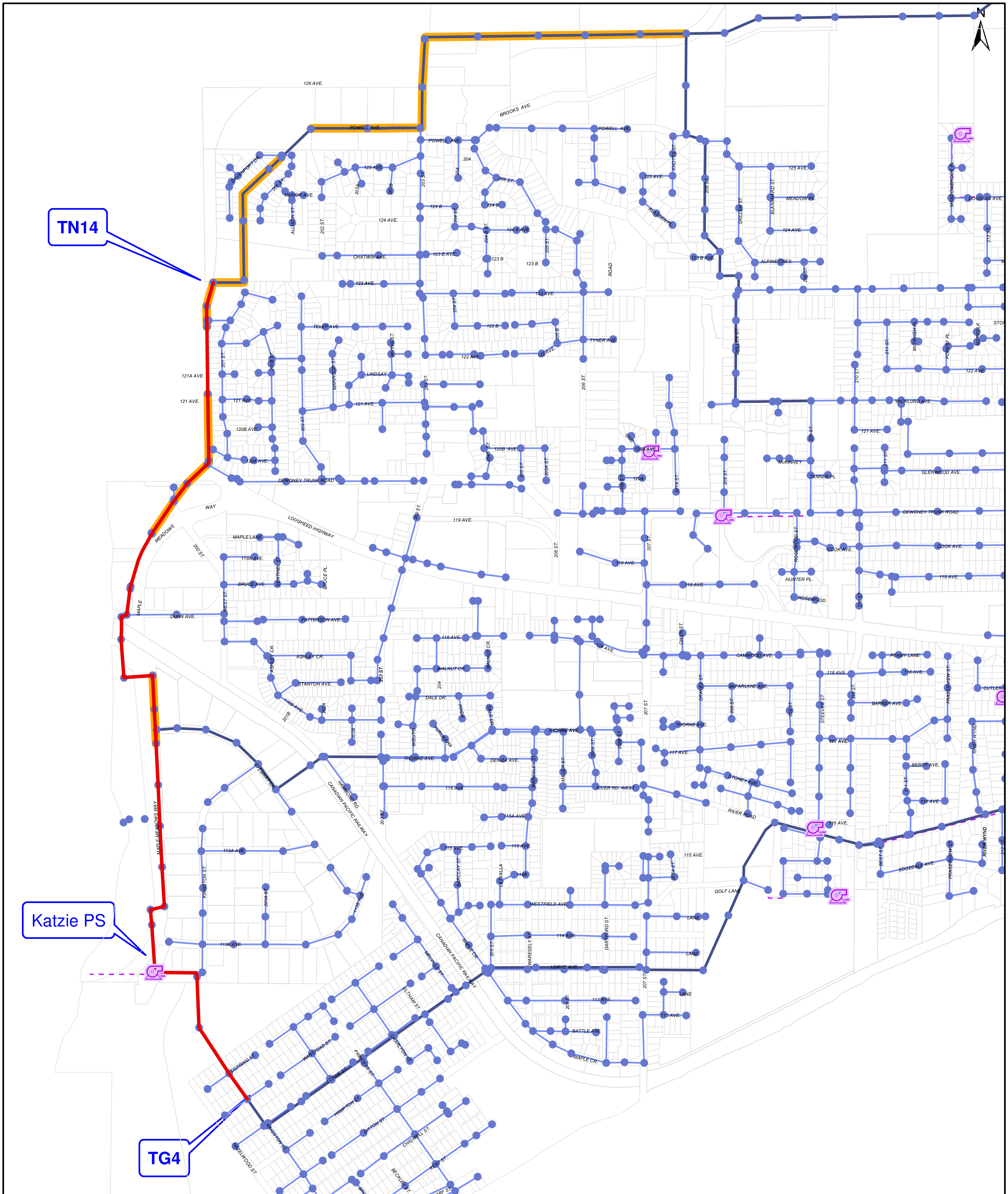
4.8 Backwater Effects

4.8.1 Backwater Effects due to Katzie Pump Station

As discussed in Section 2.2.3, the Katzie Pump Station is the only outlet for sewage from the City. Sewage is conveyed to the station by the City's major trunk sewers, the North Slope and South Slope Interceptors. Both sewers connect to the station at an invert elevation of -2.86m. Metro Vancouver's operating procedures at the Katzie Pump Station result in regular sewage backups in both interceptors under dry weather conditions. The first pump starts when levels reach an elevation of -1.43m and stop at -2.83m elevation. The wet well working volume below the interceptors is relatively small at 80m^3, therefore sewage backups occur with each pump cycle. The full extent of the backup due to pump settings is shown in **Figure 4.7**.

Accumulation of solids will occur as a result of sewage backups. The figure highlights the sections of pipe where the Average Dry Weather Flow velocity is less than 0.6 m/s, which is the minimum for self-cleansing. These sections will be the most susceptible to solids accumulation. Stagnant sewage can lead to corrosive gases forming in the pipes (H2S) which negatively affect the integrity of the concrete. It is recommended that a condition assessment of the interceptor be undertaken to determine if the sewage backups have resulted in pipe degradation, and determine any requirements for maintenance/rehabilitation. Routine maintenance could include periodic cleaning using high velocity cleansing equipment, coupled with a vacuum truck to remove debris and grease accumulations. Inspection and maintenance of the interceptor will require Metro Vancouver to run the Katzie PS and draw down water levels in the interceptor; therefore these works should only be scheduled during low-flow periods in dry months.

During storm events which cause capacity constraints in trunk sewers downstream of Katzie, MV will reduce flow from the station using the pump's Variable Frequency Drives (VFD). The reduced pumping further increases the amount of sewage backwatering in the City's interceptors. The boundary conditions used to develop the sanitary sewer model account for these backwater effects for the current, 2018 and 2023 scenarios. The OCP scenario does not account of backwatering since the Katzie PS is due to be upgraded in the future. It is recommended that the City contact Metro Vancouver to discuss future servicing, and explore options to reduce backwatering of the City's interceptors.



Maple Ridge Sanitary Master Plan

Project No: 60285153

Date: October 2014

Legend

- Pump Station
- Sanitary MH
- Forcemain
- Diversion MH
- Sewage Backwater
- Local Sewer
- Trunk Sewer
- ADWF < Cleansing Vel (0.6 m/s)

AECOM

0 75 150 300 m

Extent of Sewage Backwatering due to Pump Settings

Figure 4.7

4.8.2 Effects of 225 Street Pump Station Shut Down/Failure

AECOM developed several model scenarios to study the effects of a failure or shut down of the City's 225 Street Pump Station during a freshet. In this event, the 225 Street PS would have a sanitary sewer overflow (SSO) into the Fraser River via a connection to the storm sewer on River Rd. The emergency overflow is activated when water levels reach an elevation of 2.63m at the 225 Street PS. The storm sewer would convey flow away from the pump station discharging to an outfall on the Fraser River (invert elevation 0.82m). During a freshet the Fraser River water levels may rise, thus reducing the capacity of the emergency overflow. Table H.3 provided by the City from the *Fraser River Hydraulic Model Update* was used to identify flood stage elevations at the Albion Ferry Dock.

Two scenarios were considered for a pump station shutdown during freshet conditions;

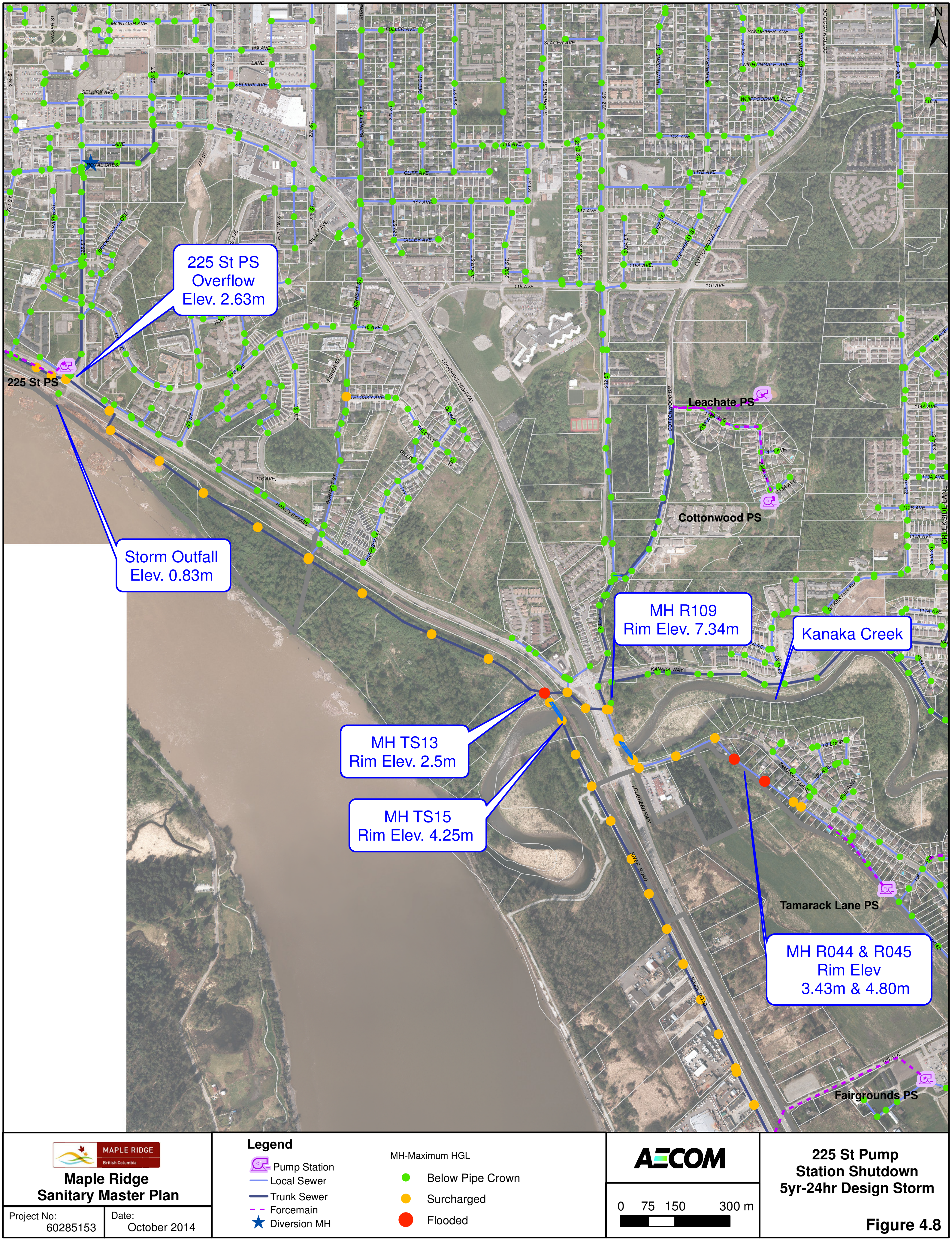
(1) 5 year design storm during river stage elevation of 4.25m, corresponding to typical Fraser River freshet conditions.

(2) Dry weather flows with the river stage elevation at 6.97m. This is the estimated high water level for the 1894 flood which has been adopted as the design flood condition for the Fraser River.

A shut down during the 5 year design storm was also assessed assuming a free flowing outfall, ie Fraser River at normal water levels. The various scenarios of the model analysis identified three manholes at risk for flooding, which are shown in **Figure 4.8**.

MH TS13 is located on the north edge of Kanaka Creek, with a rim elevation of 2.50m; this places the MH below the elevation of the 225 Street PS emergency overflow, therefore it will surcharge and potentially flood before the overflow is activated. Given that the 225 Street PS experiences a number of SSOs each year, this MH should be bolted to the rim (if not already so) and inspected for signs of previous flooding. There are no adjacent properties or developments at risk of flooding, but the model flagged this MH for all three scenarios.

In general, the River Road trunk sewer is located below the elevation of adjacent trunk and local sewers, which minimizes the possibility of an overflow to the upstream network. Model results indicated that MH TS13, due to its low rim elevation, was the only MH at risk of overflow while the 225 Street PS is offline. After MH TS13 the manholes with the lowest rim elevations on the trunk sewer are TS14 and TS11, at 3.19m and 3.80m respectively. Both of these manholes are above the emergency overflow manhole.



MAPLE RIDGE British Columbia

Maple Ridge Sanitary Master Plan

Project No: 60285153

Date: October 2014

AECOM

225 St Pump Station Shutdown 5yr-24hr Design Storm

Figure 4.8

During the first freshet scenario, with the Fraser River water level at 4.25m, many of the manholes on River Road will be submerged. The model does not account for this inundation, showing relief was achieved through MH TS13. An examination of the MH rim elevations on River Road with relation to the river elevation indicates relief would likely be achieved at MH TS15 which is located on the east bank of Kanaka Creek. TS15 has a rim just above 4.25m, and would be the first location where the HGL would be higher than the river, allowing for relief flow.

For the Fraser River design flood, the majority of the River Road trunk sewer will be submerged. This is due to the low elevation of the MHs, and their proximity to the river. Similar to the freshet conditions, An examination of the MH rim elevations helped to identify where possible relief flow may occur. MH TS4, adjacent to the 225 Street PS on Haney Bypass, would be the most likely location for an overflow. At an elevation of 7.02m, it would be the only MH on the trunk sewer not inundated by the 6.97m river level. If the HGL were to rise higher, MH R109 would be the next possible relief location at 7.34m elevation. It is located on a vacant parcel south of Kanaka Way at Lougheed Hwy.

The City has followed up with recommendations in this section and confirmed the MH's mentioned above (MH'sTS12, TS13, TS14, TS15, R052, and R109) have been raised; therefore further investigation will be required to determine the next upstream MH's that would flood.

4.9 CCTV Database Specifications

The CCTV database shall be capable of being linked to through the City's GIS, typically through the use of a unique identification code. The fields of the database and their sources should be as per ***Table 4.6***:

Table 4.6 – Fields for City Pipe Database

FIELD	Type	Length	Example	DESCRIPTION	SOURCE
Unique ID Code	String	8	P01822ST	Unique code for the pipe to allow referencing and linking to other systems.	City assigned.
Pipe material	String	3	AC = Asbestos cement	2 or 3 character code from PACP, should be from a pick list of allowed values.	City records.
Pipe diameter	Short Integer	5	350	Nominal pipe diameter in mm.	City records.
Pipe depth	Short integer	2	2	Depth of pipe in meters.	City records.
Installed year	Short integer	4	1952	Year of installation.	City records.
Cost	Short integer	5	$ 5,182	Original cost of pipe.	City records.
Upstream manhole ID	Short integer	4	2155	ID of upstream manhole or access location.	City assigned.
Downstream manhole ID	Short integer	4	2156	ID of downstream manhole or access location.	City assigned.
Inspection History	N/A	N/A	N/A	Relational table with information on inspection history, see TABLE 2.	From CCTV Contractor.
Structural Grade	String	4	3224	Latest pipe condition, 4 digit PACP Quick rating denoting condition.	From CCTV Contractor
O&M Grade	String	4	5632	Latest pipe O&M rating, 4 digit PACP Quick rating denoting O&M grade.	From CCTV Contractor
Maintenance History	N/A	N/A	N/A	Relational table with information on maintenance history, see TABLE 3.	From City maintenance records.
Next inspection	Date	5	6/2018 = Next inspection from Inspection History Relational Table	The month and year of the next scheduled inspection.	City assigned.
Next maintenance	Date	5	4/2018	The date of the next scheduled maintenance.	City assigned.

In order to track the inspection history of the pipe, a relational table should be used with the fields as described in ***Table 4.7:***

Table 4.7 – Inspection History Fields

FIELD	Type	Length	Example	DESCRIPTION	SOURCE
Unique ID Code	String	8	P01822ST	Unique code for the pipe to allow referencing and linking to other systems.	City assigned.
CCTV Video ID	String	8	VR123414	Unique ID code for video and report.	City assigned.
CCTV Link	String	99	ftp:\\CCTV\VR12314.mpg	Hyperlink to CCTV and report (if available).	City assigned.
CCTV Operator	String	99	John Smith	Name of CCTV Contractor.	CCTV Contractor
Date of inspection	Date	7	1/7/2014	Date of CCTV inspection	CCTV Contractor.
Upstream manhole ID	Short integer	4	2155	ID of upstream manhole or access location.	CCTV Contractor.
Downstream manhole ID	Short integer	4	2156	ID of downstream manhole or access location.	CCTV Contractor.
Direction of Survey	String	1	U	U = Upstream, D = Downstream	CCTV Contractor.
Height (Diameter)	Short Integer	5	300	Pipe maximum diameter if circular, height if non-circular. To nearest mm.	CCTV Contractor.
Width	Short Integer	5	300	For non-circular sewers, maximum width to nearest mm.	CCTV Contractor.
Shape	String	1	Circular = C	1 character code from PACP.	CCTV Contractor.
Material	String	3	Polyethylene = PE	2 or 3 character code from PACP, should be from a pick list of allowed values.	CCTV Contractor.
Pre-Cleaning	String	1	Jetting = J	Code that indicates what cleaning was carried out before survey.	CCTV Contractor.
Structural Grade	String	4	3224	Structural condition grade according to PACP.	CCTV Contractor or updated by verifying engineer.
O&M Grade	String	4	5632	Capacity condition grade according to PACP.	CCTV Contractor or updated by verifying engineer.
Link to report	String	99	ftp:\\CCTV\VR12314.pdf	Link to CCTV Contractors report	CCTV Contractor.
Comments	String	140	Survey abandoned	Any comments from the inspection.	CCTV Contractor.
Next Inspection	Date	5	6/2018 = Next inspection from Inspection History Relational Table	Recommended next inspection.	City engineer.
Maintenance	String	140	Point repair, grout injection at joint.	Recommended maintenance from inspection.	City engineer.

In order to track the maintenance history of the pipe, a relational table should be used with fields as described in ***Table 4.8***:

Table 4.8 – Maintenance History Fields

FIELD	Type	Length	Example	DESCRIPTION	SOURCE
Unique ID Code	String	8	P01822ST	Unique code for the pipe to allow referencing and linking to other systems.	City assigned.
Date of maintenance	Date	5	14/4/2018	Date of pipe maintenance.	Maintenance crew.
Work Crew	String	140	S. Steube, S. Shergill	Crew that undertook maintenance.	Maintenance crew.
Work Undertaken	String	8	Pt Repair	Maintenance performed, should be from a pick list of allowed values.	Maintenance crew.
Location	Short integer	3	12	Distance from upstream manhole in meters.	Maintenance crew.
Quantity	Float	3	2.3	Quantity or length of pipe worked on.	Maintenance crew.
Result	String	140	Lined pipe successfully	Outcome of maintenance.	Maintenance crew.
Cost	Short integer	6	$ 12,000	Cost of maintenance.	Maintenance crew / city records.
Comments	String	140	High groundwater, consider trenchless in future.	Any comments regarding the maintenance.	Maintenance crew.

5 Summary and Recommendations

5.1 Summary

Development of a new City wide sanitary sewers model (“an all pipe model”) has been completed for four different scenarios. The following is a summary of the key conclusions:

- Sanitary sewer model was developed using Innovyze’s InfoSWMM software package version 12;
- Sanitary sewer model was built using the City’s 2012 GIS information for pipe network and the latest census data for populations. Some of the recently constructed sewers over the past two years were added to the model, and likewise future population projections were developed for various planning level scenarios;
- Sewer model includes all of the City’s infrastructure up to its tie-in location at Katzie Pump Station;
- Sewer model was calibrated at 10 locations for dry and wet weather flow conditions and also validated at 28 pump stations (both inflow and outflow). Through this process, a good correlation was achieved;
- The total serviced population of Sites 8, 9 and 10 catchments is estimated at 62,000 people. The weighted average base sanitary flow across the above flow monitoring sites is 191L/capita/day. For the remaining 7 sites, results show that residential base sanitary flow rate varies from 147 to 252 L/capita/day in these catchments. The high variation of per capita flow rate can be attributed to small catchments, where variations in population can have a large impact on unit based sanitary flow rates;
- For ICI properties, the base sanitary flows was input based on 100% of average water consumption during the winter (based on January to March, 2012 water meter records);
- Using the flow and pump station monitoring data, GWI and RDII rates were determined through the calibration process;
- Four (4) engineering planning level scenarios were created in the model: (i) current land-use (ii) 2018 (iii) 2023 and (iv) OCP; and
- Analysis of the hydraulic capacity of the existing sanitary sewers and pump stations was completed for each planning level scenario, with upgrades sized for the OCP condition.

5.2 Recommendations

With the aid of the updated hydraulic model the City will be in a stronger position to plan capital works and respond to queries from developers. Going forward, the following is a list of recommendations for the City with respect to the sewer model, I&I, sewer system capacity and sewer model maintenance:

- The City should continue with I&I Management Plan and flow monitoring. Results from these programs should be used to further calibrate the model in localized catchment areas, and refine GWI and RTK parameters. For the I&I estimates, the RTK parameters and more specifically the “R” parameter has a significant impact on RDII volumes/flows therefore further calibration in un-metered areas would be beneficial.
- If the OCP is modified, the three growth areas may develop to be denser than predicted in the OCP and the City should review the future populations (and resulting sewage flows) for these local areas as development occurs.
- The City should visit all the manholes where surcharging is predicted by the model to observe if evidence of surcharging exists and potentially “bolt down” these manholes to minimize risk of an overflow; especially MH’s TS13 on River Rd, and R044 & R045 on Tamarack Lane.
- The River Road trunk sewer is at currently at capacity in several sections, and will be further under capacity as development continues in the Albion area (See Section 4.2). These sewers should be upgraded as described in **Appendix E**.
- The City should begin planning upgrades for the 225 Street Pump Station and force main. Conservative model predictions estimate the station will be out of capacity after the 2018 planning horizon.
- Modelling estimates show the River Bend pump station to be out of capacity for the current scenario. High rates of I&I identified adjacent to this catchment indicate inflow may be reducing the station’s capacity. This station should be further investigated to confirm existing capacity and inflows prior to planning an upgrade.
- The Fairgrounds and S240 pump stations are expected to be redirected and decommissioned respectively. These plans should be assessed to ensure adequate capacity is provided for the OCP scenario.
- The Fern Crescent Pump Station is estimated to be out of capacity for the OCP scenario. Development in Silver Valley should be reviewed closer to the planning horizon to verify if and when a capacity upgrade is required.
- Due to backwatering from the Katzie Pump Station, a condition assessment should be carried out on the affected areas of the North and South Slope interceptors to determine requirements for maintenance and or rehabilitation.
- It is recommended that the City contact Metro Vancouver to discuss servicing by the Katzie Pump Station, which is to be upgraded in the near future, and explore options to reduce backwatering of the City’s interceptors.

Appendix A

Wet Weather Calibration Results

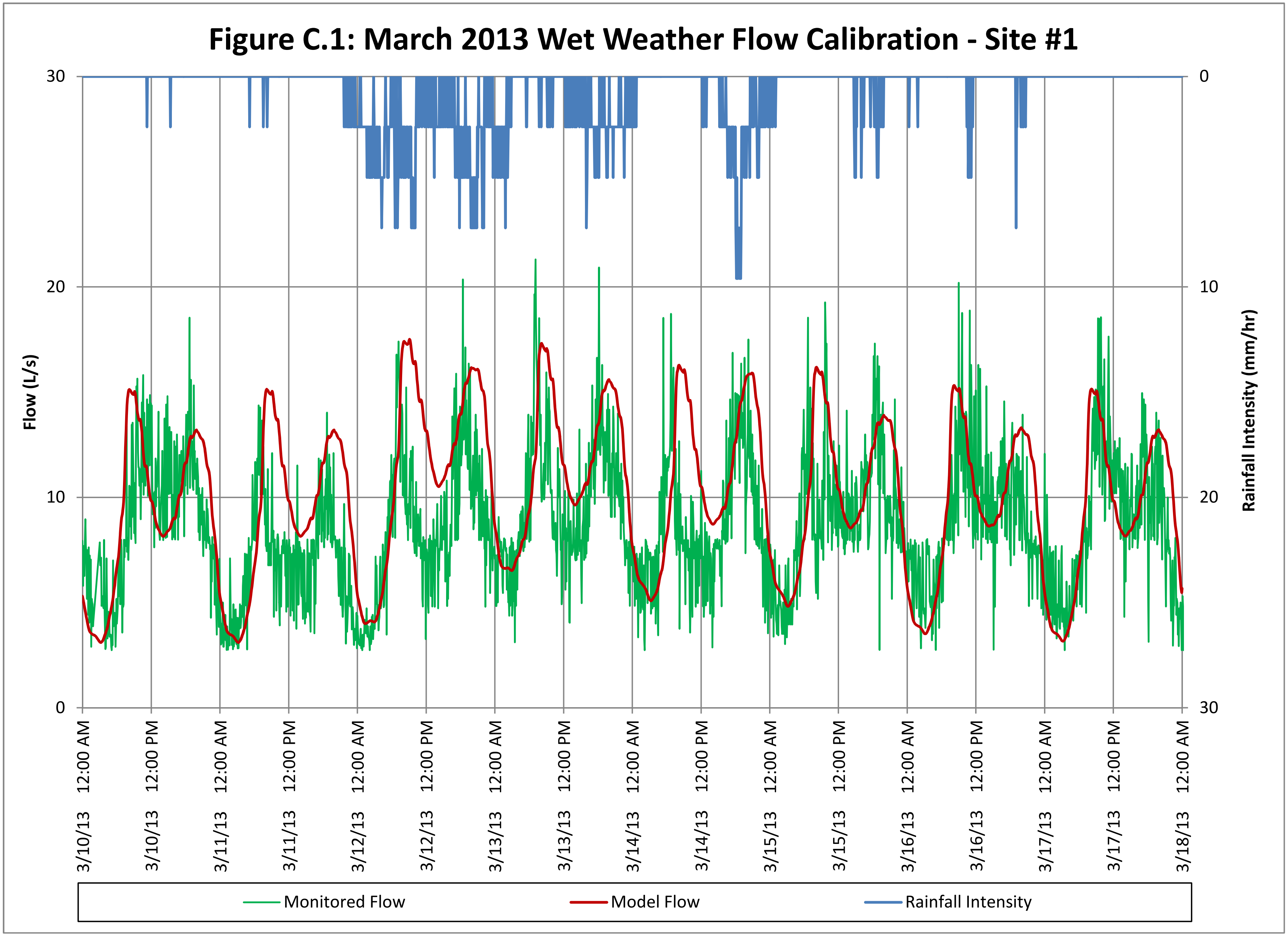
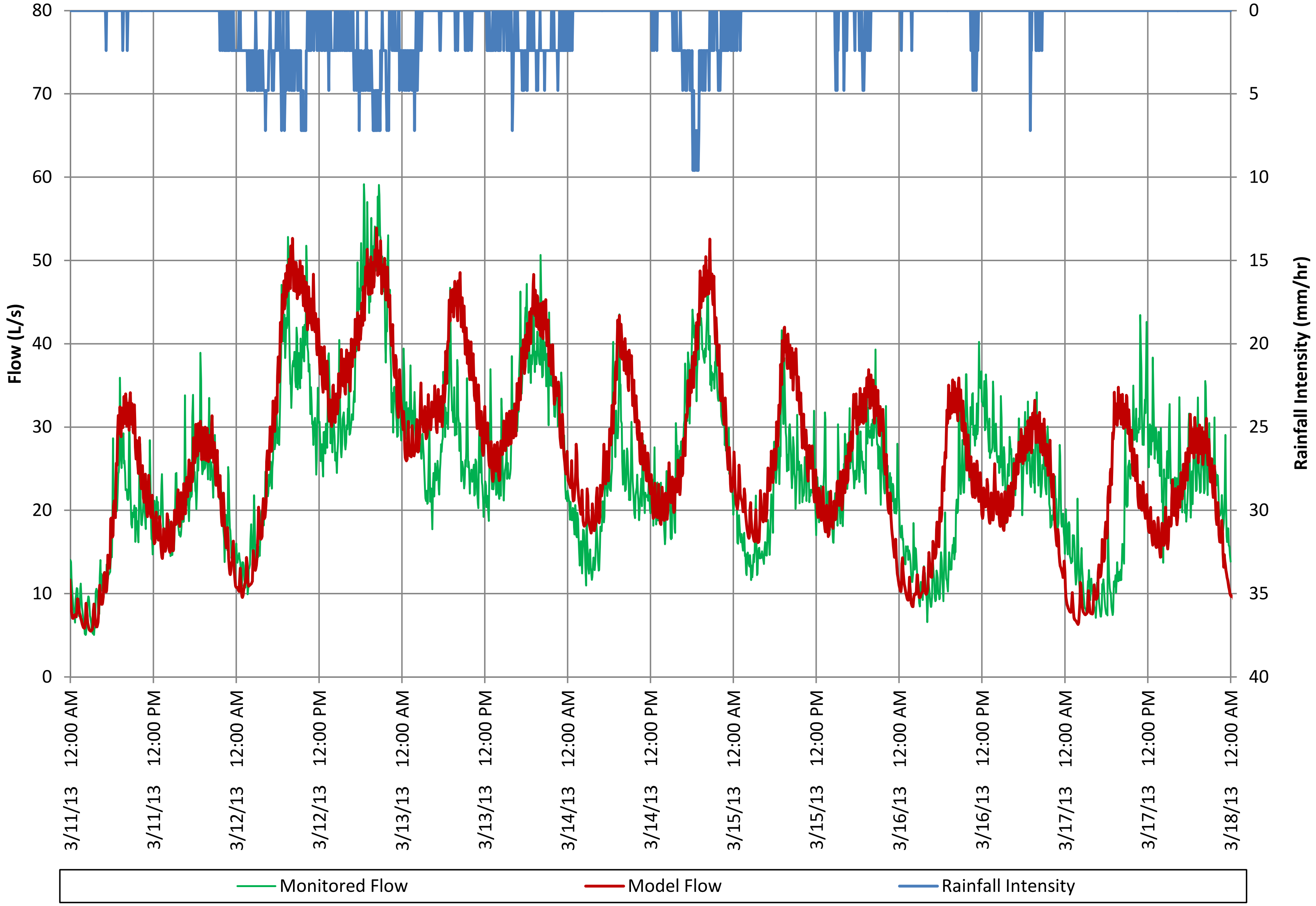


Figure C.1: March 2013 Wet Weather Flow Calibration - Site #1
Flow (L/s)
Rainfall Intensity (mm/hr)
0
10
20
30
3/10/13 12:00 AM
3/10/13 12:00 PM
3/11/13 12:00 AM
3/11/13 12:00 PM
3/12/13 12:00 AM
3/12/13 12:00 PM
3/13/13 12:00 AM
3/13/13 12:00 PM
3/14/13 12:00 AM
3/14/13 12:00 PM
3/15/13 12:00 AM
3/15/13 12:00 PM
3/16/13 12:00 AM
3/16/13 12:00 PM
3/17/13 12:00 AM
3/17/13 12:00 PM
3/18/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

Figure C.2: March 2013 Wet Weather Flow Calibration - Site #2



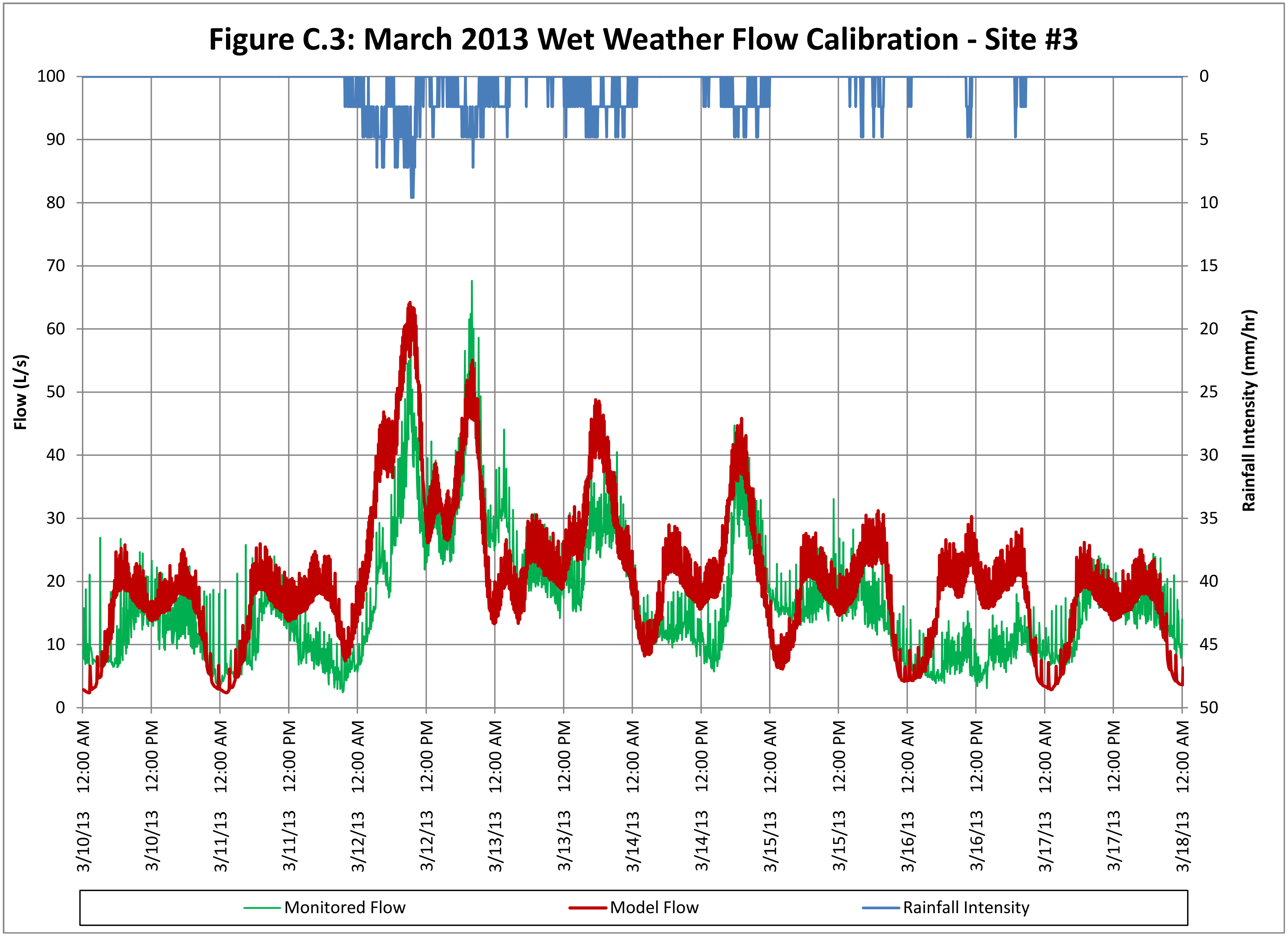
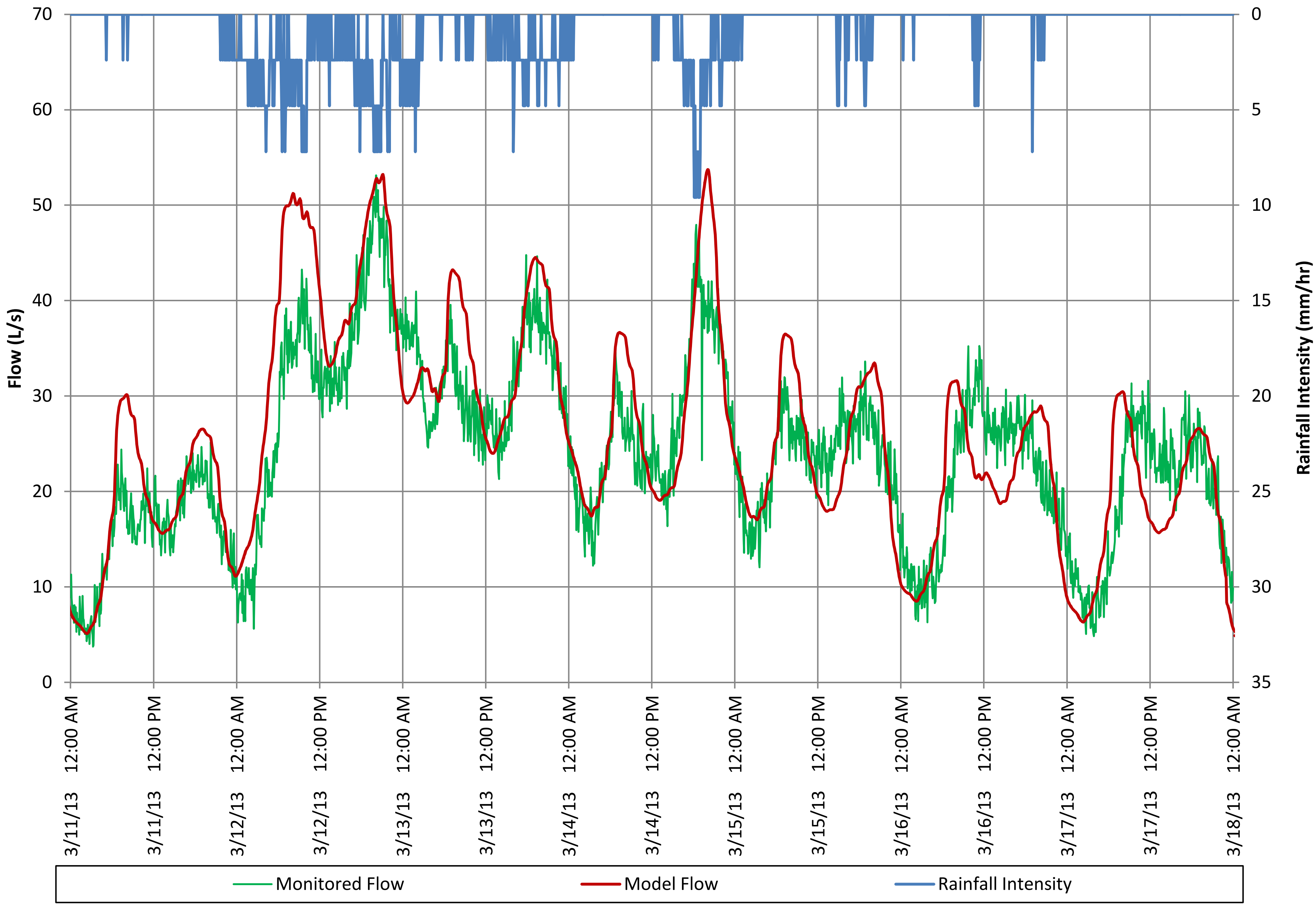


Figure C.3: March 2013 Wet Weather Flow Calibration - Site #3
Flow (L/s)
Rainfall Intensity (mm/hr)
100
90
80
70
60
50
40
30
20
10
0
5
15
25
35
45
12:00 AM 3/10/13
12:00 PM 3/10/13
12:00 AM 3/11/13
12:00 PM 3/11/13
12:00 AM 3/12/13
12:00 PM 3/12/13
12:00 AM 3/13/13
12:00 PM 3/13/13
12:00 AM 3/14/13
12:00 PM 3/14/13
12:00 AM 3/15/13
12:00 PM 3/15/13
12:00 AM 3/16/13
12:00 PM 3/16/13
12:00 AM 3/17/13
12:00 PM 3/17/13
12:00 AM 3/18/13
Monitored Flow
Model Flow
Rainfall Intensity

Figure C.4: March 2013 Wet Weather Flow Calibration - Site #4



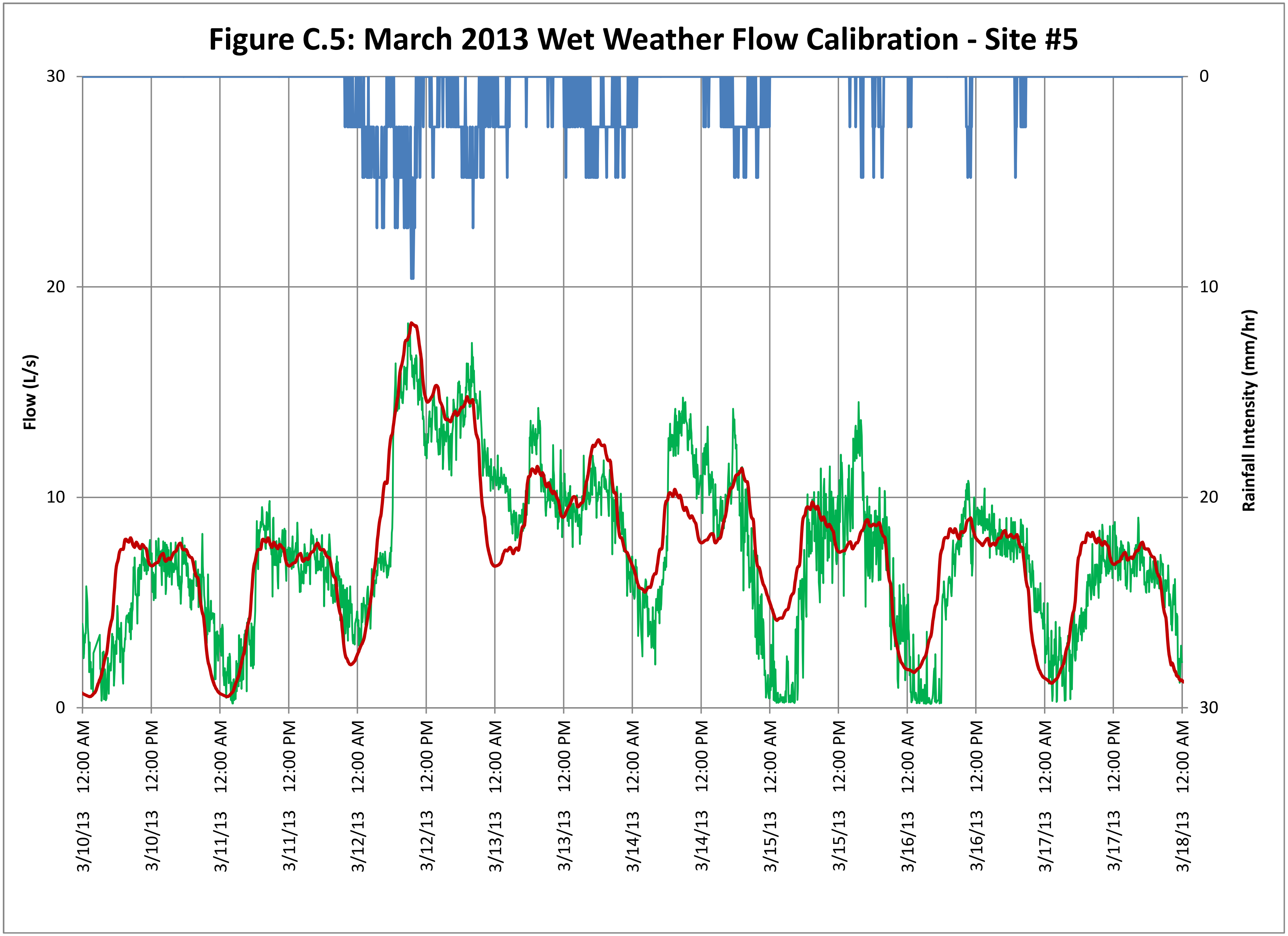


Figure C.5: March 2013 Wet Weather Flow Calibration - Site #5
Flow (L/s)
Rainfall Intensity (mm/hr)
30
20
10
0
0
10
20
30
3/10/13 12:00 AM
3/10/13 12:00 PM
3/11/13 12:00 AM
3/11/13 12:00 PM
3/12/13 12:00 AM
3/12/13 12:00 PM
3/13/13 12:00 AM
3/13/13 12:00 PM
3/14/13 12:00 AM
3/14/13 12:00 PM
3/15/13 12:00 AM
3/15/13 12:00 PM
3/16/13 12:00 AM
3/16/13 12:00 PM
3/17/13 12:00 AM
3/17/13 12:00 PM
3/18/13 12:00 AM

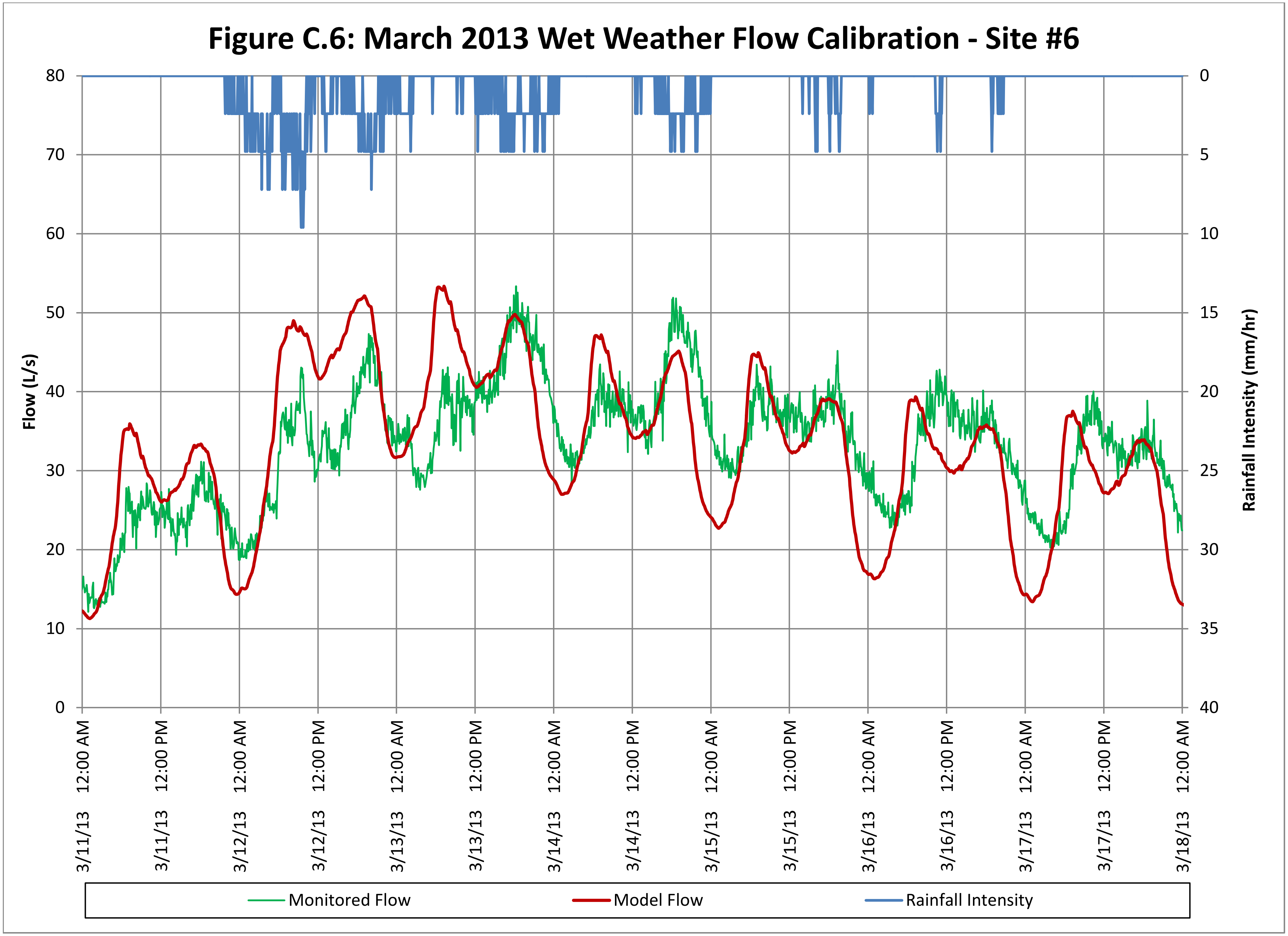


Figure C.6: March 2013 Wet Weather Flow Calibration - Site #6
Flow (L/s)
Rainfall Intensity (mm/hr)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
12:00 AM 3/11/13
12:00 PM 3/11/13
12:00 AM 3/12/13
12:00 PM 3/12/13
12:00 AM 3/13/13
12:00 PM 3/13/13
12:00 AM 3/14/13
12:00 PM 3/14/13
12:00 AM 3/15/13
12:00 PM 3/15/13
12:00 AM 3/16/13
12:00 PM 3/16/13
12:00 AM 3/17/13
12:00 PM 3/17/13
12:00 AM 3/18/13
Monitored Flow
Model Flow
Rainfall Intensity

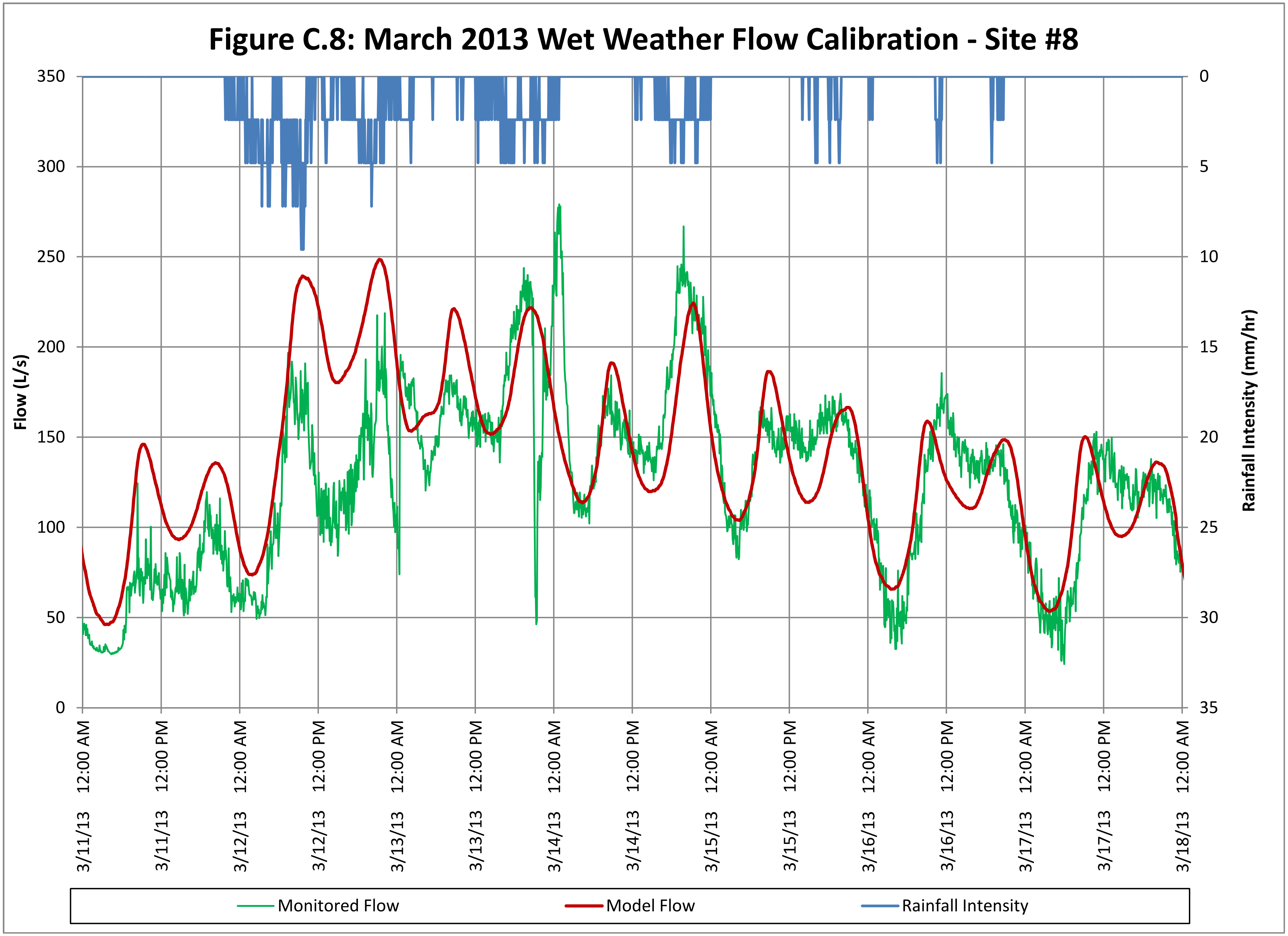


Figure C.8: March 2013 Wet Weather Flow Calibration - Site #8
Flow (L/s)
Rainfall Intensity (mm/hr)
0
50
100
150
200
250
300
350
5
10
15
20
25
30
35
3/11/13 12:00 AM
3/11/13 12:00 PM
3/12/13 12:00 AM
3/12/13 12:00 PM
3/13/13 12:00 AM
3/13/13 12:00 PM
3/14/13 12:00 AM
3/14/13 12:00 PM
3/15/13 12:00 AM
3/15/13 12:00 PM
3/16/13 12:00 AM
3/16/13 12:00 PM
3/17/13 12:00 AM
3/17/13 12:00 PM
3/18/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

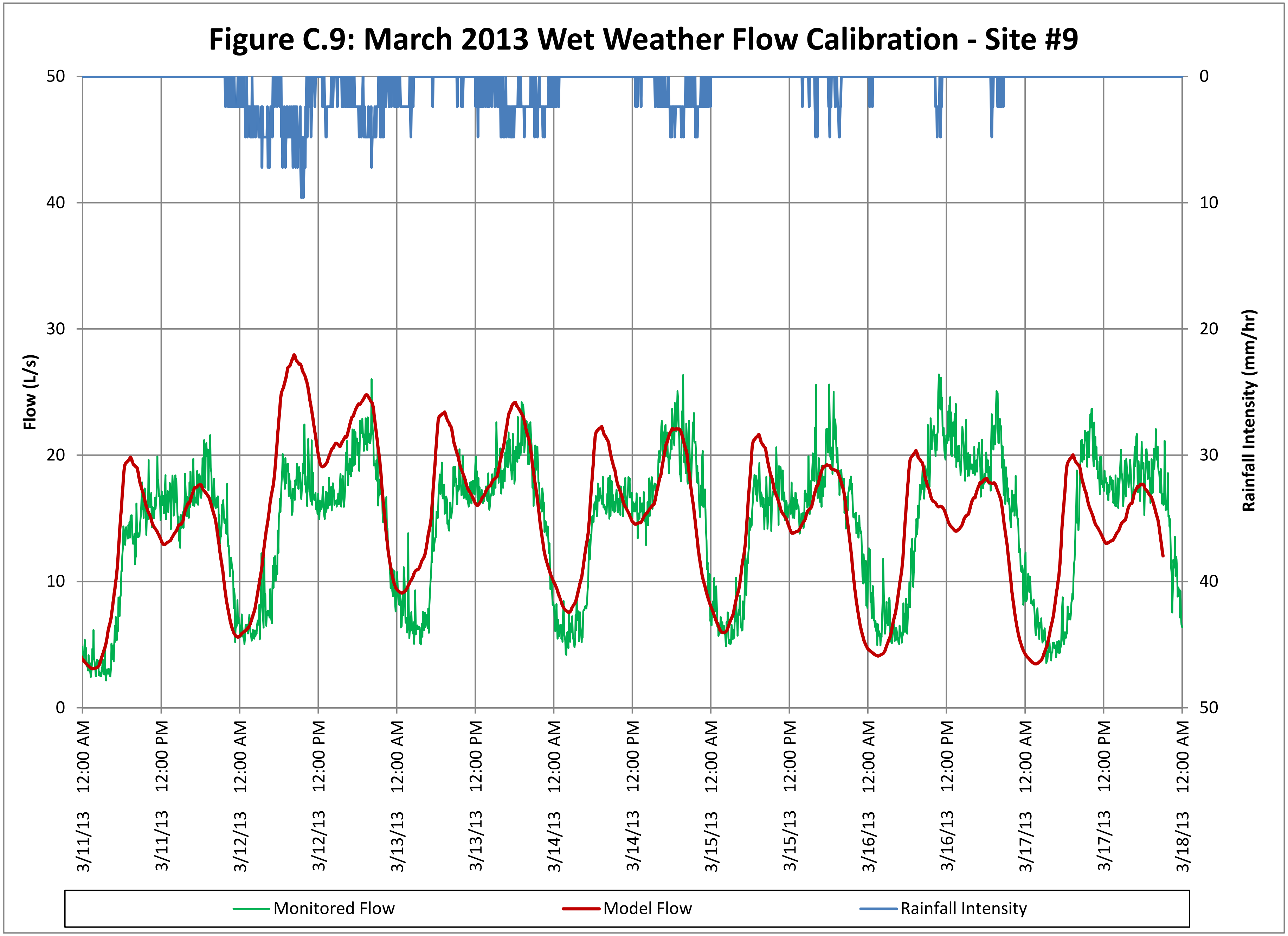


Figure C.9: March 2013 Wet Weather Flow Calibration - Site #9
Flow (L/s)
Rainfall Intensity (mm/hr)
0
10
20
30
40
50
3/11/13 12:00 AM
3/11/13 12:00 PM
3/12/13 12:00 AM
3/12/13 12:00 PM
3/13/13 12:00 AM
3/13/13 12:00 PM
3/14/13 12:00 AM
3/14/13 12:00 PM
3/15/13 12:00 AM
3/15/13 12:00 PM
3/16/13 12:00 AM
3/16/13 12:00 PM
3/17/13 12:00 AM
3/17/13 12:00 PM
3/18/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

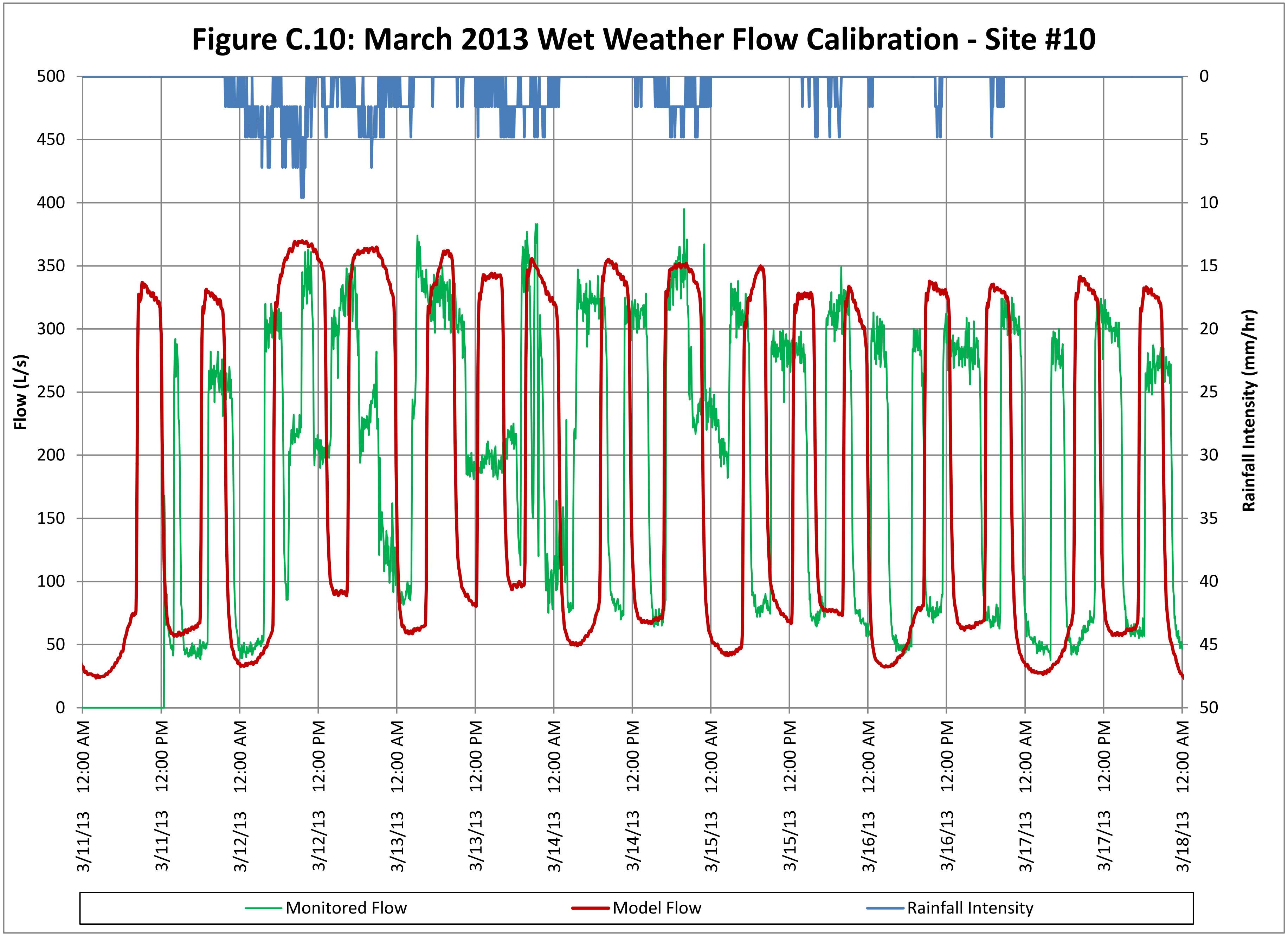
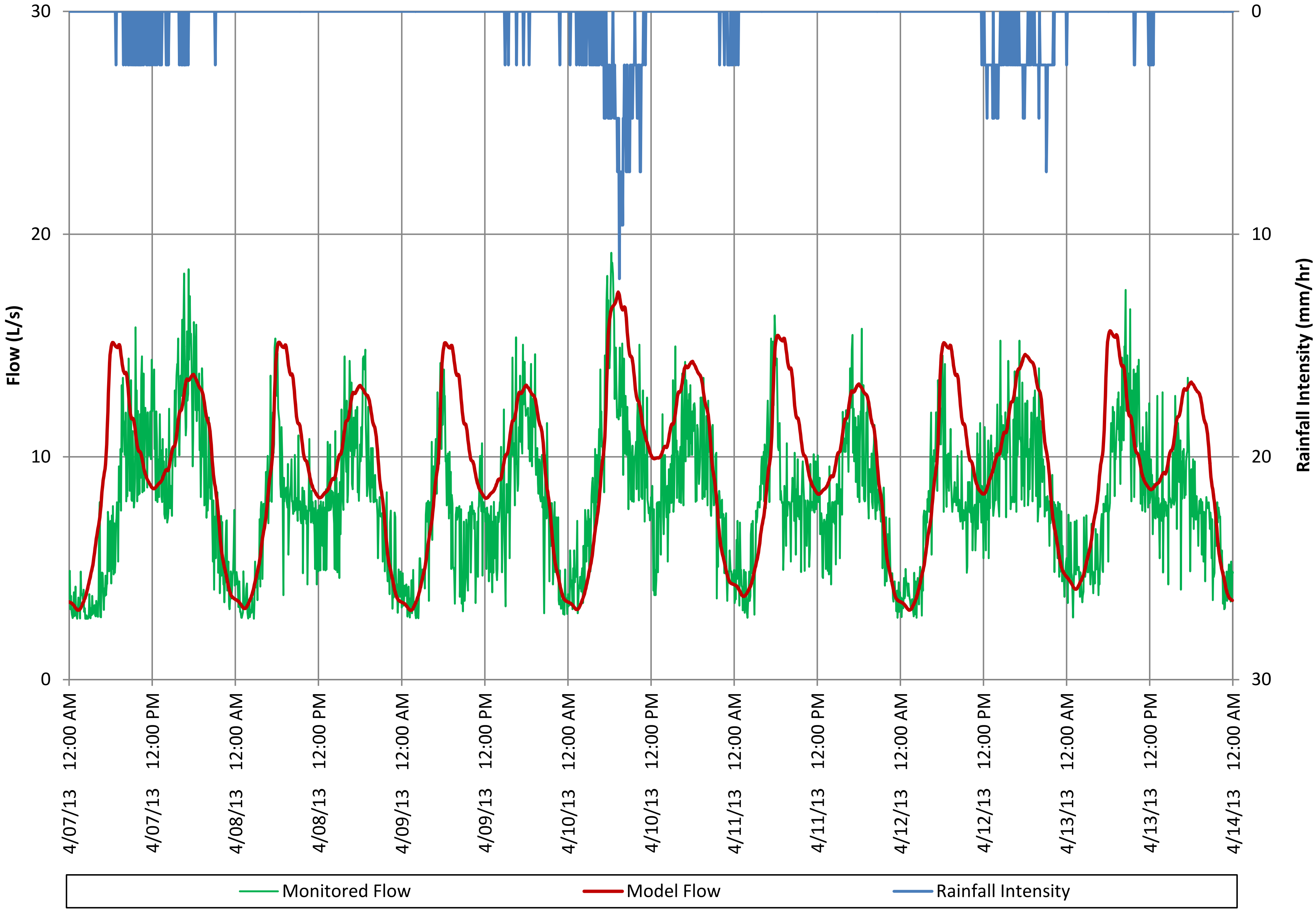


Figure C.10: March 2013 Wet Weather Flow Calibration - Site #10
Flow (L/s)
500
450
400
350
300
250
200
150
100
50
0
Rainfall Intensity (mm/hr)
0
5
10
15
20
25
30
35
40
45
50
3/11/13 12:00 AM
3/11/13 12:00 PM
3/12/13 12:00 AM
3/12/13 12:00 PM
3/13/13 12:00 AM
3/13/13 12:00 PM
3/14/13 12:00 AM
3/14/13 12:00 PM
3/15/13 12:00 AM
3/15/13 12:00 PM
3/16/13 12:00 AM
3/16/13 12:00 PM
3/17/13 12:00 AM
3/17/13 12:00 PM
3/18/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

Appendix B

Model Validation Results

Figure D.1: April 2013 Wet Weather Flow Calibration - Site #1



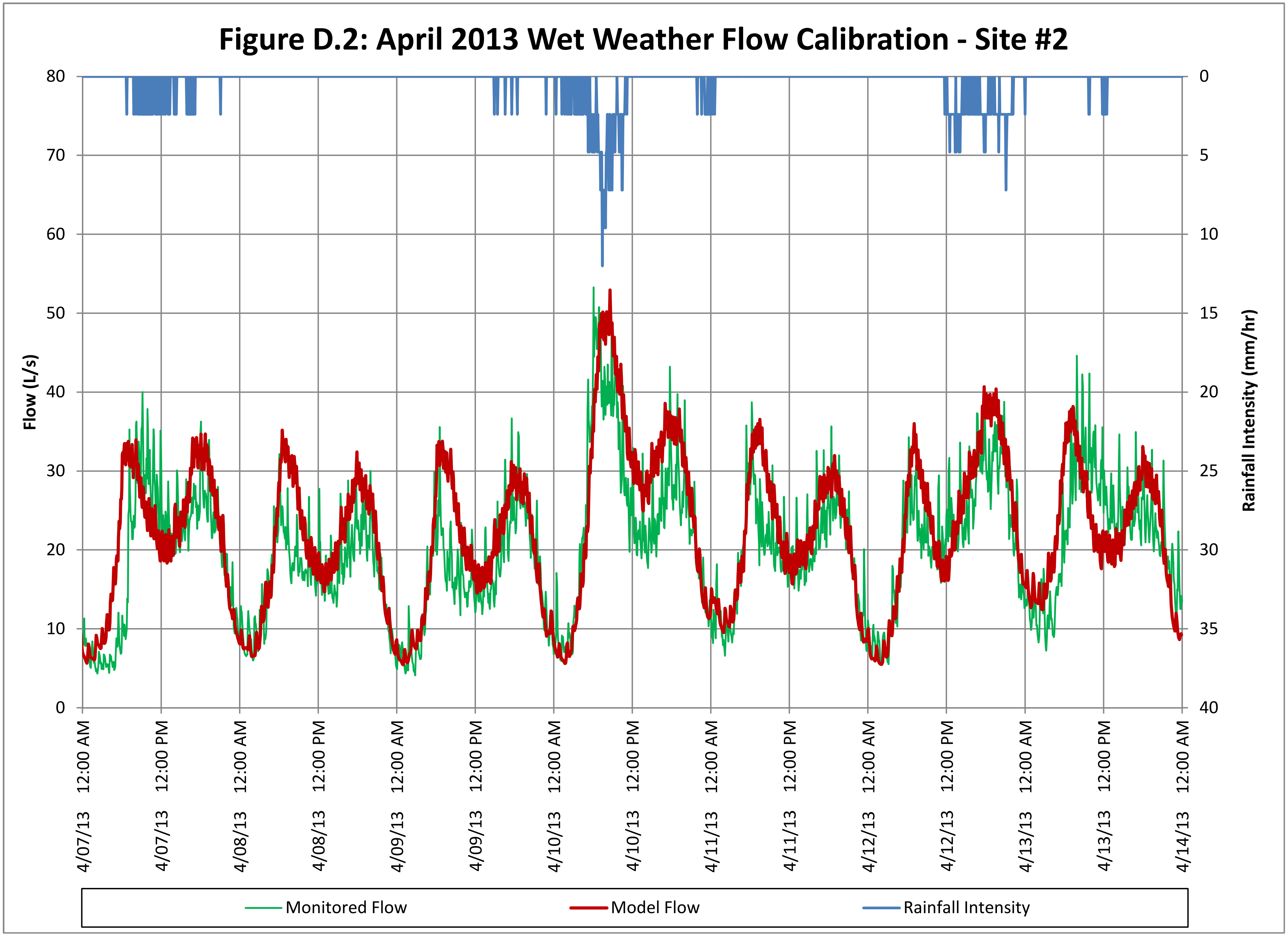


Figure D.2: April 2013 Wet Weather Flow Calibration - Site #2
Flow (L/s)
Rainfall Intensity (mm/hr)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

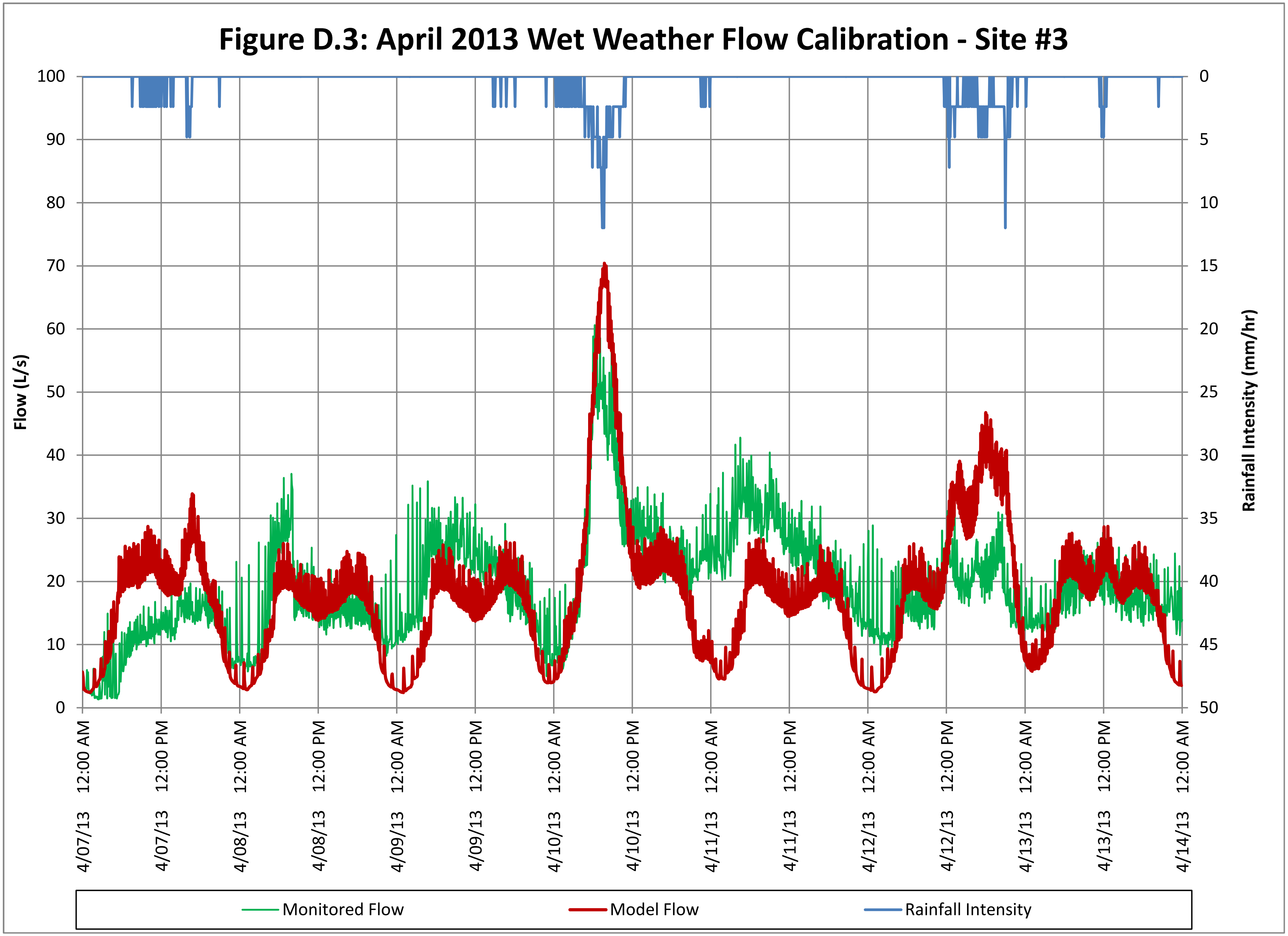


Figure D.3: April 2013 Wet Weather Flow Calibration - Site #3
Flow (L/s)
0
10
20
30
40
50
60
70
80
90
100
Rainfall Intensity (mm/hr)
0
5
10
15
20
25
30
35
40
45
50
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

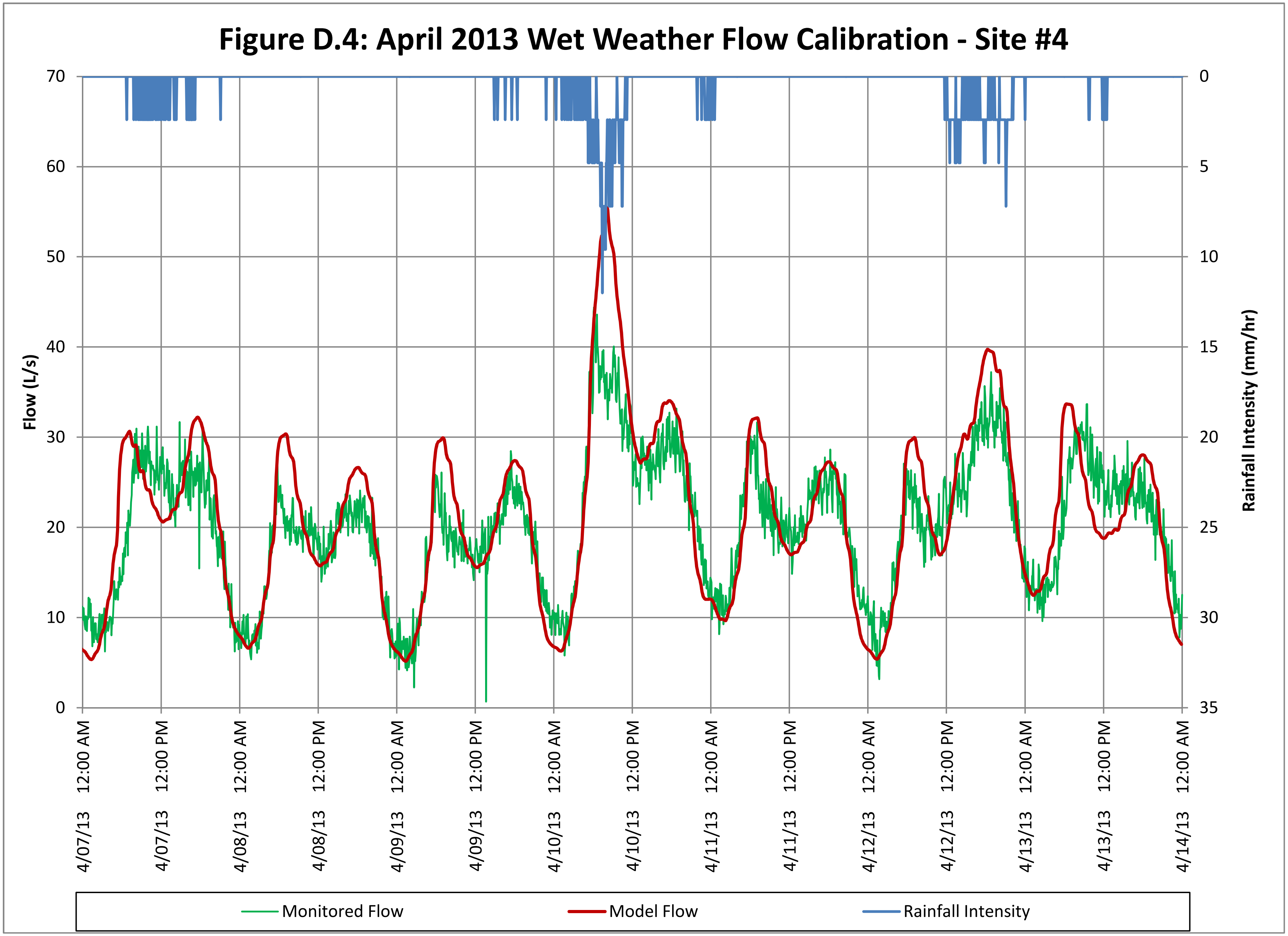
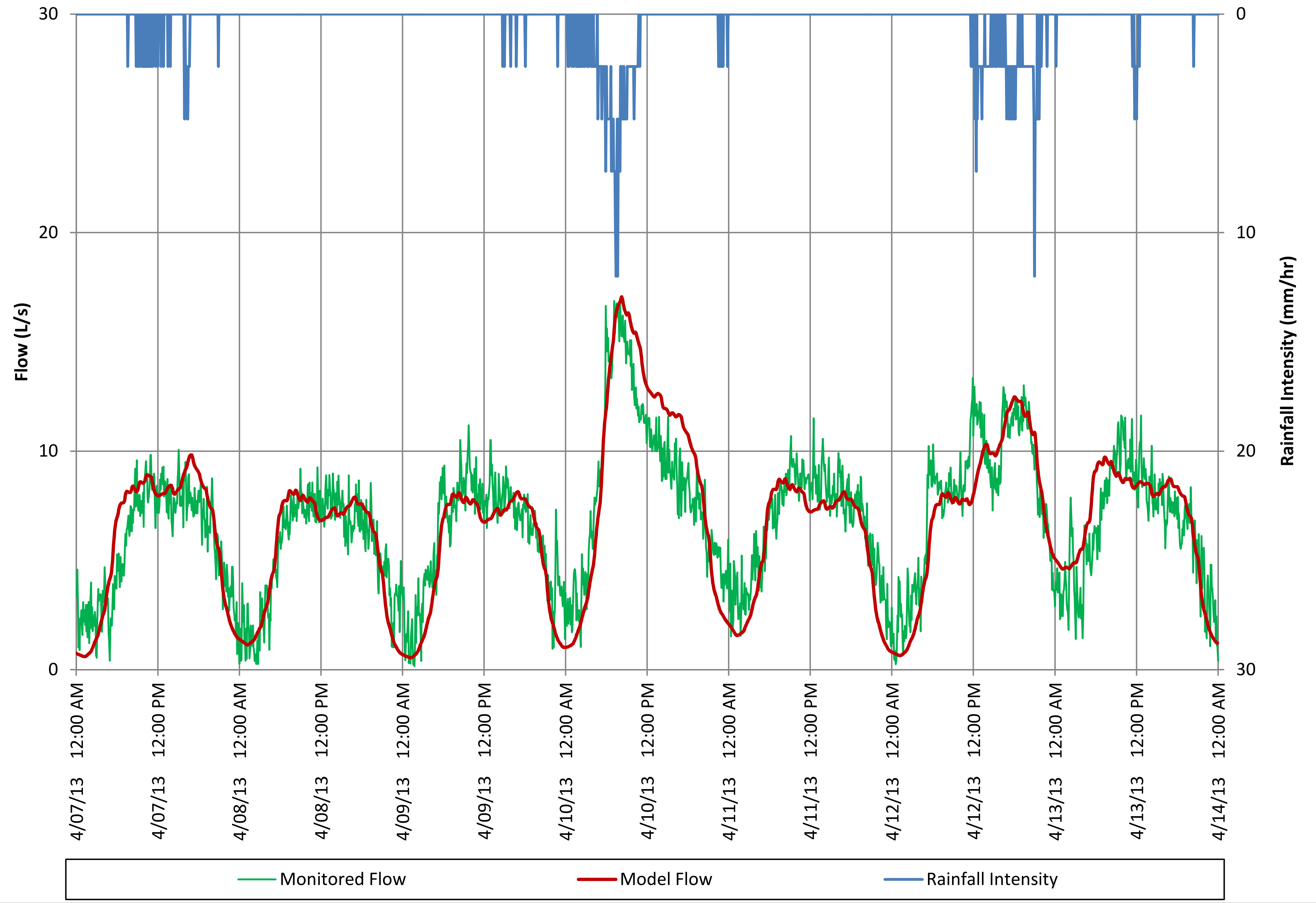


Figure D.4: April 2013 Wet Weather Flow Calibration - Site #4
Flow (L/s)
Rainfall Intensity (mm/hr)
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

Figure D.5: April 2013 Wet Weather Flow Calibration - Site #5



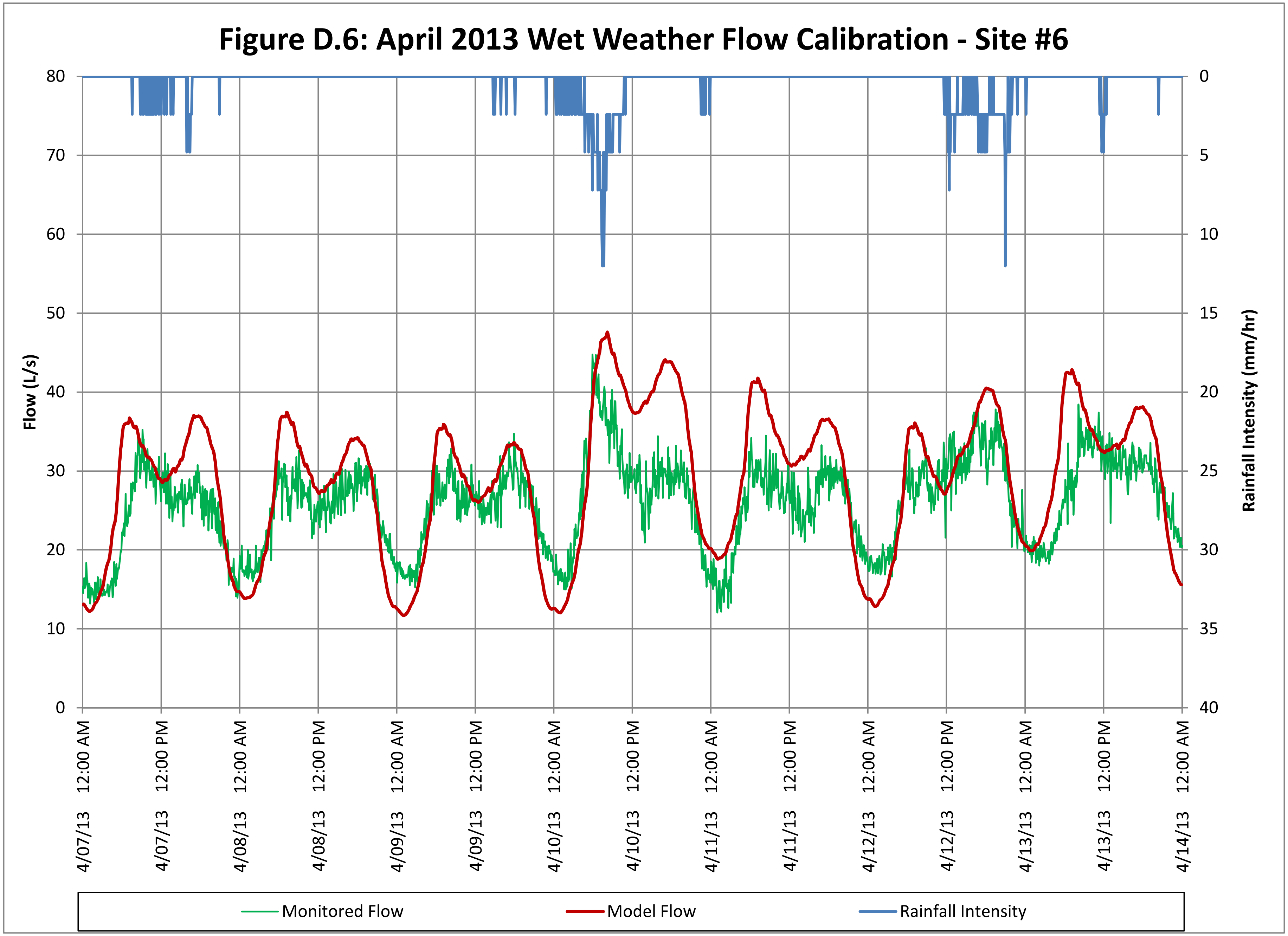


Figure D.6: April 2013 Wet Weather Flow Calibration - Site #6
Flow (L/s)
Rainfall Intensity (mm/hr)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
12:00 AM 4/07/13
12:00 PM 4/07/13
12:00 AM 4/08/13
12:00 PM 4/08/13
12:00 AM 4/09/13
12:00 PM 4/09/13
12:00 AM 4/10/13
12:00 PM 4/10/13
12:00 AM 4/11/13
12:00 PM 4/11/13
12:00 AM 4/12/13
12:00 PM 4/12/13
12:00 AM 4/13/13
12:00 PM 4/13/13
12:00 AM 4/14/13
Monitored Flow
Model Flow
Rainfall Intensity

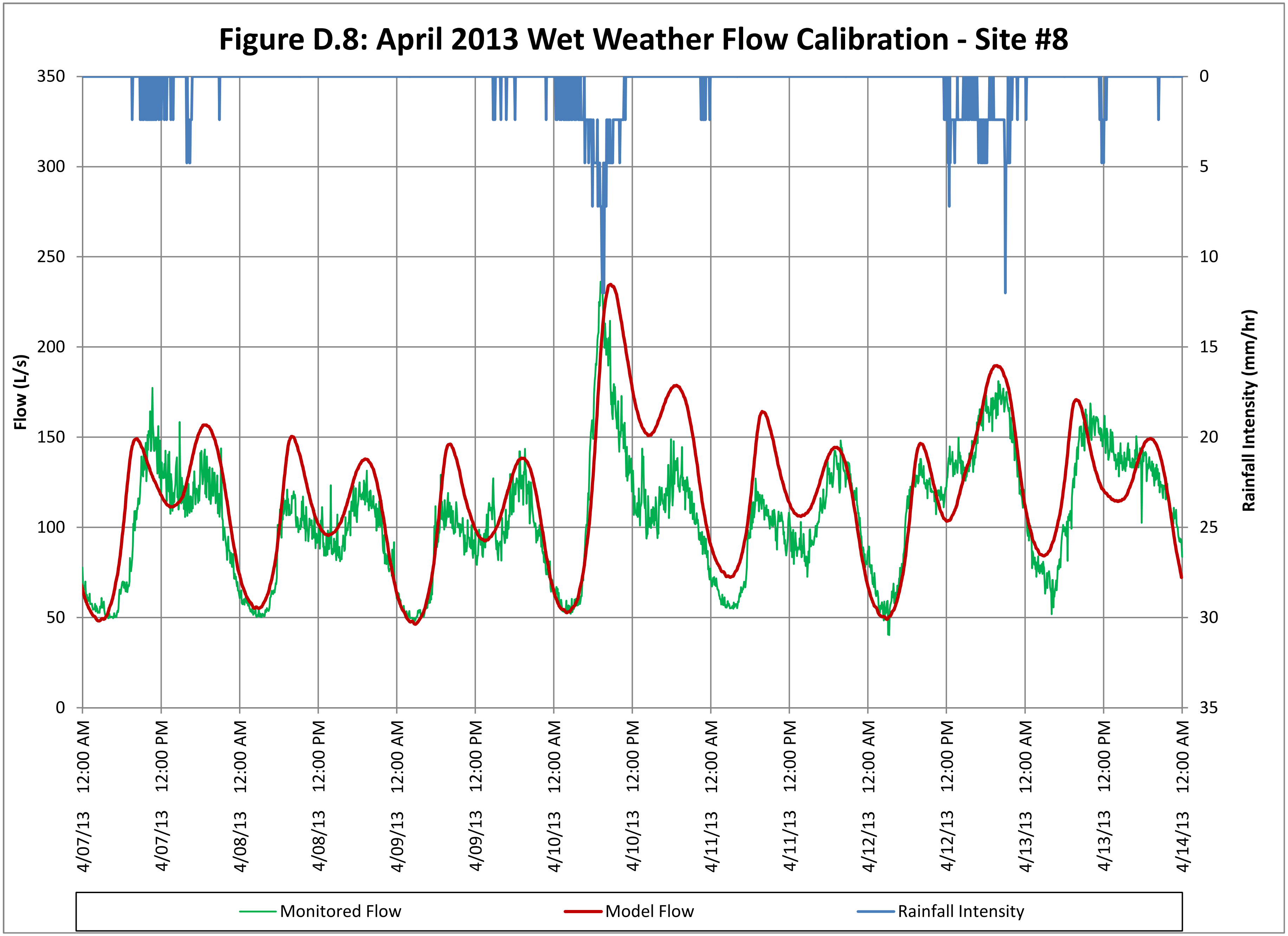


Figure D.8: April 2013 Wet Weather Flow Calibration - Site #8
Flow (L/s)
Rainfall Intensity (mm/hr)
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

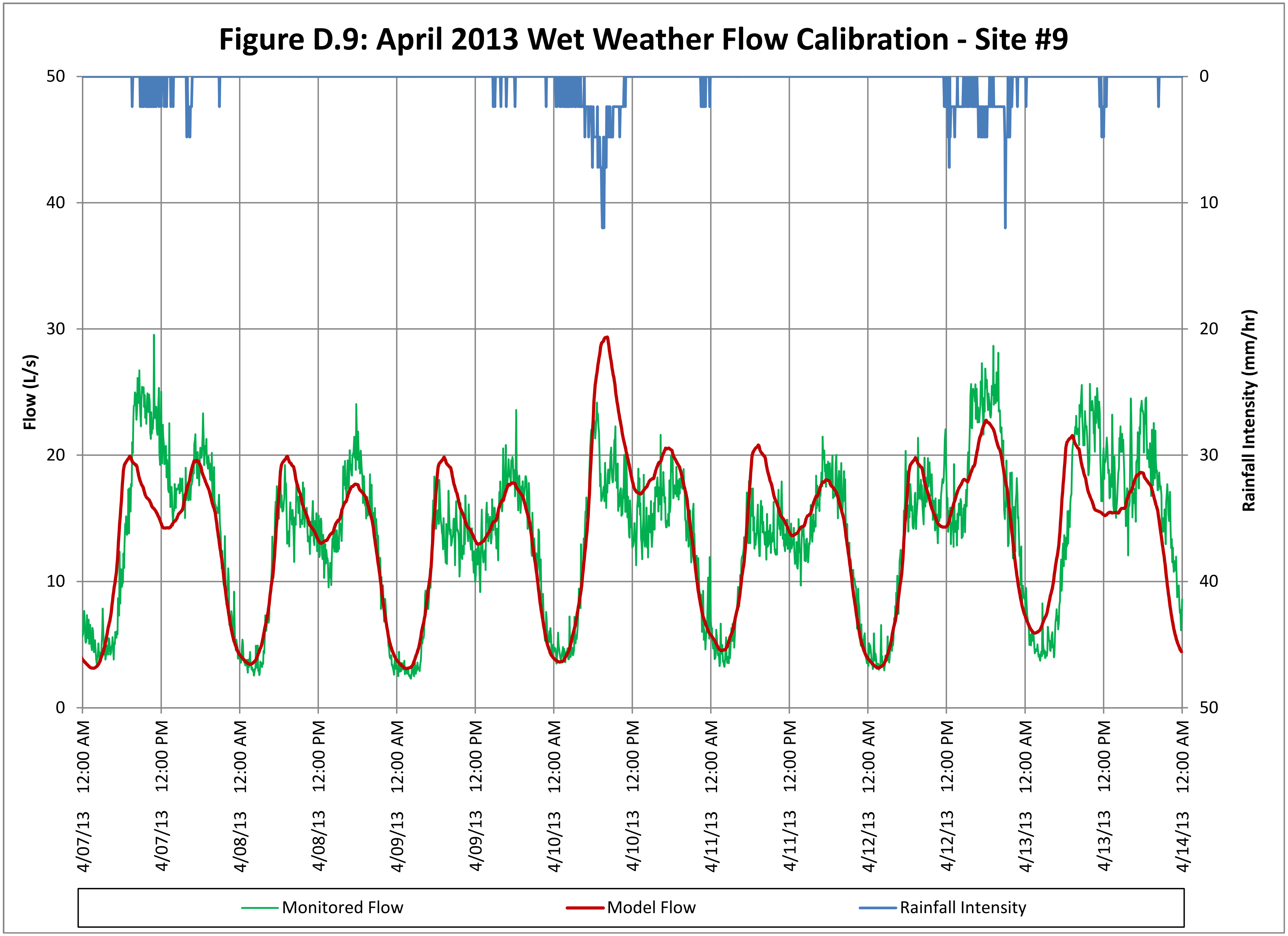


Figure D.9: April 2013 Wet Weather Flow Calibration - Site #9
Flow (L/s)
Rainfall Intensity (mm/hr)
50
40
30
20
10
0
0
10
20
30
40
50
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

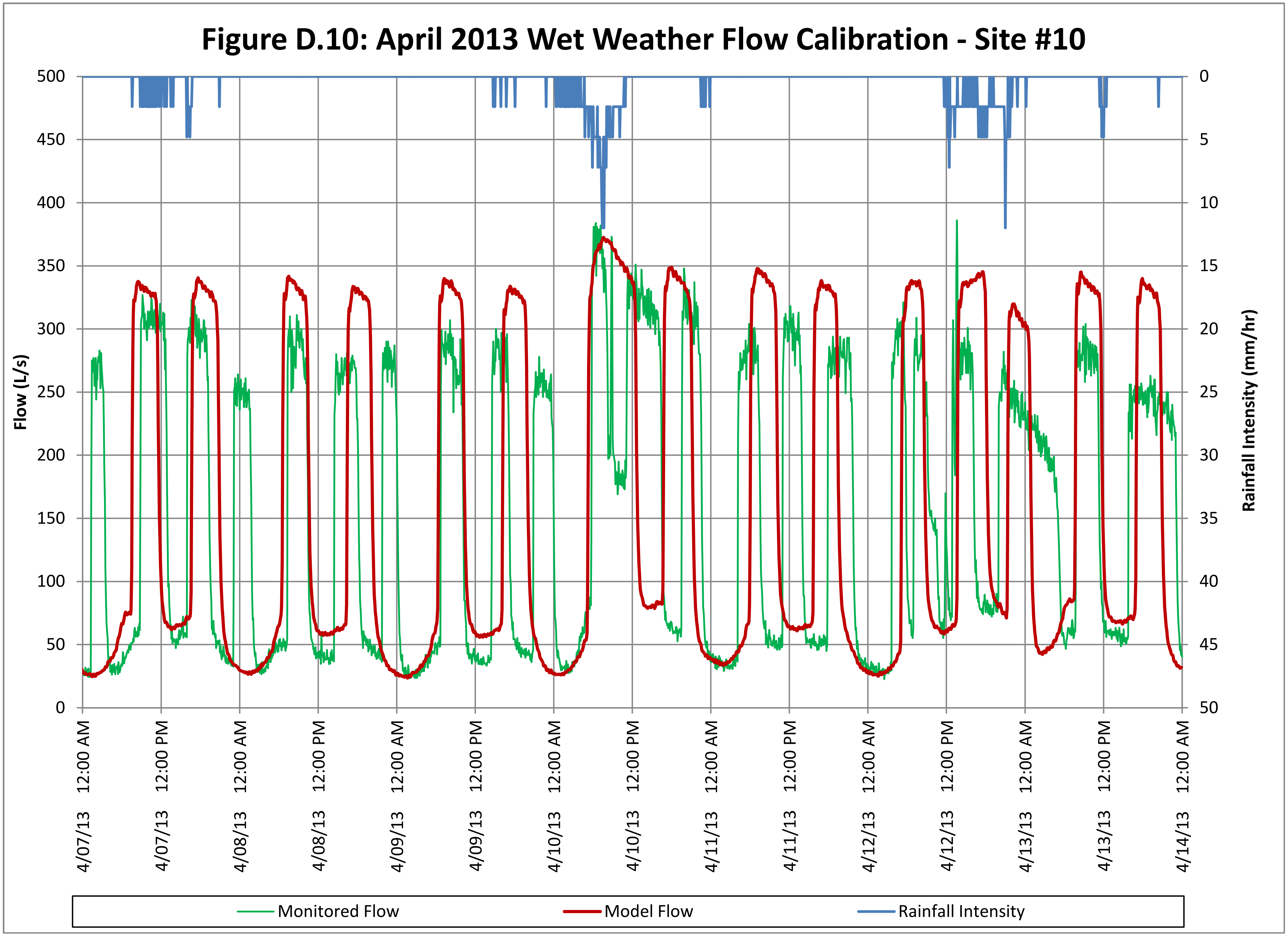
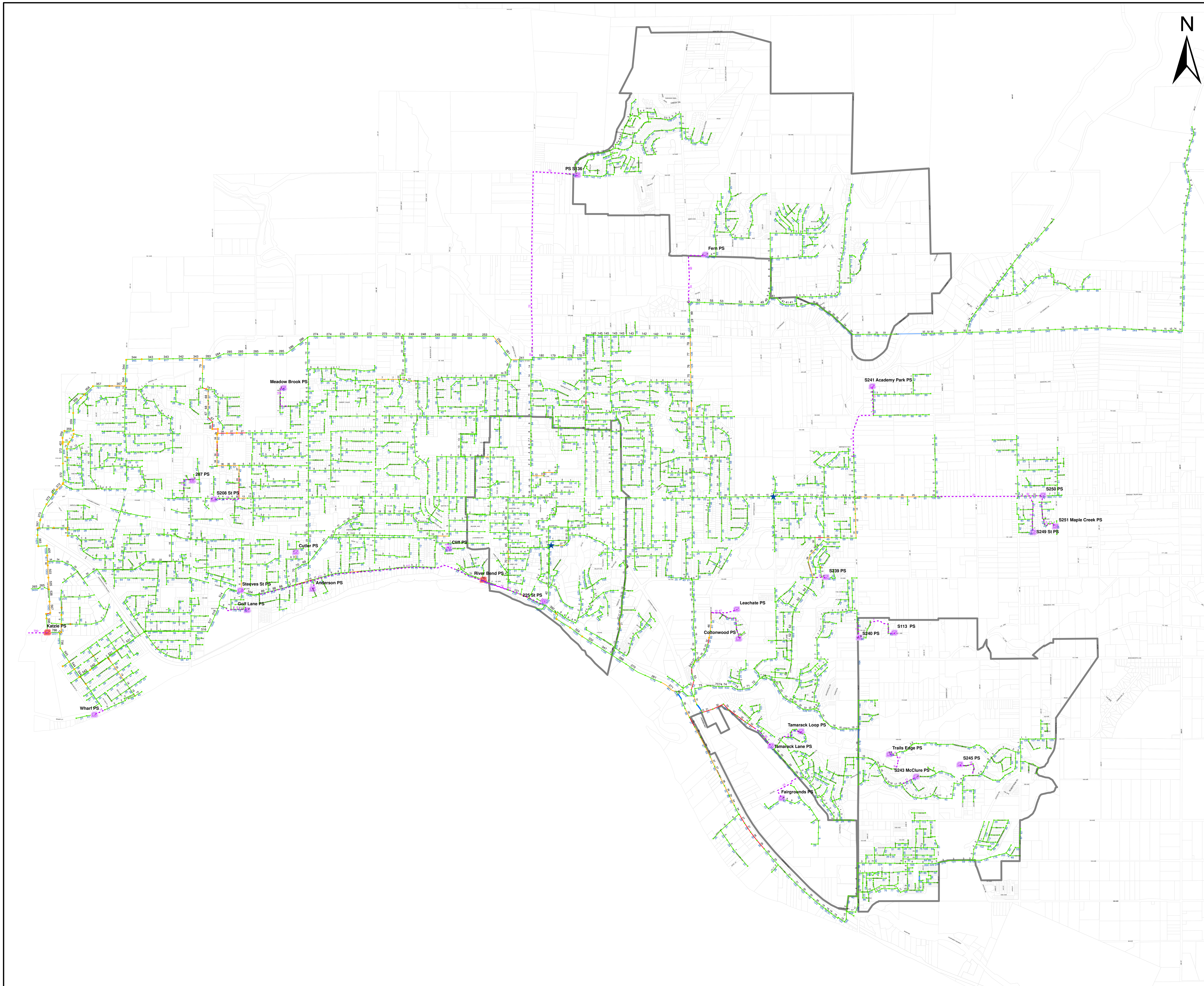


Figure D.10: April 2013 Wet Weather Flow Calibration - Site #10
Flow (L/s)
Rainfall Intensity (mm/hr)
500
450
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
45
50
4/07/13 12:00 AM
4/07/13 12:00 PM
4/08/13 12:00 AM
4/08/13 12:00 PM
4/09/13 12:00 AM
4/09/13 12:00 PM
4/10/13 12:00 AM
4/10/13 12:00 PM
4/11/13 12:00 AM
4/11/13 12:00 PM
4/12/13 12:00 AM
4/12/13 12:00 PM
4/13/13 12:00 AM
4/13/13 12:00 PM
4/14/13 12:00 AM
Monitored Flow
Model Flow
Rainfall Intensity

Appendix C

Full Size Hard Copy Maps of Model Results and Proposed Upgrades



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 | Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 | Q<40 L/s; q/Q>0.70

Peak Flow (L/s)
Full Flow Capacity (L/s)

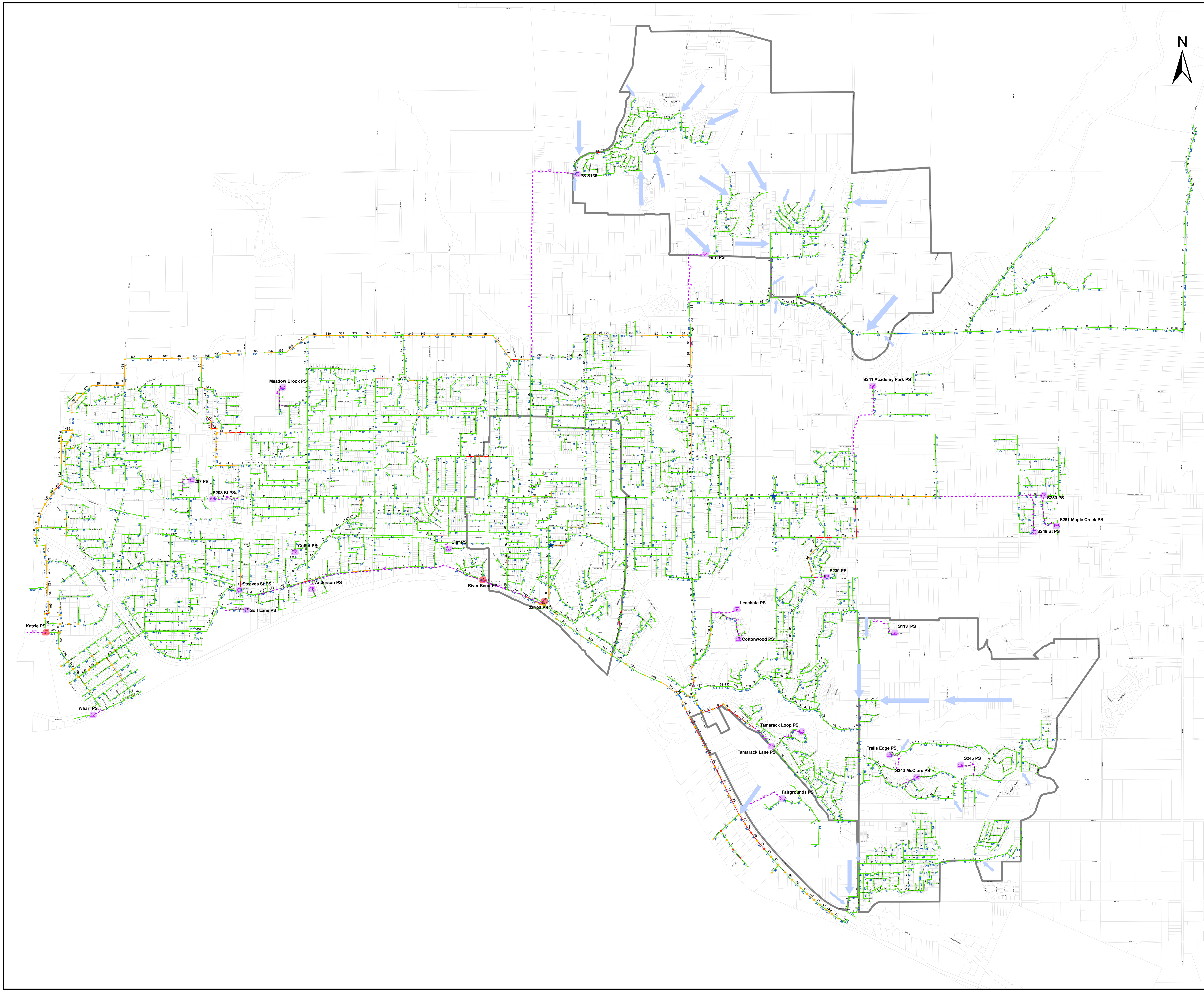
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

Current Scenario 5 yr-24hr Design Storm Results

Figure 4.1



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 — Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 — Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)
Full Flow Capacity (L/s)

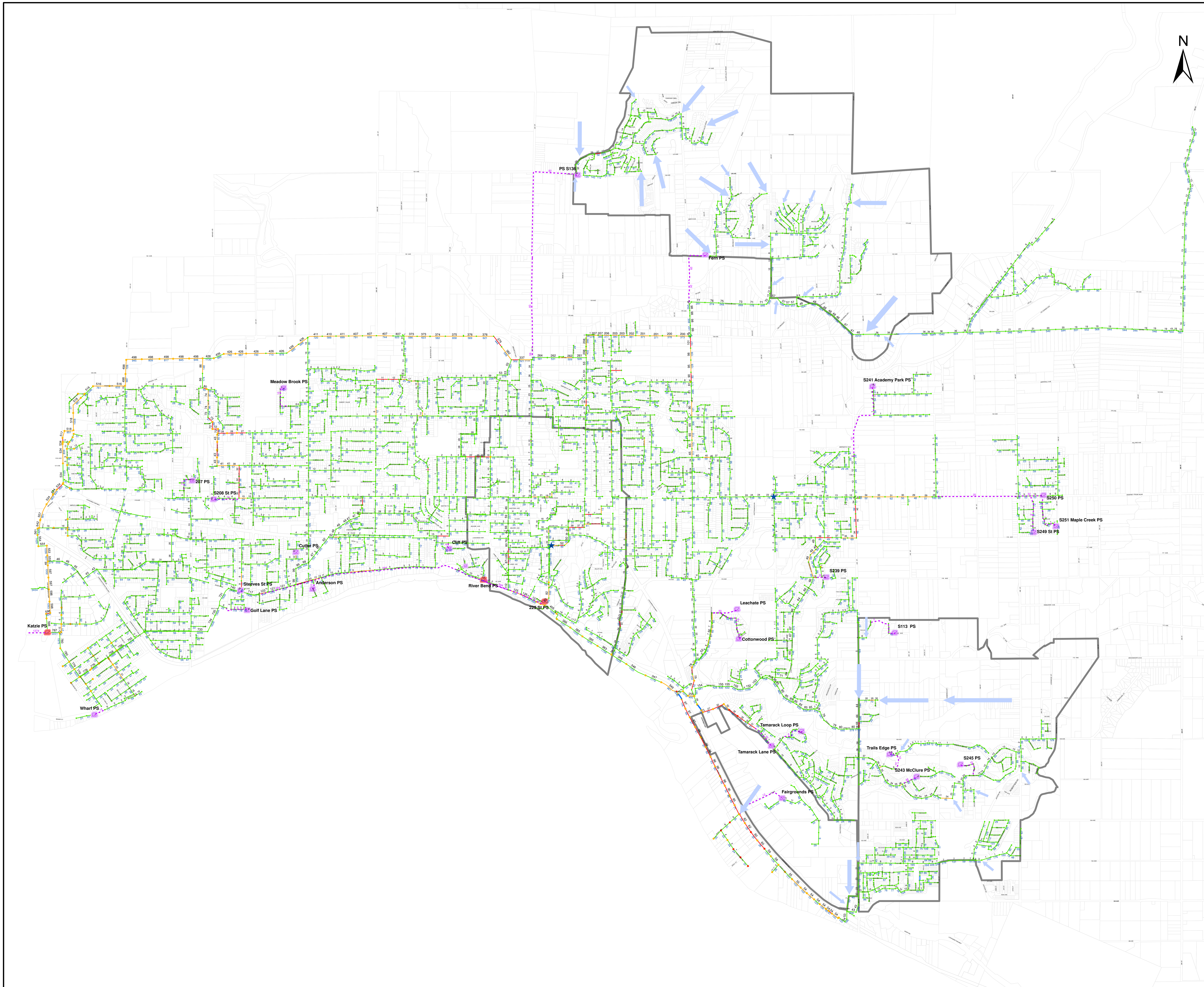
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

2018 Scenario 5 yr-24hr Design Storm Results

Figure 4.2



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 — Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 — Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)
Full Flow Capacity (L/s)

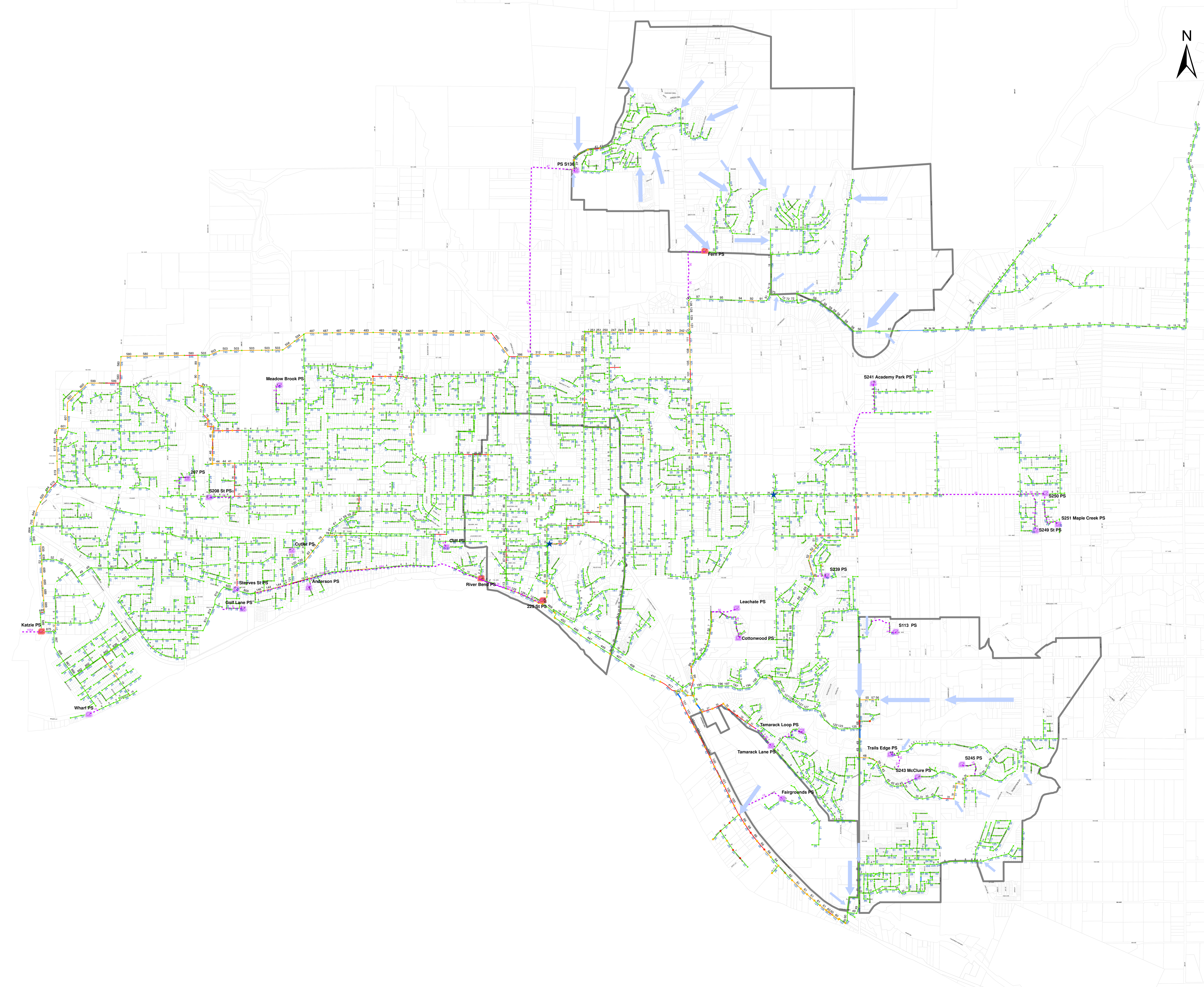
AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

**2023 Scenario
5 yr-24hr Design Storm
Results**

Figure 4.3



MAPLE RIDGE
British Columbia

Maple Ridge Sanitary Master Plan

Legend

- Diversion MH
- Siphon
- Forcemain
- Pump Station
- Undersized Pump Station
- Neighbourhood Boundary

Manhole HGL Depth

- Below Pipe Crown
- Surcharged
- Flooded

Sewer Capacity

- q/Q < 0.50
- Q>40 L/s; 0.50 <q/Q< 0.835 — Q<40 L/s; 0.50 <q/Q<0.70
- Q>40 L/s; q/Q > 0.835 — Q<40 L/s; q/Q>0.70

Peak Wet Weather Flow Loaded to MH

Peak Flow (L/s)
Full Flow Capacity (L/s)

AECOM

0 500 1000 2000m

Project No.	Date
60285153	July 2014

OCP 2041 Scenario 5 yr-24hr Design Storm Results

Figure 4.4

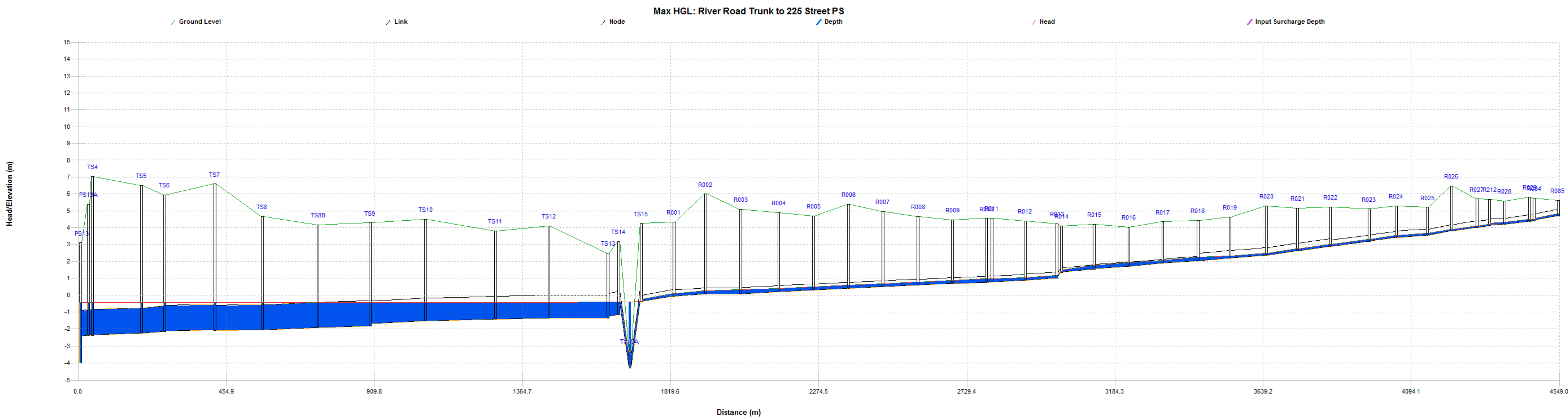
Appendix D

Max HGL Profile of Trunk Sewers

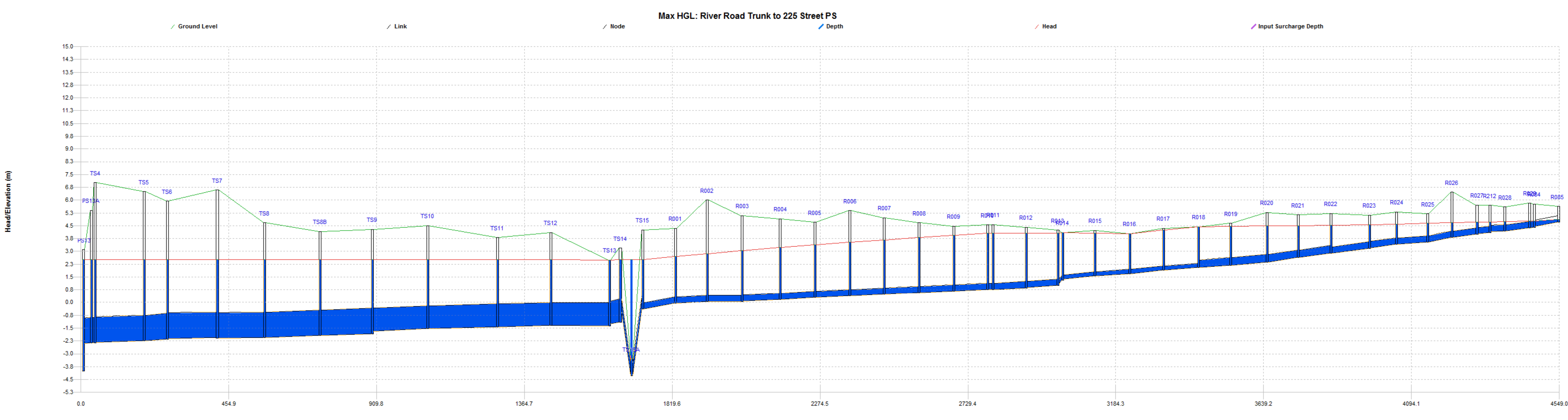
Max HGL – Trunk Sewer Profiles

River Road Trunk Sewer

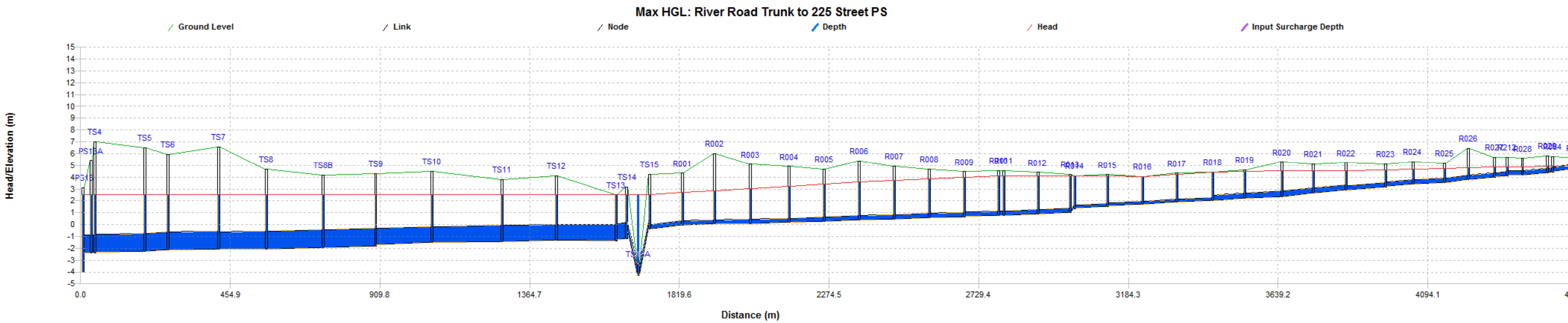
Current Scenario



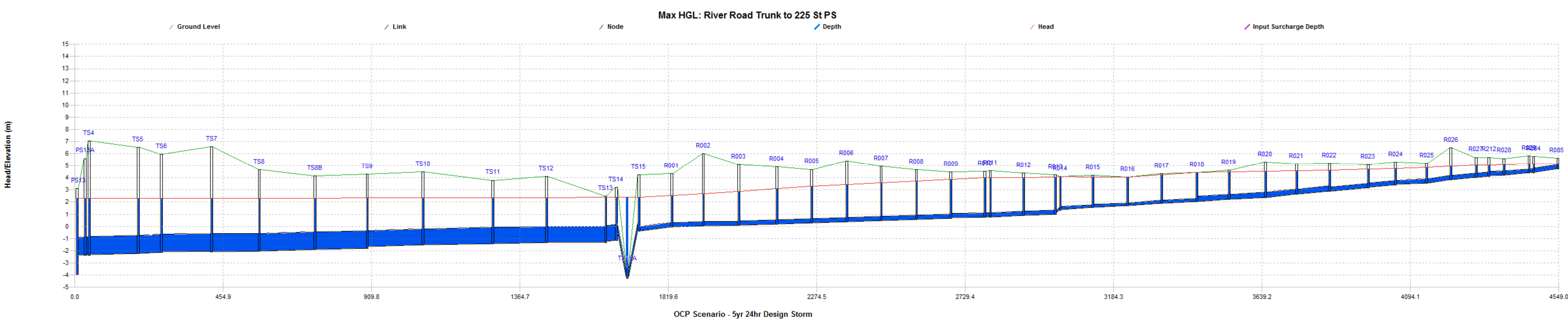
2018 Scenario



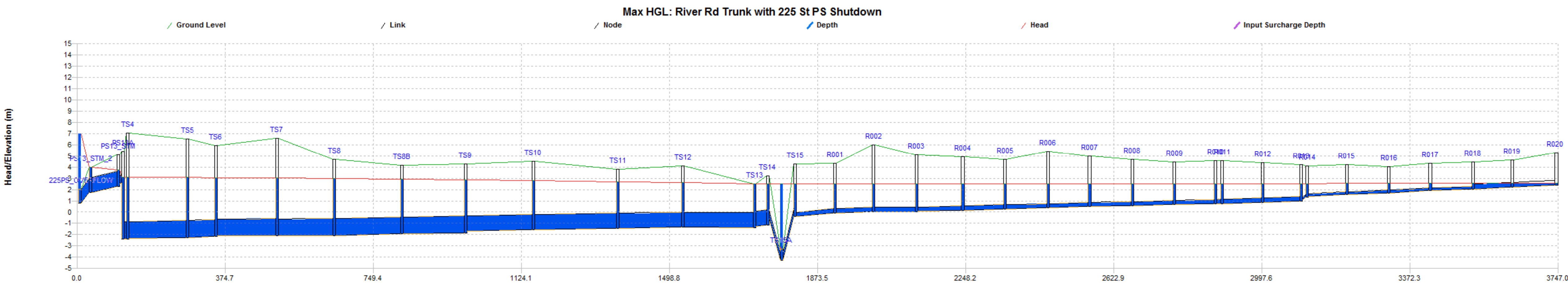
2023 Scenario



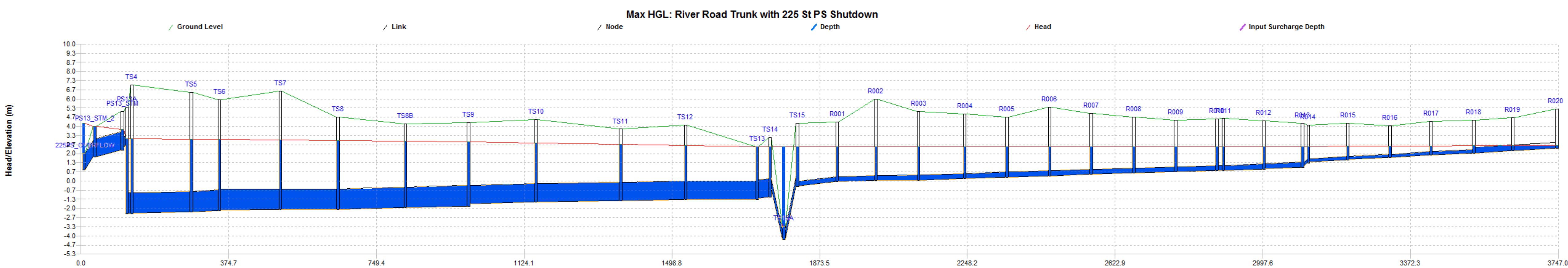
OCP Scenario



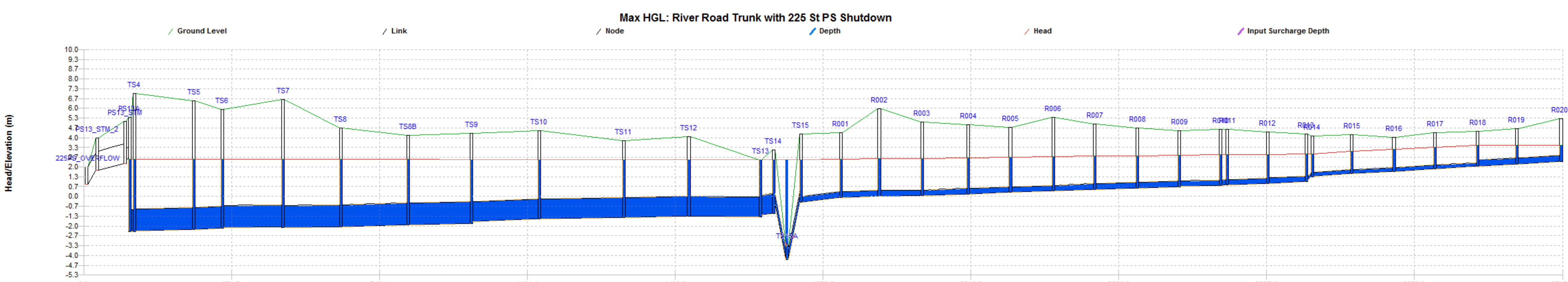
River Road with 225 St PS Shutdown



DWF - Fraser River Stage = 6.97m



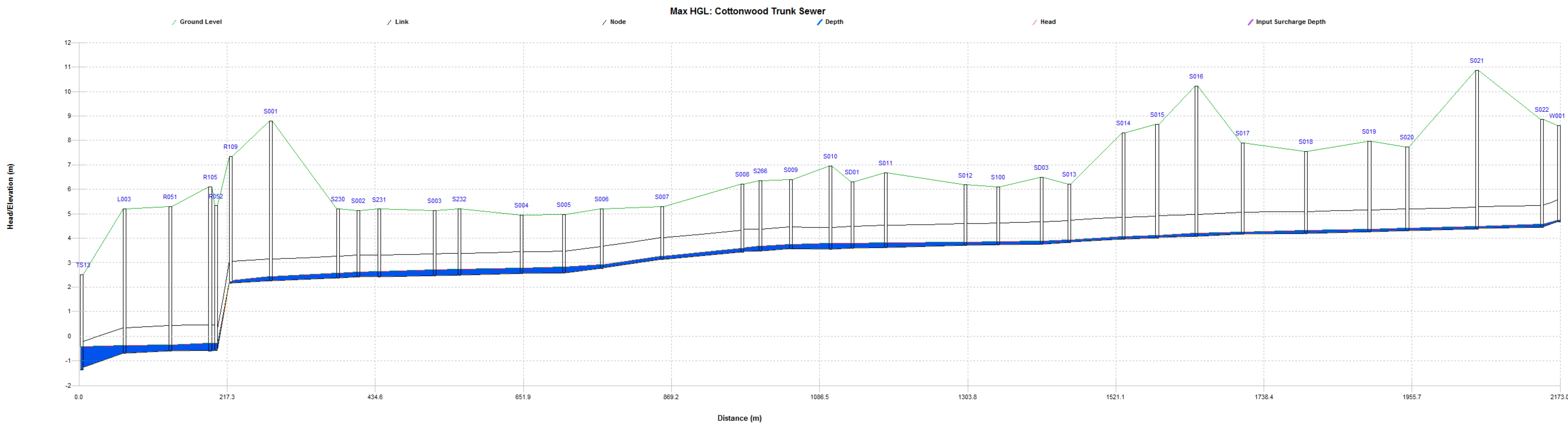
Fraser River Stage = 4.25m



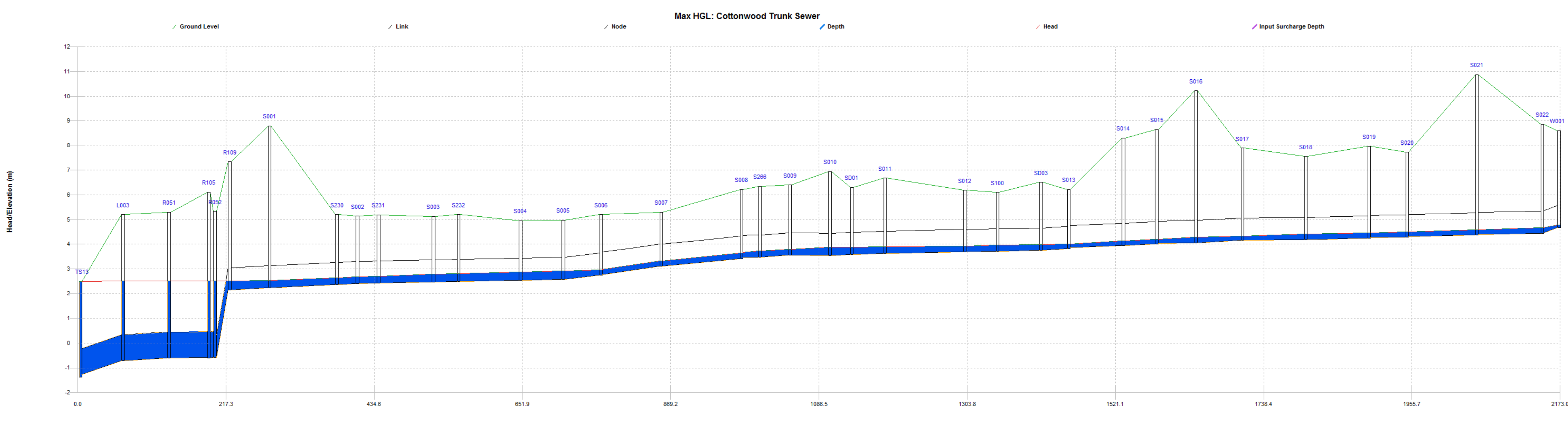
5yr-24hr Storm - Free Flow Outfall

Cottonwood Trunk Sewer

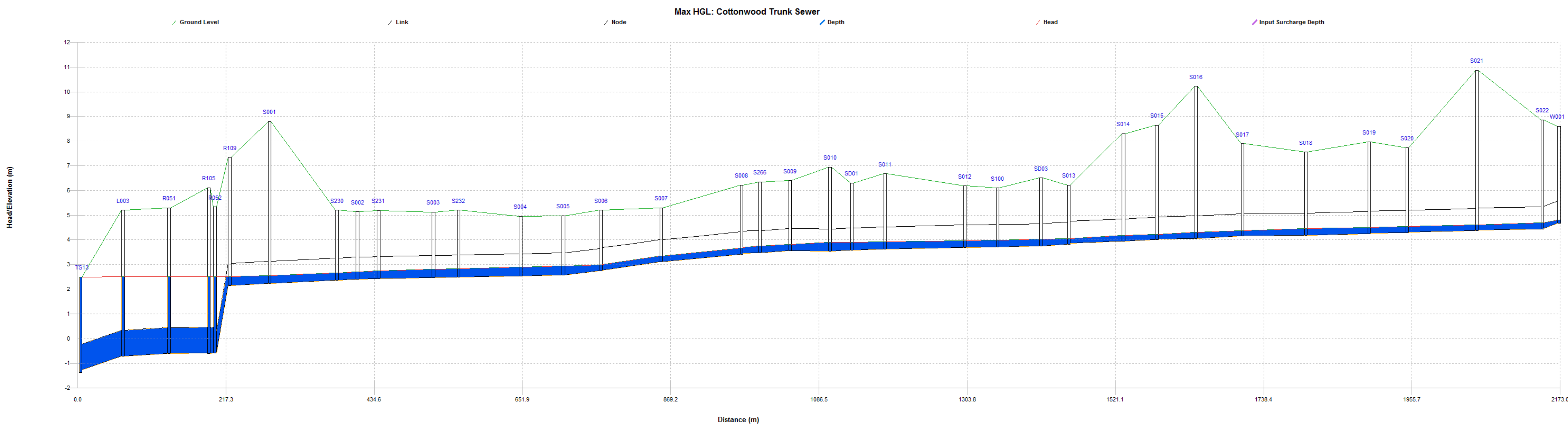
Current Scenario



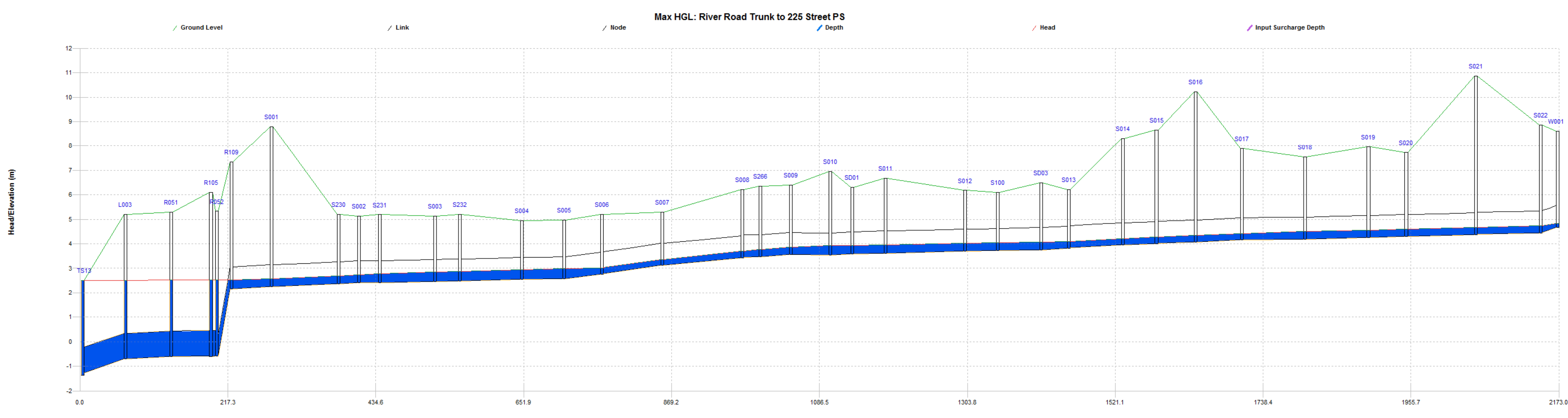
2018 Scenario



2023 Scenario

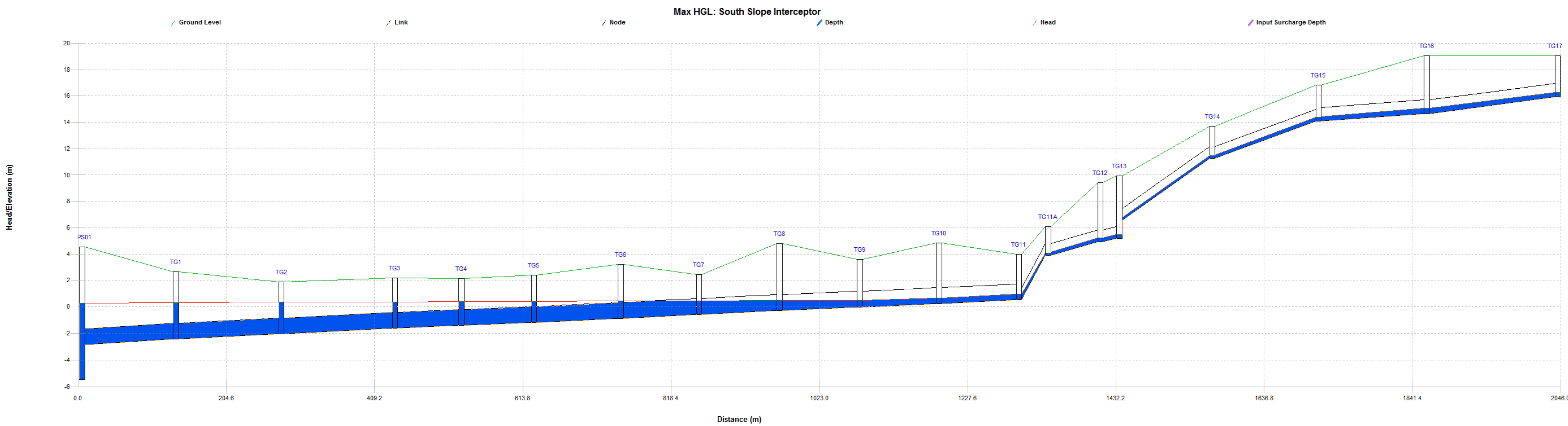


OCP Scenario

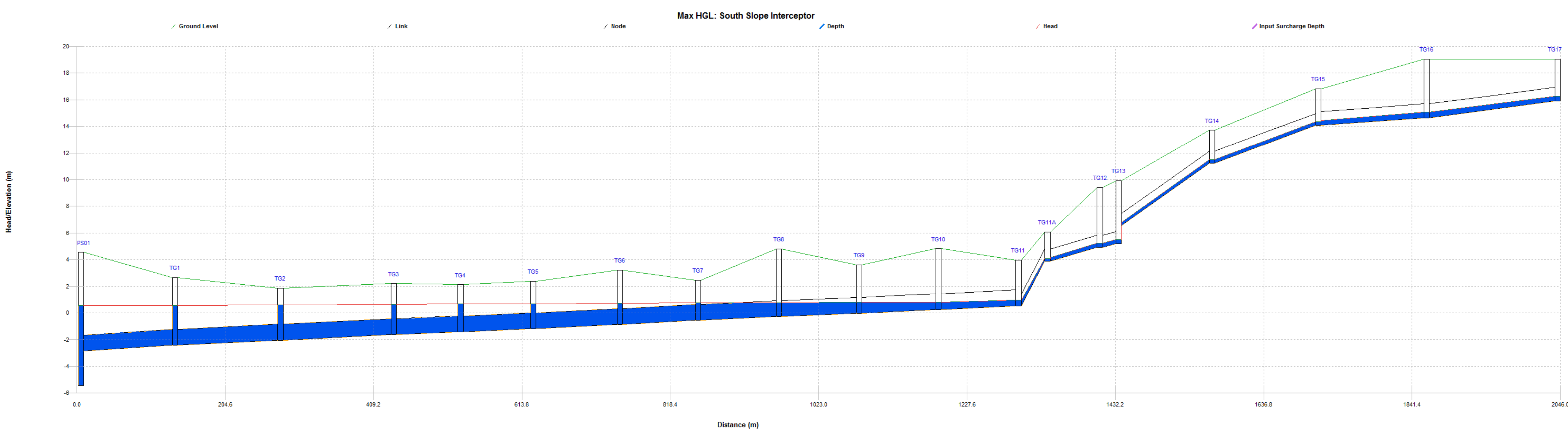


South Slope Interceptor

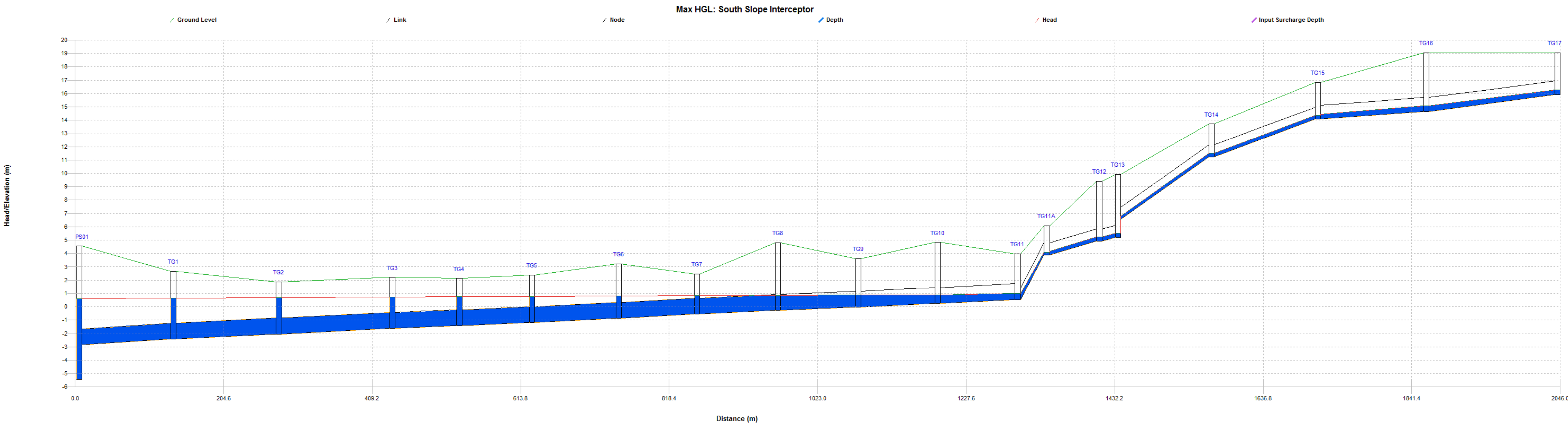
Current Scenario



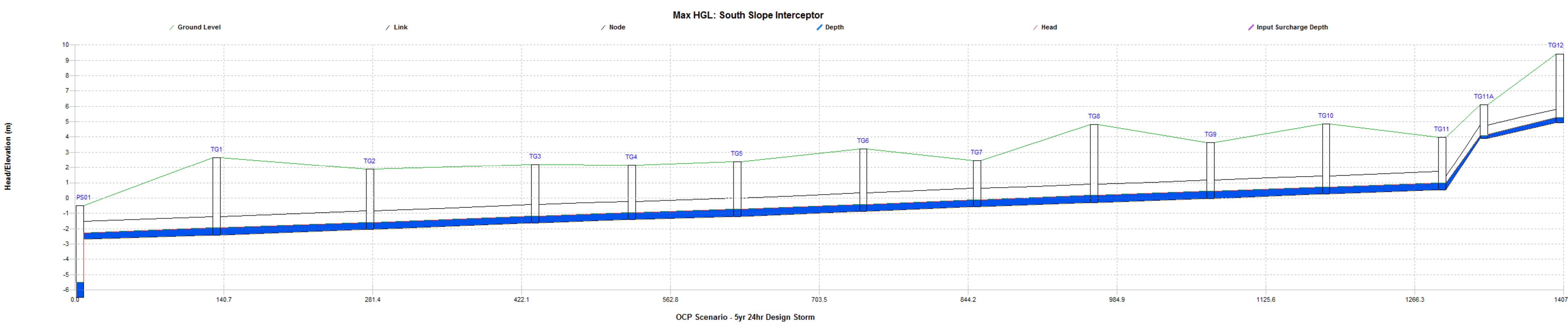
2018 Scenario



2023 Scenario

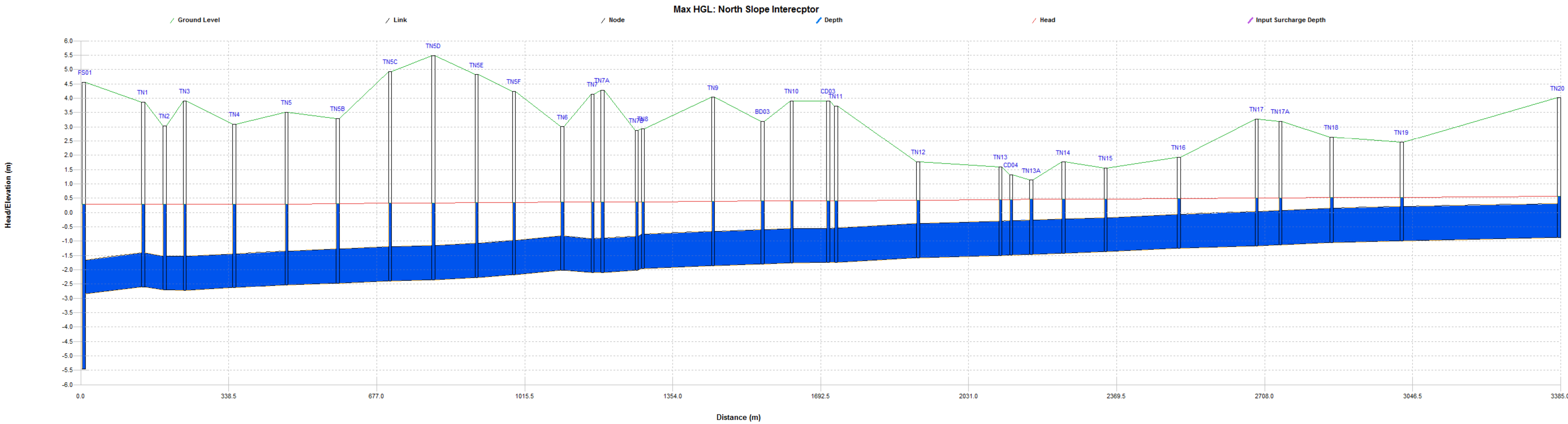


OCP Scenario

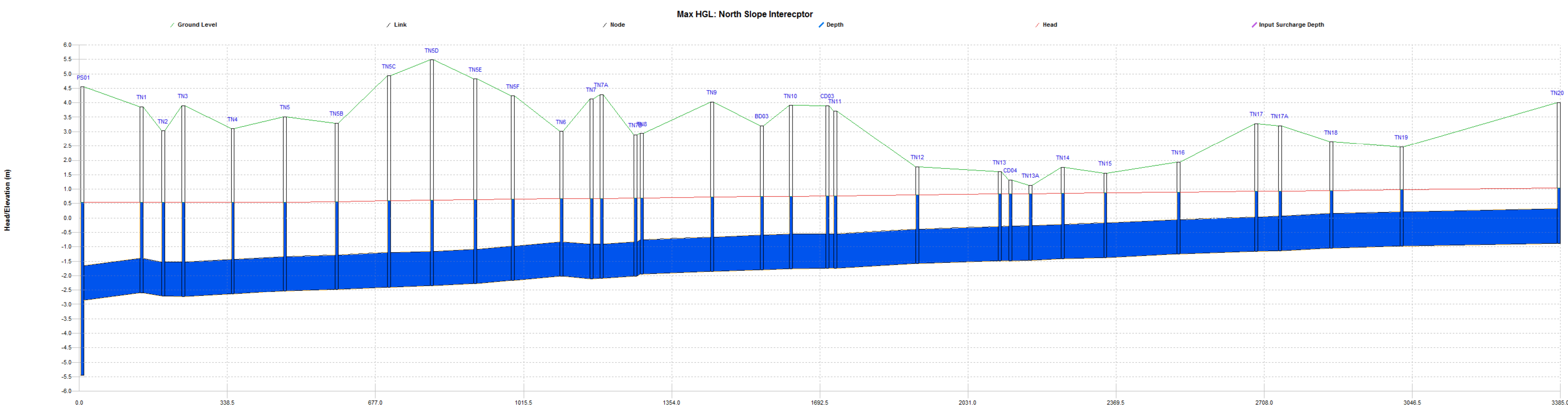


North Slope Interecptor

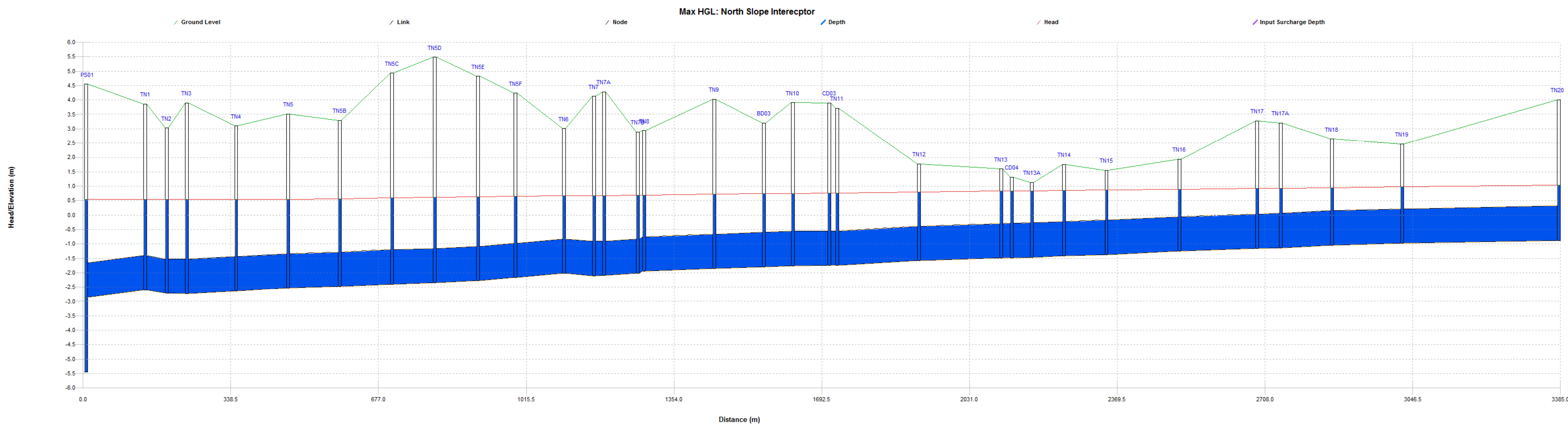
Current Scenario



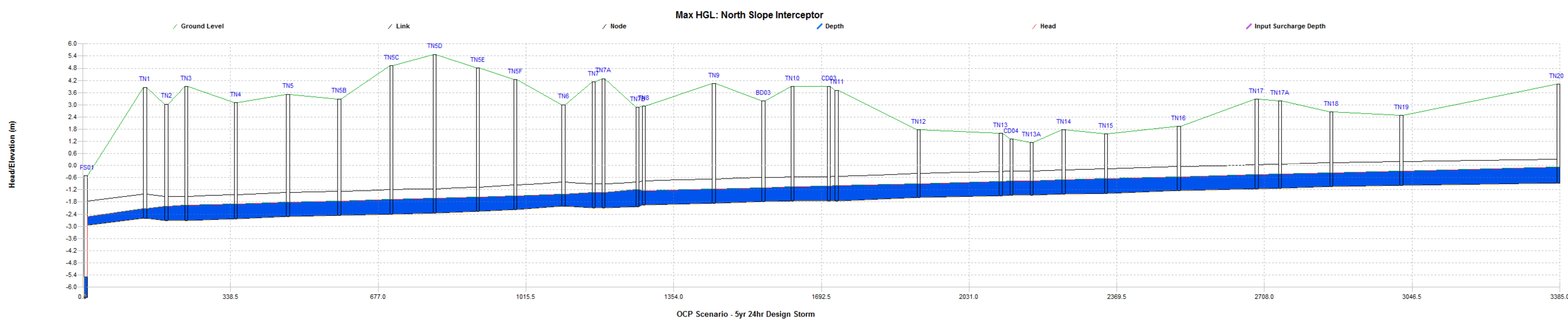
2018 Scenario



2023 Scenario



OCP Scenario



Appendix E

Tabular Summary of Proposed Pipe Upgrades

Appendix E - Table E.1 - Recommended Pipe Upgrades

Pipe ID	From MH	To MH	Type	Length (m)	Existing Capacity (L/s)	Existing PWWF Flow (L/s)	2018 PWWF (L/s)	2023 PWWF (L/s)	OCP 2041 PWWF (L/s)	Existing Diameter (mm)	Proposed Diameter for OCP Flow (mm)	Scenario When Capacity Upgrade is Required	Project Name	Cost	%DCC Eligible	Priority
R018R017	R018	R017	Trunk	109.0	26	26	48	60	66	250	450	Current	Current-Trunk: River Road Interceptor	$ 370,000	0%	1
R017R016	R017	R016	Trunk	104.9	30	26	48	60	66	250	450	Current				
R016R015	R016	R015	Trunk	107.3	25	27	50	62	68	250	450	Current				
R015R014	R015	R014	Trunk	100.8	28	27	50	62	69	250	450	Current				
D016D164	D016	D164	Trunk	67.6	44	38	42	43	46	300	375	Current	Current-Trunk: 208 St	$ 367,000	6%	2
D164D015	D164	D015	Trunk	9.5	54	38	42	43	46	300	375	2041 (OCP)				
D015D014	D015	D014	Trunk	76.2	40	38	42	43	46	300	375	Current				
D013D012	D013	D012	Trunk	16.6	47	38	43	43	47	300	375	2018				
D012D011	D012	D011	Trunk	42.4	58	66	71	72	75	300	375	Current				
D011D010	D011	D010	Trunk	55.3	55	66	72	72	75	300	375	Current				
D010D009	D010	D009	Trunk	70.1	48	66	72	72	76	300	450	Current				
D009D008	D009	D008	Trunk	65.8	61	67	72	73	76	300	375	Current				
D004D003	D004	D003	Trunk	41.5	45	74	80	80	84	300	450	Current				
R001TS15	R001	TS15	Trunk	101.2	116	39	79	97	101	375	675	2023	2018-Trunk: River Road Interceptor	$ 1,353,000	92%	3
R002R001	R002	R001	Trunk	97.6	66	39	79	97	101	375	675	2018				
R003R002	R003	R002	Trunk	109.2	27	39	79	97	101	375	675	Current				
R004R003	R004	R003	Trunk	118.7	61	39	80	98	101	375	525	2018				
R005R004	R005	R004	Trunk	107.5	67	39	80	98	102	375	525	2018				
R006R005	R006	R005	Trunk	109.1	62	39	80	98	102	375	525	2018				
R007R006	R007	R006	Trunk	106.4	63	37	78	96	100	375	525	2018				
R008R007	R008	R007	Trunk	108.7	60	37	78	96	101	375	525	2018				
R009R008	R009	R008	Trunk	107.0	62	36	78	96	101	375	525	2018				
R010R009	R010	R009	Trunk	105.0	52	36	77	95	100	375	525	2018				
R011R010	R011	R010	Trunk	10.7	89	34	76	94	99	375	450	2018				
R012R011	R012	R011	Trunk	102.7	65	34	76	95	100	375	525	2018				
R013R012	R013	R012	Trunk	97.9	71	34	75	95	100	375	525	2018				
R028R212	R028	R212	Trunk	42.6	61	19	42	54	60	375	450	2023				
TN51ATN51	TN51A	TN51	Trunk	78.9	302	236	313	334	387	900	1200	2018	2018-Trunk: NSI	$ 782,000	87%	4
CD03TN10	CD03	TN10	Trunk	81.9	494	372	502	534	619	1200	1500	2018				
TN11TN10	TN11	CD03	Trunk	11.6	494	372	502	533	619	1200	1500	2018				
TN3TN2	TN3	TN2	Trunk	41.7	604	562	562	598	689	1200	1350	Current				
TN50TN49	TN50	TN49	Trunk	134.5	444	256	351	379	440	900	1050	2023				
K014K013	K014	K013	Trunk	58.1	49	27	42	52	53	300	375	2018	2018-Trunk:Town Centre	$ 185,000	100%	5
K050K004	K050	K004	Trunk	112.5	85	68	80	85	85	300	375	2018				
K004K003	K004	K003	Trunk	58.2	91	70	82	87	87	300	375	2018				
M294M004	M294	M004	Trunk	91.3	92	75	87	90	101	525	600	2018	2018-Trunk: 232 St	$ 225,000	100%	6
M002M001	M002	M001	Trunk	112.9	107	78	98	101	111	525	600	2018				
L078L079	L078	L079	Trunk	95.5	52	51	78	79	90	300	450	Current	Current-Trunk: 232 St	$ 138,000	0%	7
L076L077	L076	L077	Trunk	66.9	62	54	78	79	90	300	375	Current				
P040P008	P040	P008	Local	24.8	9	7	10	12	12	250	375	Current	Current-Local: Town Centre	$ 64,000	0%	8
H015H014	H015	H014	Local	19.9	8	9	14	15	15	150	250	Current				
K131K118	K131	K118	Local	8.5	4	10	13	19	19	250	600	Current				
K012K147	K012	K147	Trunk	5.3	26	24	36	44	43	450	600	Current				
K272K312	K272	K312	Trunk	16.9	30	37	55	71	79	250	450	Current				
R056R055	R056	R055	Local	107.0	33	31	27	27	27	300	375	Current	Current-Local: Tamarack Lane	$ 412,000	0%	9
R055EN12	R055	EN12	Local	25.7	34	31	27	27	27	300	375	Current				
R068R043	R068	R043	Local	87.1	41	30	31	31	31	250	300	Current				
R045R044	R045	R044	Local	103.6	42	32	32	32	32	250	300	Current				
R046R045	R046	R045	Local	97.5	40	32	32	32	33	250	300	Current				
R057R056	R057	R056	Local	118.6	41	32	32	32	33	300	375	Current				
D087D086	D087	D086	Local	94.6	36	27	27	27	28	200	250	Current	Current-Local: 208 St	$ 201,000	0%	11
D086D085	D086	D085	Local	97.7	29	27	28	28	28	200	300	Current				
D085D012	D085	D012	Local	91.3	21	28	28	28	29	200	300	Current				
D023D022	D023	D022	Local	109.3	38	28	30	30	32	250	375	Current	Current-Local:209 St	$ 276,000	32%	13
D022D021	D022	D021	Local	42.2	38	27	30	31	33	250	375	Current				
D020D019	D020	D019	Local	109.8	45	27	31	32	34	300	375	2023				
D021D020	D021	D020	Local	64.2	38	28	32	32	35	250	375	Current				
D170D018	D170	D018	Trunk	15.4	25	36	41	41	44	300	450	Current				
K180K067	K180	K067	Local	16.2	3	9	12	14	14	250	675	Current	Current-Local: Selkirk Ave	$ 106,000	84%	14
K046K179	K046	K179	Local	110.0	28	14	18	25	25	250	375	2023				
S082S041	S082	S041	Local	91.5	46	33	38	38	38	250	375	Current	Current-Local: 118 Ave	$ 130,000	44%	10
S083S082	S083	S082	Local	77.3	49	32	37	37	37	250	300	2018				
H143H004	H143	H004	Local	67.3	2	22	34	38	39	300	450	Current	Current-Local: 227 St	$ 65,000	9%	12
H007H006	H007	H006	Local	7.6	45	18	28	32	32	250	300	2023				
A082A067	A082	A067	Local	23.3	15	15	16	16	16	250	375	Current	Current-Local: Princess St	$ 19,000	0%	15
F035F010	F035	F010	Local	22.0	12	9	12	14	18	150	250	Current	Current-Local: 126 Ave	$ 166,000	91%	16
F063F062	F063	F062	Local	122.2	12	7	10	11	16	150	250	2018				
F062F035	F062	F035	Local	101.5	12	7	11	12	16	150	250	2018				
J086J084	J086	J084	Local	46.8	20	16	19	20	19	200	300	Current	Current-Local: west River Rd at 212 St	$ 122,000	72%	17
J084J081	J084	J081	Local	66.6	22	14	17	17	18	200	250	2018				
J081J009	J081	J009	Local	63.4	23	14	16	17	17	200	250	2023				
G013G012	G013	G012	Local	62.8	15	7	16	19	19	150	250	2018	2018-Local: 122 Ave/ Town Centre	$ 177,000	100%	18
G012G011	G012	G011	Local	37.6	19	8	17	20	20	200	300	2018				
G011G010	G011	G010	Local	45.4	21	8	17	20	20	200	300	2018				
G010G009	G010	G009	Local	101.5	22	9	18	22	22	200	300	2018				
L057L058	L057	L058	Local	38.5	27	17	21	23	24	200	250	2018	2018-Local: Burnett St /Town Centre	$ 75,000	100%	19
L059L060	L059	L060	Local	72.1	30	17	22	23	24	200	250	2018				
S172S171	S172	S171	Local	112.3	46	31	34	34	34	250	300	2018	2018-Local: 240 St	$ 243,000	100%	20
S173S172	S173	S172	Local	128.3	46	31	35	34	35	250	300	2018				
S174S173	S174	S173	Local	94.6	46	31	35	34	35	250	300	2018				
S128S037	S128	S037	Local	30.6	26	18	19	19	20	200	250	2018	2018-Local: 116 Ave	$ 21,000	100%	21
B003B002	B003	B002	Local	82.3	12	6	9	9	11	200	250	2018	2018-Local: Steeves St	$ 56,000	100%	22
J265J264	J265	J264	Local	129.5	11	5	8	8	9	150	200	2018	2018-Local: Lougheed at 220 St	$ 84,000	100%	23
TN14TN13	TN13A	CD04	Trunk	41.9	602	359	486	517	601	1200	1350	2023	2023-Trunk: NSI	$ 105,000	100%	24
W002W037	W002	W037	Trunk	67.1	48	1	28	44	68	200	375	2023	2023-Trunk: 240 St/Kanaka Ck	$ 155,000	100%	25
W004W001	W004	W001	Trunk	98.0	37	16	32	38	260	200	525	2023				
K265K287	K265	K287	Trunk	86.1	147	101	119	126	129	450	525	2023	2023-Trunk: River Rd, 220 St	$ 88,000	100%	26
X008X180	X008	X180	Trunk	68.4	32	10	23	27	41	200	250	2023	2023-Local: Silver Valley	$ 47,000	100%	27
J326J321	J326	J321	Local	71.9	20	8	13	15	21	200	300	2023	2023-Local: Ashbury Crt	$ 69,000	100%	28
J321J322	J321	J322	Local	8.9	21	8	13	15	21	200	300	2023				
J027J326	J027	J326	Local	14.5	27	8	12	14	21	200	250	2041 (OCP)				
K023K022	K023	K022	Local	84.2	12	5	8	8	10	150	200	2023	2023-Local: Town Centre	$ 186,000	100%	29
K276K262	K276	K251	Local	5.4	30	14	18	25	25	250	375	2023				
K020K019	K020	K019	Local	85.9	51	20	28	37	38	300	375	2023				
K019K018	K019	K018	Local	72.0	50	21	30	39	40	300	375	2023				
TN27TN26	TN27	TN26	Trunk	134.9	646	343	466	498	580	1200	1350	2041 (OCP)	OCP-Trunk: NSI	$ 1,592,000	100%	30
TN21TN20	TN21	TN20	Trunk	122.4	687	344	466	498	580	1200	1350	2041 (OCP)				
TN20TN19	TN20	TN19	Trunk	379.4	633	357	484	516	599	1200	1350	2041 (OCP)				
TS13TS12	TS13	TS12	Trunk	188.0	461	271	310	349	417	1350	1500	2041 (OCP)	OCP-Trunk: River Rd Interceptor	$ 527,000	100%	31
J025J024	J025	J024	Trunk	20.5	62	29	42	46	59	400	450	2041 (OCP)	OCP-Trunk: Donovan Ave/Asbury Ct	$ 70,000	100%	32
J364J363	J364	J363	Trunk	20.8	64	29	43	46	59	375	450	2041 (OCP)				
J365J364	J365	J364	Trunk	9.0	69	29	43	46	59	375	450	2041 (OCP)				
J322J323	J322	J323	Local	29.0	24	8	13	15	22	200	250	2041 (OCP)				
J323J324	J323	J324	Local	8.9	26	9	14	15	22	200	250	2041 (OCP)				
EN05F013	EN05	F013	Local	45.5	24	13	16	16	18	200	250	2041 (OCP)	OCP-Local: 126 Ave	$ 100,000	100%	33
F013F012	F013	F012	Local	45.5	24	13	16	17	18	200	250	2041 (OCP)				
F011F010	F011	F010	Local	56.4	25	13	16	17	18	200	250	2041 (OCP)				
D024D023	D024	D023	Local	86.6	24	13	15	15	17	200	250	2041 (OCP)	OCP-Local: 209 St	$ 59,000	100%	34
W061W055	W061	W055	Local	120.8	45	7	18	24	33	250	300	2041 (OCP)	OCP-Local: Albion	$ 88,000	100%	35
A061A060	A061	A060	Local	72.5	32	21	22	23	23	250	300	2041 (OCP)	OCP-Local: Princess St	$ 53,000	100%	36

Appendix E - Table E.2 - Pump Station Upgrades - Class 'D' Cost Estimate

Project Location	Scenario When Capacity is Reached	Item Description	Unit	Quantity	Unit Price	Amount
River Bend Pump Station	Current	Mobilization and Demobilization	Lump Sum	1	$10,000	$10,000
		Traffic regulation	Lump Sum	1	$10,000	$10,000
		Supply and Installation of 150mm Diameter Forcemain	Lin m	270	$600	$162,000
		Supply and Install Upgraded Pumps	ea	2	$10,000	$20,000
					Subtotal	$202,000
					Engineering (12%)	$24,240
					Contingency (25%)	$50,500
					Total	**$280,000**
					Funding Type	**0% by DCC's**
225 St. Pump Station Forcemain & d/s Gravity Sewer on River Rd.	2018	Mobilization and Demobilization	Lump Sum	1	$30,000	$30,000
		Traffic regulation	Lump Sum	1	$30,000	$30,000
		Supply and Installation of 750mm Diameter Forcemain	Lin m	1056	$1,500	$1,584,000
		Supply and Installation of 750mm Diameter Sanitary Sewer	Lin m	2573	$1,310	$3,370,630
		Supply and Installation of1050mm Diameter Manholes	ea	4	$4,000	$16,000
					Subtotal	$5,030,630
					Engineering (12%)	$603,676
					Contingency (25%)	$1,257,658
					Total	**$6,892,000**
					Funding Type	**100% by DCC's**
Fern Crescent Pump Station	OCP	Mobilization and Demobilization	Lump Sum	1	$8,000	$8,000
		Supply and Install Upgraded Pumps	ea	2	$15,000	$30,000
					Subtotal	$38,000
					Engineering (12%)	$4,560
					Contingency (25%)	$9,500
					Total	**$53,000**
					Funding Type	**100% by DCC's**

Appendix E - Table E.3 Unit Costs

Item / Description	Unit	Unit Cost
200mm sewer (c/w backfill & asphalt restoration)	m	$645
250mm sewer (c/w backfill & asphalt restoration)	m	$675
300mm sewer (c/w backfill & asphalt restoration)	m	$725
375mm sewer (c/w backfill & asphalt restoration)	m	$805
450mm sewer (c/w backfill & asphalt restoration)	m	$875
525mm sewer (c/w backfill & asphalt restoration)	m	$1,020
675mm sewer (c/w backfill & asphalt restoration)	m	$1,050
600mm sewer (c/w backfill & asphalt restoration)	m	$1,100
750mm sewer (c/w backfill & asphalt restoration)	m	$1,310
900mm sewer (c/w backfill & asphalt restoration)	m	$1,630
1050mm sewer (c/w backfill & asphalt restoration)	m	$1,800
1200mm sewer (c/w backfill & asphalt restoration)	m	$2,200
1350mm sewer (c/w backfill & asphalt restoration)	m	$2,500
1500mm sewer (c/w backfill & asphalt restoration)	m	$2,800
1800mm sewer (c/w backfill & asphalt restoration)	m	$3,200