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City of White Rock Sewer Master Plan Update

ISL Engineering and Land Services

Final Report

August 2019

Corporate Authorization

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Executive Summary

The City of White Rock retained ISL Engineering and Land Services to update the current Sewer Master Plan and the sanitary sewer model. The City's Sewer Master Plan was originally completed in 2005 by KWL and was updated in 2010 by KWL and 2013 by AECOM. Since the last update, there has been developments in the City with associated new sanitary infrastructure. The sanitary model needed to be updated to incorporate the new infrastructure constructed since 2013 in order to reassess the City's sanitary system capacity to determine if upgrades are required. The model was assessed under both existing and future growth conditions projected in the 2017 Official Community Plan (OCP).

Currently, the City of White Rock has a population of 19,952 people. According to the OCP, the City's population is expected to reach to between 23,900 and 27,300 people by 2045. Most of the development is expected to occur near Town Centre and the surrounding areas (Town Centre Transition and Lower Town Centre). Increased redevelopment and infill activities are also expected in the mature neighbourhoods.

The City's existing sanitary sewer system consists of over 82.6 km of sanitary sewers including 3.1 km of forcemain and siphon. Within the City's sewer system, gravity sewers range in sizes from 100 mm to 600 mm, forcemains range from 100 mm to 150 mm, and siphons ranging from 450 mm to 525 mm. Sanitary flows from the entire system drain to the Metro Vancouver (MV) Pump Station located just west of the Oxford Street and Marine Drive intersection. The MV Pump Station pumps sanitary flows up Oxford Street to the South Surrey Interceptor located on North Bluff Road through a 600 mm diameter forcemain. There are three City owned and operated pump stations located near Marine Drive (Bergstrom, Ash and Keil). These pump stations pump sanitary flows from lower areas of the City to the MV Pump Station.

To update the existing sanitary model, GIS data of the sanitary system including pipe and manhole data were reviewed, as well as record drawings of recent developments. Flow monitoring data in the City were not available during this update and model calibration was not completed. However, flow monitoring data of the MV Pump Station were obtained to validate the model against three wet weather events and one dry weather event. Since model calibration was not completed, flow generation rates from the previous model were maintained, including residential and non-residential dry weather flow, rainfall dependent inflow and infiltration, and groundwater infiltration. The average dry weather flow under existing and future conditions are provided in Table 3.1.

The City's sanitary sewer system was evaluated for peak wet weather flow conditions under the 50-Year 2-Hour storm event under both existing and future growth conditions. The gravity sewers were assessed based on the ratio of peak flow to the maximum design flow and the hydraulic grade line (HGL) in the system. Forcemains and siphons were evaluated based on their maximum velocity. The assessment criteria in this master plan is that pipes exceeding 100% of their design flow capacity are at a higher risk of basement or surface flooding under the 50-Year 2-Hour storm event. These pipes should have a higher priority when staging upgrades. Pipes with peak flows ranging from 80% to 100% of their maximum design flow and with HGL below the pipe obvert have a lower upgrading priority. These pipes are recommended as optional upgrades and the upgrades can be completed with future developments.

Based on the assessment results, it was noted that the increase in population and non-residential areas under future growth conditions did not increase the peak flows in the system significantly. A significant portion of the peak flows in the system was a result of inflow and infiltration during the wet weather event. It is recommended that the City establish an I&I program to reduce the inflow and infiltration in the sanitary sewer system and prevent unnecessary wet weather related upgrades in the future. In addition, the City should continue to complete regular CCTV assessments of the system to monitor and address structural or service defects on a timely manner.

Some sewer upgrades were identified along Marine Drive, from Terry Road to the east of Nichol Road and from High Street to the west of Oxford Street. Additional upgrades were identified on Buena Vista Avenue and on Stayte Road. These upgrades were generally similar to the upgrades proposed in the previous master plan with some exceptions in the selection of individual pipe upgrades and sizes. Section 6.0 discusses the proposed upgrades in detail.

A number of condition upgrades identified during the condition assessment for Areas A to E were also included in Section 6.0. Specific repair strategies were recommended in the Condition Assessment Report for each pipe.

A major upgrade identified in the assessment was the upgrade required for the siphon along Marine Drive. Based on the simulated results, the siphon along Marine Drive and on Maple Street would cause back flows to the gravity sewers on Columbia Avenue. When the siphon was replaced in 2000, it appears that some intermediate sections of the siphon was constructed with a smaller size (450 mm compared to 525 mm). The reduction in siphon size and addition of flows from the Ash Pump Station causes flows to back up to the residential area on Columbia Avenue during a 50-Year storm event. This can be observed through a steeper HGL in the siphon as shown in Figure 5.3. It is recommended for the City to investigate the feasibility and cost associated with different upgrade options to address the backwater effects from this siphon. Upgrade options may include upsizing the 450 mm siphon to 525 mm, underground storage, and flow diversions.

As identified in the previous master plans, the MV Pump Station is currently under capacity to service the City under the 50-Year 2-Hour storm. An alternative to upgrading the MV Pump Station (if an upgrade is not planned in the near future) is to construct an underground storage to temporarily store the inflows that exceed the pump station’s capacity. Further discussion on the MV Pump Station Upgrade is presented in Section 6.3.

The City’s Bergstrom Pump Station is also under capacity based on the estimated design capacity. The pump station is causing back flows to upstream residential areas on Marine Drive and on 136 Street. The City should confirm the pump station capacity prior to designing upgrades. It is also recommended that the City confirm the capacity at the other two City pump stations to ensure the model did not overestimate design capacities.

A summary of the estimated expenditures for the proposed upgrades is provided in Table ES.1 below.

Table ES.1: Summary of Proposed Upgrades and Cost Estimate

Year	Approximate Length of Repair/Upgrades (m)	Cost Estimate
2018	234	$262,049
2019	681	$922,105
2020	1,239	$2,063,884
2021	609 (& pump station)	$2,067,151
2022*	--	--
Total	**2,963**	**$5,315,187**

*Note: Cost of optional repairs are not included in the table. Additional optional repairs can be completed in 2022.

1.0 Introduction

The City of White Rock ("the City") retained ISL Engineering and Land Services (ISL) to complete a comprehensive update to the current Sewer Master Plan and sanitary sewer model. The City's Sewer Master Plan was originally completed in 2005 and was updated in 2010 and 2013. Since the last update, there had been developments in the City and new sanitary infrastructure were constructed. The sanitary model was updated to incorporate the new infrastructure constructed since 2013 and the updated model was reassessed under future growth conditions projected in the new Official Community Plan (OCP). Upgrades were identified and prioritized based on existing conditions and future development needs to ensure that the strategic and sustainable vision presented in the OCP is fulfilled. A Capital Plan was also developed for the City as a cost effective, sustainable and practical guide for infrastructure asset decision-making process.

The study area is provided in Figure 1.1. Based on 2016 Census data, White Rock has a population of 19,952 people. The population in the city is expected to reach between 23,900 and 27,300 people by 2045 as projected in the current OCP. A comparison of the population densities between the 2016 Census data and the OCP projection is provided in Figure 1.2. White Rock has a land area of 512 Ha (2016) and is divided into 11 Land Use designations in the OCP as shown in Figure 1.3. The land use designations and their approximate parcel areas are listed as follows:

- Town Centre – 11.4 Ha
- Town Centre Transition – 22.5 Ha
- Lower Town Centre – 5.7 Ha
- Waterfront Village – 11.2 Ha
- Urban Neighbourhood – 24.0 Ha
- North Bluff East – 1.4 Ha
- Mature Neighbourhood – 231.3 Ha
- East Side Large Lot Infill Area – 3.5 Ha
- Neighbourhood Commercial – 1.0 Ha
- Institutional – 17.9 Ha
- Open Space & Recreation – 25.5 Ha

1.1 Goals & Objectives

The goal of this project is to create a plan suitable to address future upgrades and capital planning needs for the City. The plan will incorporate the City's vision, mission, and values, as outlined in the OCP, to provide a cost effective, sustainable, and practical guide for the City's Council and Administration. To achieve the objectives of this project, the following were completed:

- Reviewed existing sanitary system, including gravity, pump station and siphon system components
- Updated the sanitary model with new infrastructure constructed since the last model update
- Validated dry weather and wet weather flows in the updated model using flow data collected at the forcemain at the Metro Vancouver Pump Station
- Reviewed system performance under existing and future OCP condition
- Recommended upgrades based on system and condition assessments to meet current and future development needs
- Prioritized upgrades into a 5-Year Capital Plan

1.2 Previous Studies and Relevant Reports

Sewer Master Plan Update by AECOM, 2013

In the 2013 Sewer Master Plan Update, a new sanitary model was developed using XPSWMM. The model was calibrated with flow monitoring data, and the system was assessed under future land use conditions to determine necessary upgrades. The 2013 study focused on a review of I&I rates and AECOM recommended the City to use the 50-Year storm to assess the sanitary system rather than the 100-Year storm that was previously used. The 50-Year storm is less conservative but given the limited budget on capital projects it was determined to be more realistic and achievable for the City. The 2013 study also recommended the City to upgrade the Metro Vancouver (MV) Pump Station rather than constructing a new pump station with associated diversions that was previously proposed. The study explained that the main benefit of a new pump station is to reduce flows to the MV Pump Station while the MV Pump Station would still be under capacity under future conditions. The upgrades to the MV Pump Station would not be funded directly by the City.

A copy of this report is provided in Appendix A.

City of White Rock Official Community Plan by the City of White Rock, 2017

The Official Community Plan (OCP) is intended to protect cherished characteristics of White Rock while managing growth and shaping change to achieve the community's vision and goals. The objectives and policies established in the OCP will guide the decisions on planning and land use management. The current OCP is developed to the year 2045 and provides population projections and growth plans of the City. By 2045, the population in White Rock is expected to reach between 23,900 and 27,300 people, at an annual growth rate between 0.6% and 1.0%. This growth rate can only be accommodated through infill and redevelopment. Development will focus in Town Centre with additional developments in Town Centre Transition and Lower Town Centre areas. Growth will follow a transition principle with the greatest height and density at the intersection of Johnston and North Bluff Roads then transitioning down to the south, east, and west.

Policies in the OCP that are most directly related to the development of this Sewer Master Plan Update are Policy 12.4.1 and Policy 16.1.

City of White Rock Corporate Report – North Bluff Road Study by the City of White Rock, 2018

In this study, the Land Use and Planning Committee of the City of White Rock proposed to increase the OCP guidelines of building height and allow additional density in the study area. The study area is located adjacent to North Bluff Road, between Oxford Street and Finlay Street. The area consists of 36 properties with a total area of 16 Ha. There are two significant "blocks" within the study area: Town Centre ("Core Block") and Peach Arch Hospital ("Campus Block"). Properties in the Core Block will be redeveloped into mixed-use areas with the highest population and building densities in the City. In Campus Block, a majority of the area is identified to be part of the new hospital expansion. The study states that additional height and/or density will only be considered if developer can provide constructed, "tangible" amenities in the area. The proposed adjustments follows the OCP height and density transition principle with Town Centre having the tallest building and highest densities, and decreases towards surrounding areas (Town Centre Transition and Lower Town Centre). The study proposed an additional 0.1 to 1.0 FAR to the existing FAR proposed in the OCP.

1.3 Key Terms & Abbreviations

- **Dry Weather Flow (DWF):** sewage flows in the sanitary system generated by service users in residential, commercial and industrial areas, with additional flows from groundwater infiltration.
- **Groundwater Infiltration (GWI):** non-rainfall dependent flow that enters the sewer system through holes or cracks in the sewers and manholes, misaligned joints and service laterals.
- **Inflow and Infiltration (I&I):** The total inflow and infiltration that enters the sanitary sewer system (i.e. flows from both GWI and RDII).
- **Rain Dependent Inflow and Infiltration (RDII):** water that enters sewer systems through direct and indirect sources such as footing drain connections, leaky manholes and pipe joints due to the influence of rainfall.

1.4 Existing Sanitary Infrastructure

There are 10 major sanitary catchment areas in the City, these catchment areas are shown on Figure 1.4. An overview of the existing sanitary sewer system is shown in Figure 1.5. There are over 82.6 km of sanitary sewers including 3.1 km of forcemain and siphon in the City. A summary of the City's sanitary sewers is provided in Table 1.1. Currently, flows from the entire sanitary sewer system drains to the Metro Vancouver (MV) Pump Station located just west of the Oxford Street and Marine Drive intersection. The MV Pump Station pumps sanitary flows up Oxford Street to the South Surrey Interceptor at North Bluff Road through a 600 mm diameter forcemain.

The City owns and operates three smaller pump stations that pump sanitary flows from lower areas to the MV Pump Station. The three pump stations are located near Marine Drive, on Bergstrom Street, Ash Street, and Keil Street. Detailed information on the pump stations are presented in Table 1.2.

Table 1.1: Sanitary Sewer Overview

Size Range	Length of Pipe
100-200	64,870 m
250-300	12,023 m
375-600	3,648 m
Total (Gravity)	**80,541 m**
Forcemains	2,093 m

Table 1.2: Sanitary Pump Stations (Data from the 2013 Study)

Pump Station	Catchment Area (Ha)	Pump Type	Primary Pump On/Off Levels	Secondary Pump On/Off Levels
Bergstrom	19.4	2 x 5 Hp MT CP3102.181	64.12 m/63.72 m	64.18 m/63.72 m
Ash	8.2	2 x 15 Hp MT CP3102.181	1.436m/1.05m	1.462 m/1.05 m
Keil	8.0	2 x 20 Hp MT CP3152	-2.18m/-2.43m	-2.145 m/-2.43 m

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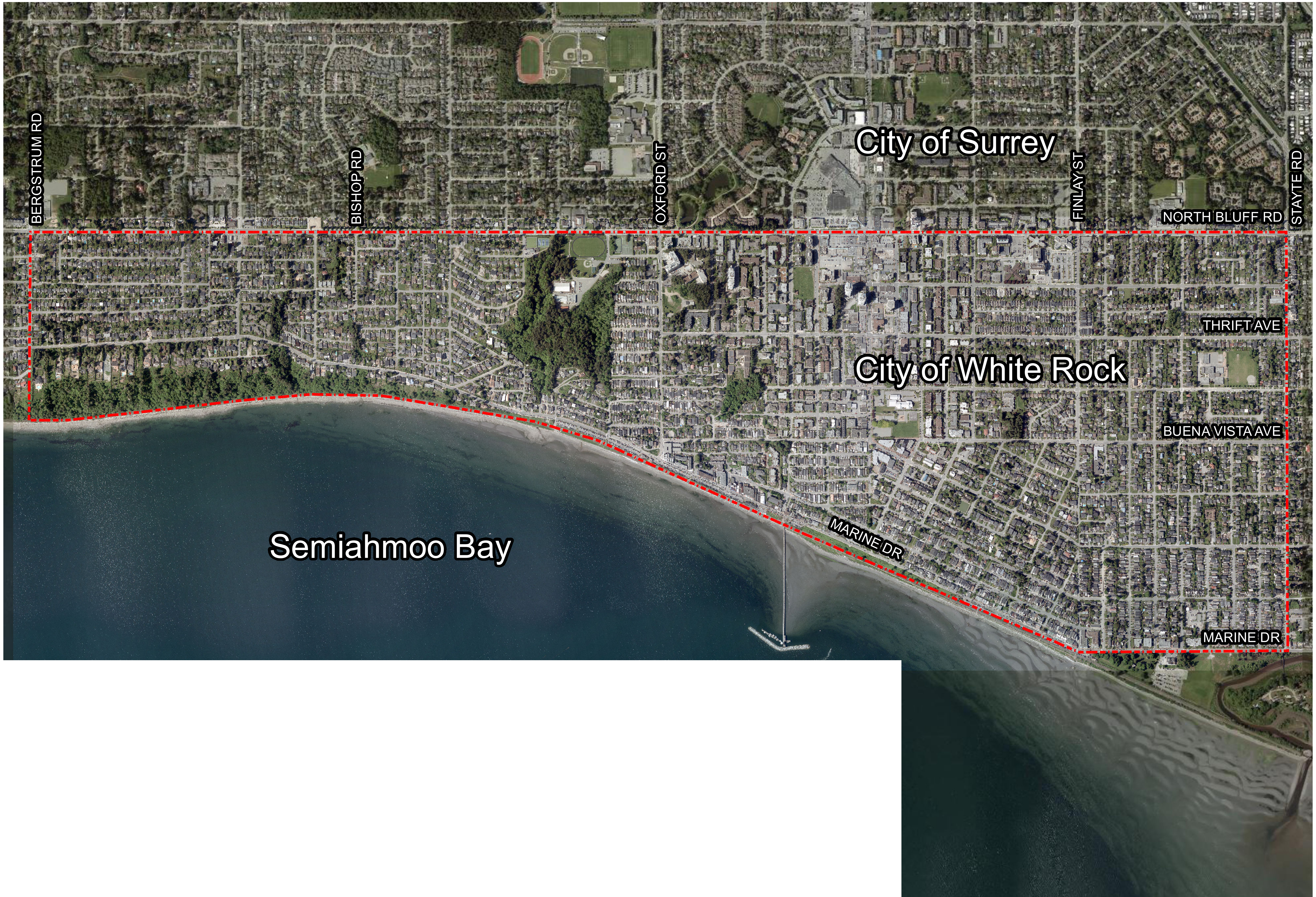
FIGURE 1.1



Legend

Study Area

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Study Area

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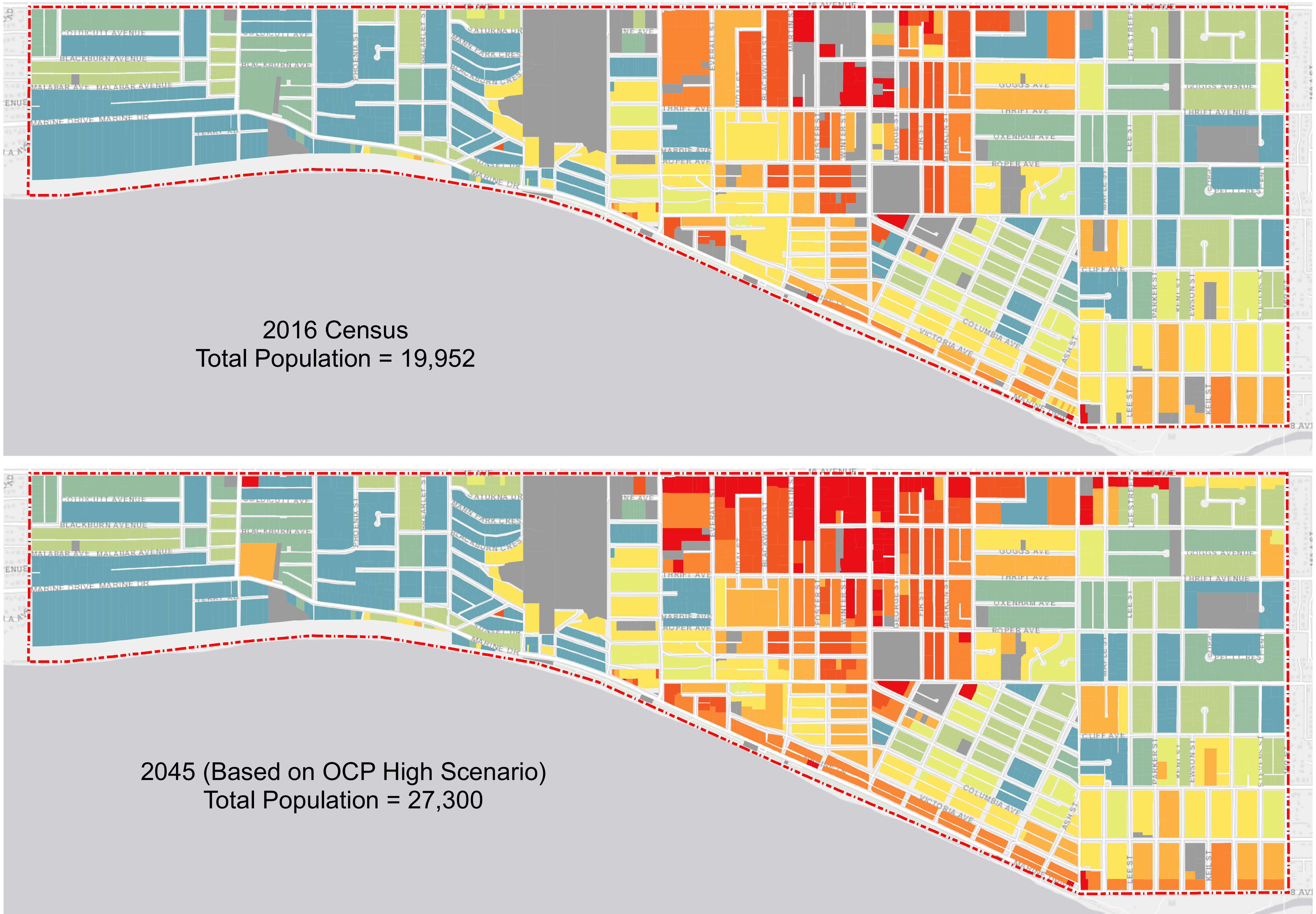
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

FIGURE 1.2



Legend

Study Area

Population Density (people/net ha)

- 0.00
- 0.01 - 35.00
- 35.01 - 40.00
- 40.01 - 50.00
- 50.01 - 65.00
- 65.01 - 85.00
- 85.01 - 110.00
- 110.01 - 150.00
- 150.01 - 250.00
- 250.01 - 700.00

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Population Densities

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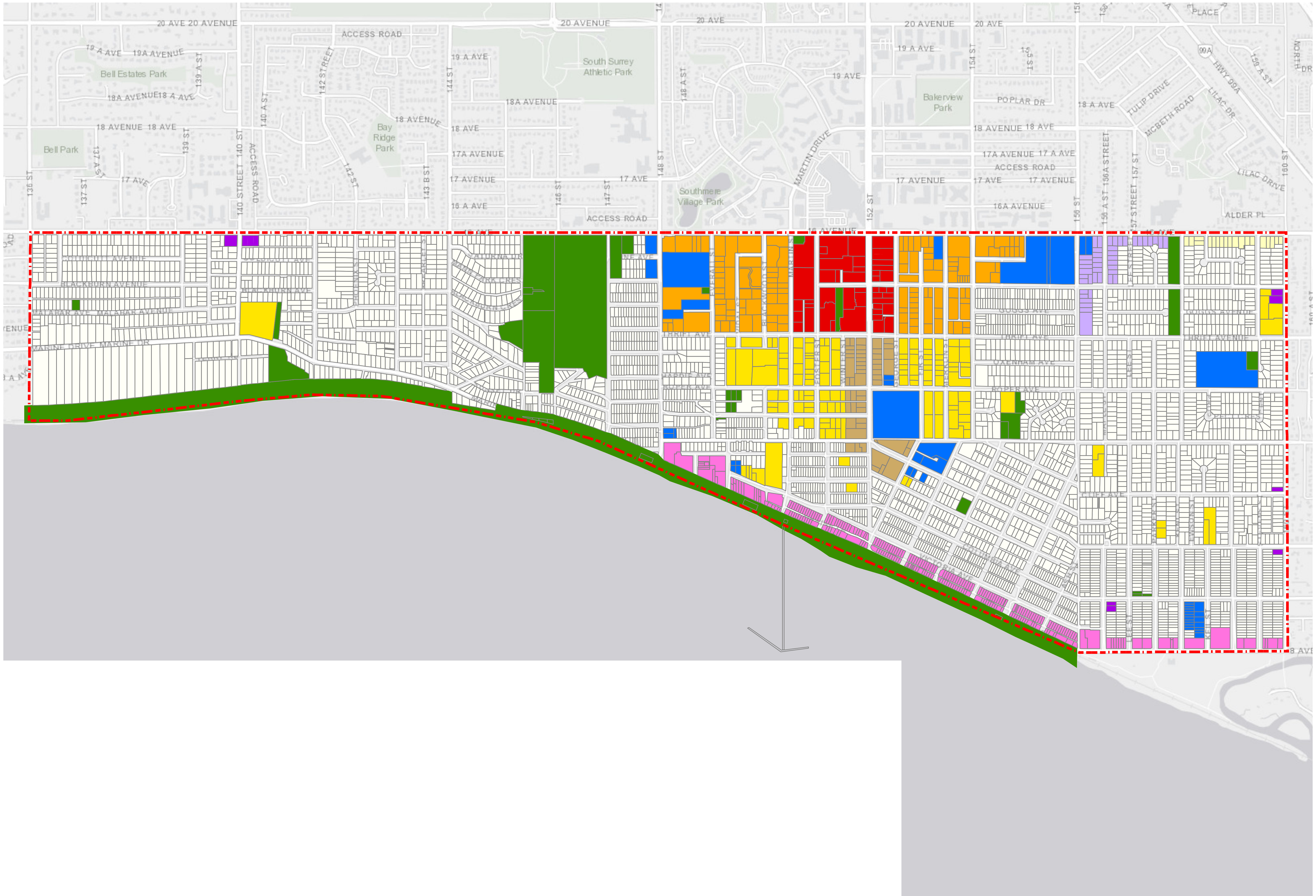
FIGURE 1.3



Legend

Study Area

New_Neighbrhd

- Town Centre
- Town Centre Transition
- Lower Town Centre
- Waterfront Village
- Urban Neighbourhood
- North Bluff East & West
- Mature Neighbourhood
- East Side Large Lot Infill Area
- Neighbourhood Commercial
- Institutional
- Open Space & Recreation

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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2017 Official Community Plan

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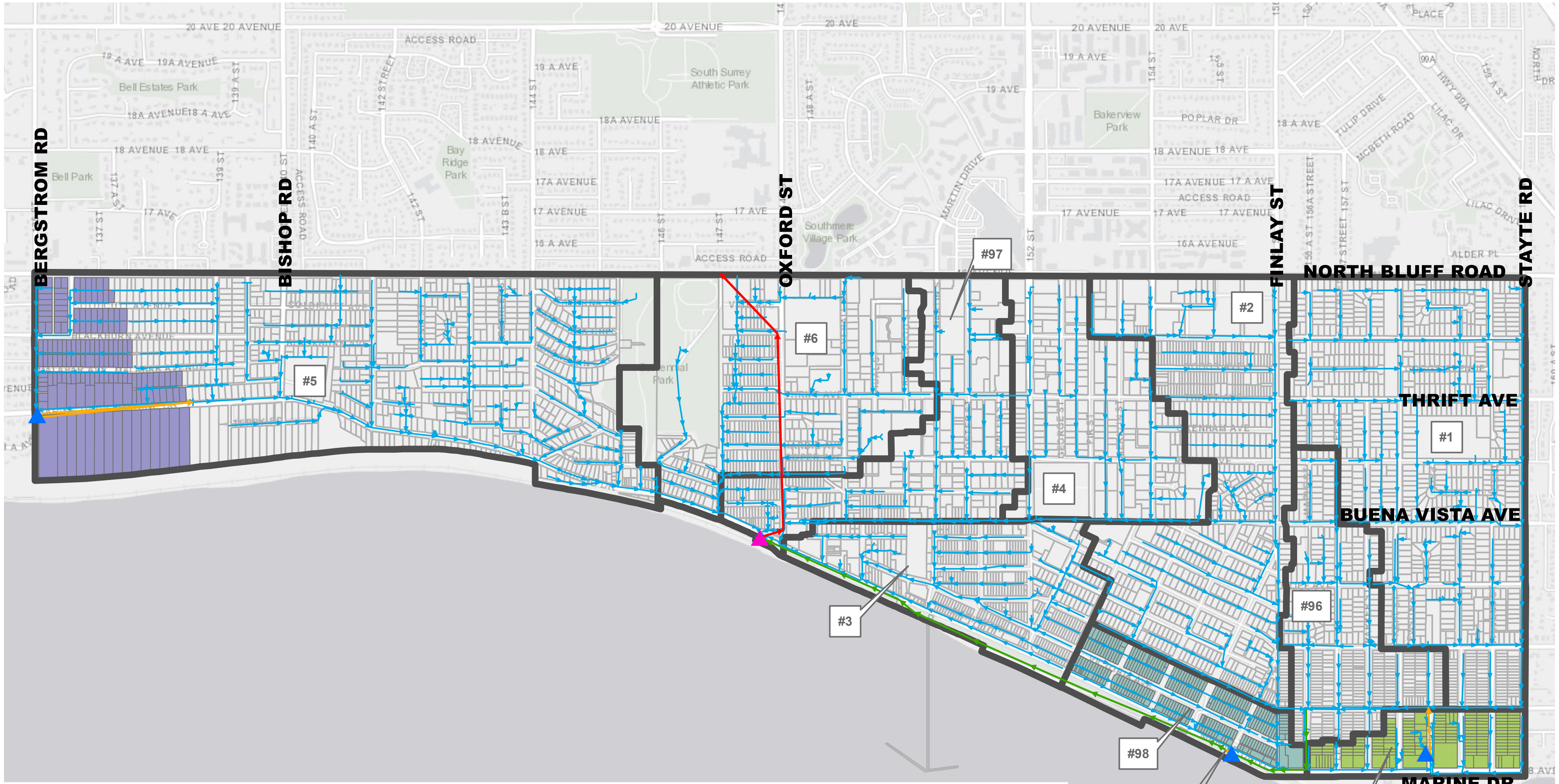
FIGURE 1.4



FORCEMAIN (NOT VISIBLE BEHIND THE PUMP STATION SYMBOL) IS CONNECTED FROM THE ASH PUMP STATION TO THE SIPHON

Catchment #	Area (Ha)
1	81.7
2	65
3	33.4
4	37.9
5	125.3
6	59.7
96	28.9
97	36.7
98	17.2
99	13.1

Legend

- MV Pump Station
- City Pump Stations
- Forcemain (Metro Van)
- Forcemain
- Siphon
- Gravity Sewers
- Major Catchments
- Catchments to Ash
- Catchments to Bergstrom PS
- Catchments to Keil PS

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Sanitary Sewer Catchment Areas

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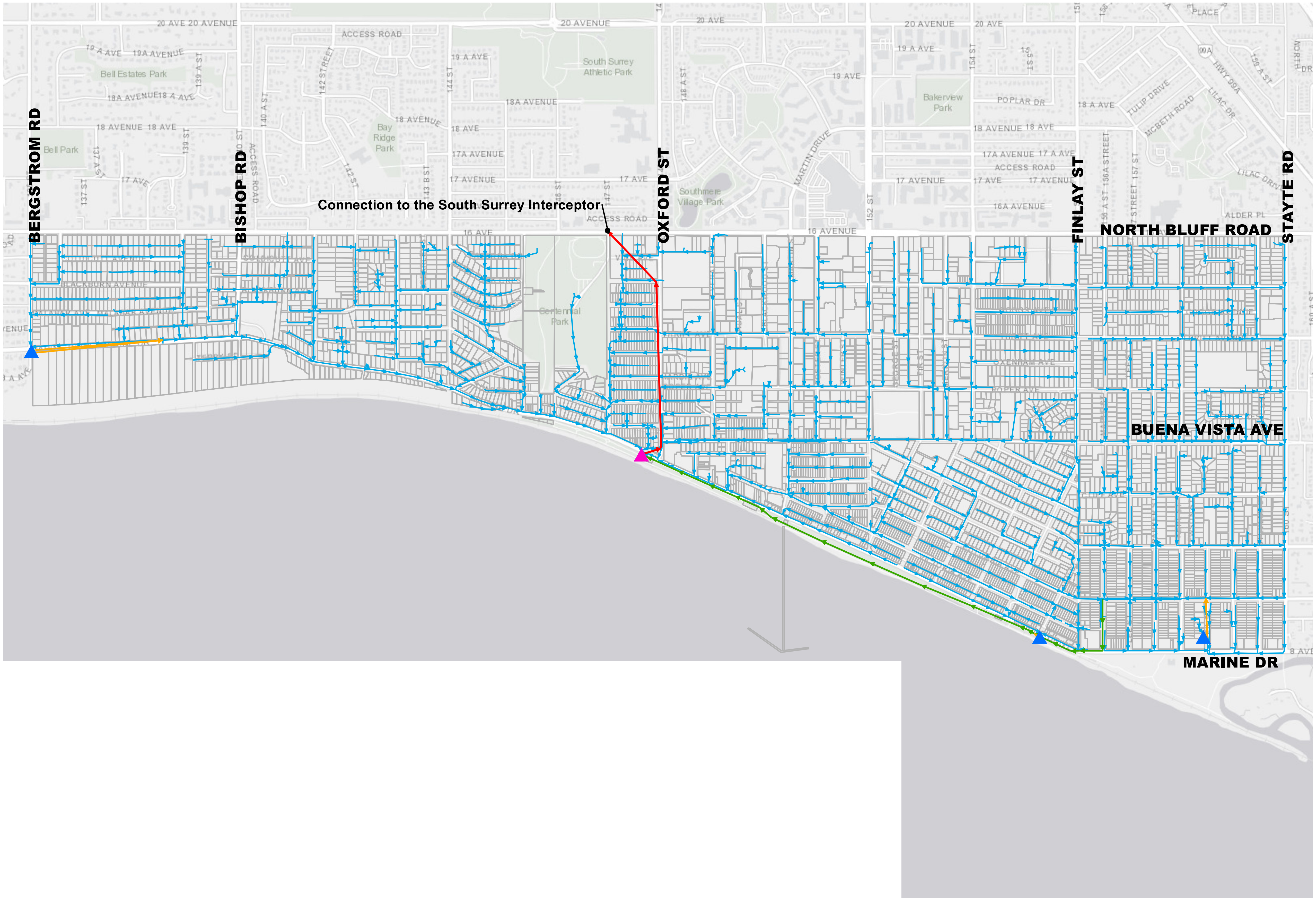
FIGURE 1.5



Legend

- MV Pump Station
- City Pump Stations
- MV Forcemain
- Forcemain
- Siphon
- Gravity Sewers

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Sewer Master Plan Update 2019

Existing Sanitary Sewer System

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2.0 Model Update

The previous sanitary model was developed by AECOM as part of the Sanitary Master Plan Update in 2013. The 2013 model was used as a basis for the development of the current model. In the previous update, the software used to develop the model was changed from HYDRA (used in the 2010 model) to XPSWMM.

The sanitary model in this update was developed using XPSWMM version 2018.1. The model was developed from the previous sanitary model, but the setup of the model was modified as part of the update as the previous model was developed with an older version of the software. This will be explained in Section 2.2 Model Development.

2.1 GIS Data Review

During the data checking process, a number of data gaps were identified in the GIS database. Typical data gaps were pipes with missing or incorrect invert elevations and sizes. Since the model was to be updated from the 2013 model, only a small number of data gaps needed to be corrected. Some data gaps were verified and corrected using available record drawings. Others that could not be verified were corrected with assumptions based on surrounding conditions (i.e. elevation or size of upstream or downstream pipes). Survey work was not within the scope of this project.

Following the data review and corrections, new infrastructure in the GIS database were added to the 2013 sanitary model. The model was reviewed for additional errors. The most common type of error in the model was connectivity error caused by incorrect pipe elevations and slopes, or disconnected pipe segments.

2.2 Model Development

New sanitary infrastructure that were constructed since the last model update were added to the current model. These upgrades were identified by comparing the most recent GIS databases of sanitary infrastructure with the previous model. Record drawings that were available were used to verify data in the GIS and identify additional upgrades that were not yet updated in the GIS database. In addition to the recently constructed infrastructure, upgrades that are currently being designed were also added to the model if design drawings were available, to reflect upgraded conditions.

The model setup was modified from the previous model. There are three types of flow data in the sanitary model: dry weather flow (DWF), groundwater infiltration (GWI), and rainfall dependent inflow & infiltration (RDII). The DWF in the model is the base sanitary flow without GWI, which is the sewage flows generated by residential, commercial, institutional, and industrial service users only. The data structure and input location of DWF and GWI in the model were changed in the update. In the previous model, GWI and DWF were both recorded as “Dry Weather Flow” under the Hydraulics and Sanitary blocks, respectively. GWI in the model is a constant flow determined based on the size of the sub-catchment. A more suitable input location for this data is under “Constant Inflow” in the Hydraulics block. The input unit of GWI data had to be manually converted from litres per second to cubic meters per second as the default input unit in the system could not be changed. To prevent the model from rounding small flows to zero and causing underestimated flows in the simulation, the converted GWIs in each node were grouped into larger values if the flows end up in the same downstream conduit. The DWF data was relocated from the Sanitary block to Hydraulics block, under “Time Series Inflow, Dry Weather Flow”. RDII data was kept under the Runoff block. Following the changes in model setup, the model now simulates simultaneously under the Hydraulics and Runoff blocks.

2.3 Flow Monitoring Data

The City does not currently have any permanent flow monitor stations set up or recent flow monitoring data. For the purpose of model validation, flow monitoring data was obtained from the MV forcemain at the MV's White Rock Pump Station flow monitoring station.

2.4 Flow Generation in the Model

Since flow monitoring data were not available, sewage flows were generated using data collected from the 2013 study and the 2016 Census data from Statistics Canada. Parcel connections from the previous model were also not available and this was completed by assigning parcels to the upstream node of the closest pipe. After automatic connections were completed, a manual check was completed to re-assign parcels where necessary so the connections made sense based on topographic condition of the area. Parcels connected to the same nodes were lumped to obtain the sub-catchment area for that node.

For residential dry weather flow, the total population was distributed to each parcel based on parcel size and land use. The diurnal curve for residential land use was determined in the 2013 study using data collected during the flow monitoring period. A weekday and a weekend pattern were both determined in the study. The weekend pattern was used for system capacity analysis since it had a higher peaking factor. An average flow generation rate of 210 L/c/day was used for residential flows as it was more representative of the actual usage in the City compared to the 360 L/c/day required for designing new systems. This value was the overall average of the data collected at MV forcemain.

For institutional and commercial flows, a separate diurnal pattern was applied. This pattern was also from the 2013 study and it was derived from previous sanitary modelling studies for cities in the Lower Mainland. The non-residential or ICI (institutional/commercial/industrial) generation rate was estimated by AECOM to be 85% of ICI water meter records, which was calculated to be 2000 L/ha/day in this study.

GWI in the model was the same as the 2013 study with the exception of lumping the data which was explained in Section 2.2 Model Development. GWI data from the 2013 study was collected during winter months which results in worse (more conservative) conditions in the simulation as groundwater table is higher in the wet season. For areas where the flow monitoring data did not cover, a unit rate of 0.06 L/s/Ha or 5,184 L/Ha/day specified in the Subdivision Bylaw 777 Schedule B was applied. The total GWI in the City was approximately 26.7 L/s.

RDII values were also kept from the previous calibration. As explained in the 2013 study, XPSWMM uses the RTK approach which combines three unit hydrographs representing the amount of inflow and infiltration that occur at various stages during and after a rainfall event. The three factors of the unit hydrographs as described in the previous study are:

- R: percentage of rainfall that enters the sanitary sewer system
- T: time from the onset of rainfall to the peak of the unit hydrograph in hours
- K: ratio of time of recession to time of peak of the unit hydrograph

In the 2013 study, the "R" values were derived from both the temporary flow monitoring sites and White Rock Pump Station data. The "R" values that were applied to different locations in the City can be found in Figure 2.6 of the 2013 Sewer Master Plan Update.

2.5 Model Validation

One dry weather event and three wet weather events were simulated to validate the model which was calibrated in the last model update in 2013. The four events were: September 2nd – 5th, 2017 (dry weather), October 4th – 12th, 2017, October 16th – 21st, 2017 and January 23rd – 31st, 2018. Peak flow rates, average flow rates, and total runoff volumes from the simulations were compared against those recorded from the flow monitoring site. The results are summarized in Table 2.1.

The simulated flow data in the September and October events are generally 30% - 40% higher than the metered data. The reason for the higher predicted flows in September and October is because the model was calibrated using data between January and March. As this is a wet season, the GWI is much higher than it would be in a drier season like September. Evidently, the predicted flows in January is significantly closer to the metered data, with differences ranging from 0% to 12%. The model was calibrated for winter conditions as a "worst case" scenario with higher wet weather flows and GWI.

Table 2.1: Model Validation Summary

Simulation Period & Rainfall Depth	Return Period		Flow Monitoring Site	Model Results	% Error
Sept 2-5, 2017 --	(Dry Weather Event)	Peak Flow (L/s)	145	154	6.2%
		Average Flow (L/s)	60	83	38.3%
		Total Event Volume (m^3)	21,869	28,755	31.5%
Oct 4-12, 2017 35.6mm	1:10 year event on Oct 11, 2017	Peak Flow (L/s)	285	368	29.1%
		Average Flow (L/s)	64	93	45.3%
		Total Event Volume (m^3)	50,148	72,560	44.7%
Oct 16-21, 2017 81.2mm	1:2 year event on Oct 18, 2017	Peak Flow (L/s)	284	360	26.8%
		Average Flow (L/s)	90	120	33.3%
		Total Event Volume (m^3)	45,289	63,142	39.4%
Jan 23-31, 2018 83.4mm	< 1:2 year event on Jan 28, 2017	Peak Flow (L/s)	290	255	12.1%
		Average Flow (L/s)	110	110	0.0%
		Total Event Volume (m^3)	87,776	85,732	2.3%

Return periods of the simulated events were estimated based on Metro Vancouver's All Duration IDF curve with data collected between 1994 and 2014. The return periods were determined using available tools on PCSWMM 2017. Figure 2.1 shows the IDF curves of the three modelled wet weather events generated from PCSWMM 2017.

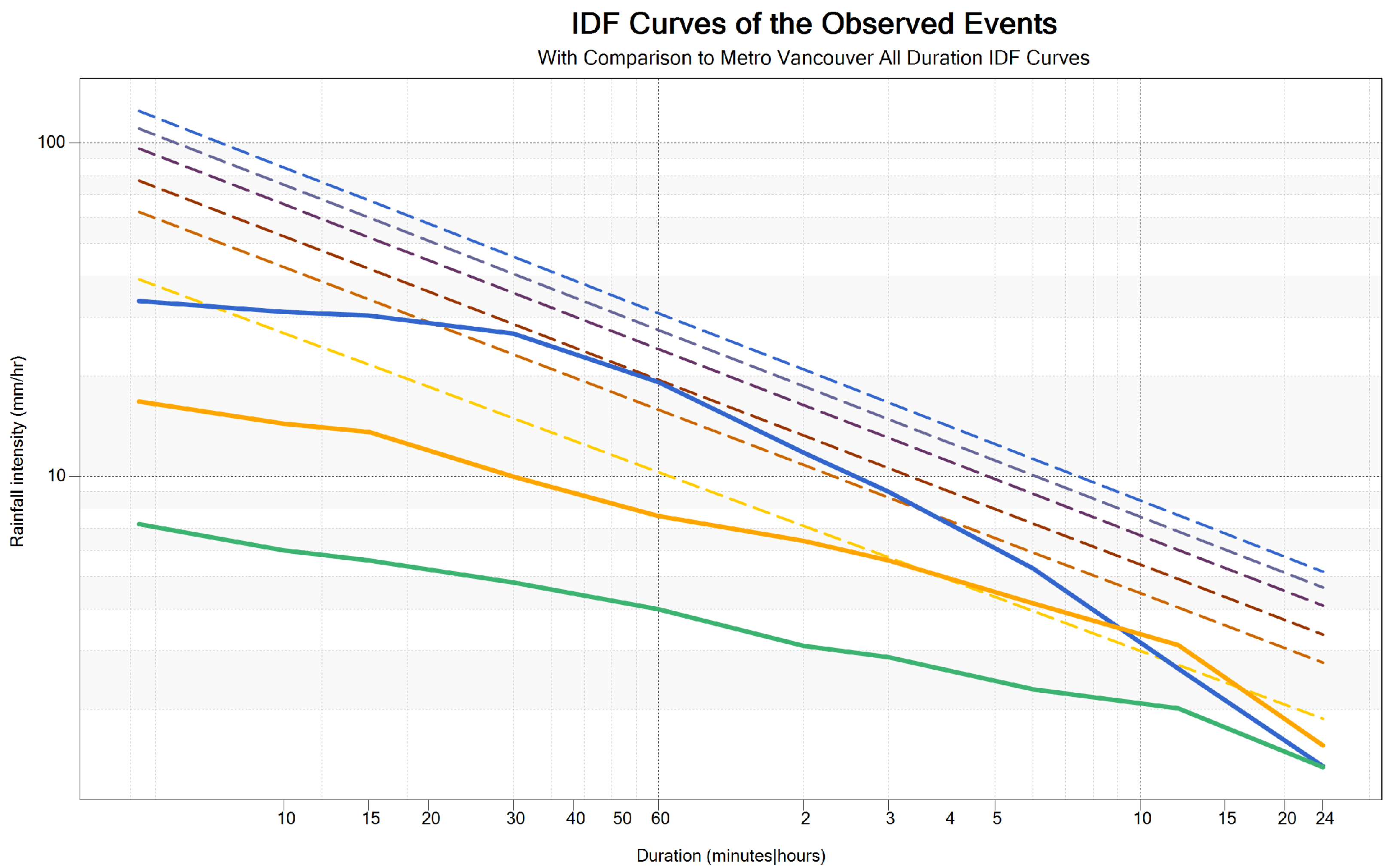
2.6 Future Growth Projected in the OCP

After model validation was completed, a second model was developed to incorporate future conditions described in the OCP. The "future" model contains increased populations and additional non-residential areas that contribute to the total sewage flow generations and sanitary flows. With the land use conditions described in the OCP, the number of mixed-use parcels will increase under the future condition. This resulted in a larger number of nodes in the future model to generate both residential and non-residential DWF. The future model was used to assess the performance of the existing system under future growth and development conditions.

The total population growth (difference between projected 2045 high scenario and 2016 population) was distributed to areas with expected growth based on the maximum allowable density for each area. The allowable density was determined from the maximum possible floor area ration (FAR) in each area as per the OCP and the North Bluff Road Study memo assuming there is a direct relationship between population density and FAR. Although it is noted that this relationship does not necessarily hold true as it depends on the actual developments that happen in each area. The population density was adjusted upwards at areas where the City had already approved for higher densities. The population growth was re-distributed after the adjustment. Areas that are not expecting significant growth according to the OCP are Mature Neighbourhood, Open Space & Recreation, Institutional, and some parcels of East Side Large Lot Infill Area. Parcels in these areas have no increase in population in the future model, with the exception of areas dedicated to affordable housing and assisted living.

Additional non-residential areas were added to the model based on the OCP's land use policies. The majority of the non-residential developments will be located in Town Centre, Town Centre Transition and Lower Town Centre.

FIGURE 2.1



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IDF Curves
of the Observed Events

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3.0 Existing and Future Demands

A summary of the existing and future demands is provided in Table 3.1.

Table 3.1: Average DWF under Existing and Future OCP Conditions

	Existing Condition	Future OCP Condition
Residential Area	299 Ha	283 Ha
Non-Residential Area	52 Ha	37 Ha
Mixed-use Area	4 Ha	35 Ha
Population	19,952	27,300
Residential Generation Rate	210 L/c/day	210 L/c/day
ICI Generation Rate	2,000 L/Ha/day	2,000 L/Ha/day
ADWF (without GWI)	50 L/s	70 L/s
Total GWI	26.7 L/s	26.7 L/s
Total ADWF	**77 L/s**	**97 L/s**

4.0 Design Criteria

The City's current design criteria is provided in Subdivision Bylaw 777 Schedule B. The Bylaw states that for new systems, the ADWF shall be not less than 360 L/c/day and the average infiltration rate shall be 0.06 L/s/ha.

For the assessment of the existing system, an ADWF of 210 L/c/day was used as a more representative value of the generation rate.

The same wet weather criteria as the previous master plan was selected for consistency. The peak wet weather event used to simulate the RDII component was the 50-Year 2-Hour storm event. ISL also performed a check to make sure the 2-Hour event resulted in the highest flows in the system among the 1, 2, 6, 12, and 24 hour storms.

5.0
Evaluation of the Sanitary System

5.1 Assessment Criteria

The capacity of gravity sewers in the sanitary system was assessed based on the ratio of peak flow rate to maximum design flow rate and the level of hydraulic grade line (HGL) in the system. Pressured pipes (i.e. forcemains and siphons) were evaluated based on their maximum velocity as higher velocities would result in undesirable head loss and scour. The entire system was evaluated under the 50-Year 2-Hour storm event. The 2-Hour event was determined to result in the highest flows in the system compared to the 1, 6, 12, and 24 hour events.

The ratio of peak flow rate to maximum design flow rate (Qpeak/Qcapacity) were divided into four categories based on the risk of surcharging in the pipe. The four categories are listed as follows:

- *Qpeak/Qcapacity less than 50%*: risk of surcharging is insignificant
- *Qpeak/Qcapacity between 50% and 80%*: low risk of surcharging
- *Qpeak/Qcapacity between 80% and 100%*: moderate risk of surcharging
- *Qpeak/Qcapacity greater than 100%*: high risk of surcharging

The HGL shows the peak water level in the system and provides a visual indication of whether a surcharge will result in surface or basement flooding. The HGL was measured with reference to the ground level at nodes in the model and they were categorized into the following:

- HGL above ground
- HGL less than 1.2m below ground
- HGL between 1.2m and 2.4m below ground
- HGL greater than 2.4m below ground

For buildings that have basements, the HGL should be below basement levels to minimize the risk of basement flooding at service connections.

Velocities in pressured pipes have a threshold of 3.0 m/s as per industry standards. The simulation results of pressured pipes were shown as the following:

- Maximum velocity <3.0 m/s
- Maximum velocity >3.0 m/s

The assessment criteria in this master plan was slightly different from the previous study in 2013. The previous study proposed that any pipes that received flows greater than 80% of its capacity would require an upgrade. Given the City's limited capital budget, this approach may be slightly over conservative and not cost effective. In this study, the system was evaluated so that pipes with flow rates exceeding 100% their design capacity would have priority to be upgraded to reduce the risk of surface or basement flooding. In comparison, pipes with flow rates greater than 80% but less than 100% of their capacity would not require an upgrade as long as the HGL is within the pipe (i.e. below pipe obvert). Keeping the HGL below pipe obvert reduces the risk of surcharged pipes under the simulated event. For consistency with the previous study, pipes with flow rates between 80% and 100% of their capacity were identified as optional upgrades. These upgrades can be completed with future development as necessary.

5.2 Assessment Results

The existing sanitary sewer system capacity was assessed for the peak wet weather flow condition under the 50-Year 2-Hour storm event. Figures 5.1 and 5.2 show the assessment results under the existing and future OCP population and land use conditions. The results are also summarized in Table 5.1 through Table 5.4.

Table 5.1: Sanitary Sewer Capacity Summary

	Existing Condition		Future OCP Population	
Qpeak/Qcapacity	**Number of Pipe Segments**	**Length (m)**	**Number of Pipe Segments**	**Length (m)**
0-50%	1,381	70,091	1,373	71,149
50-80%	55	3,176	60	3,209
80-100%	16	1,169	20	1,179
>100%	18	2,144	17	1,043

As shown in Table 5.1, the result of increased population and non-residential areas did not increase the peak flows in the system significantly. This is because a significant portion of the peak flows was a result of the RDII generated during the wet weather flows.

Table 5.2: HGL at Manholes/Model Nodes

	Existing Condition	Future OCP Population
Depth	**Number of Nodes**	**Number of Nodes**
>2.4 m below ground	202	201
1.2 – 2.4 m below ground	1,151	1,150
<1.2 m below ground	126	130
Above ground	26	24

As shown in Table 5.2, the HGL is less than 1.2 m below ground at over 100 nodes. At the majority of these nodes, the pipe has enough capacity carrying the peak flows and the HGL is below the pipe obverts. The criteria of keeping the HGL at least 1.2 m below ground does not necessarily reflect the risks of basement flooding in the City as the pipes were generally built at a shallower depth.

Table 5.3: Forcemain and Siphon Velocity Summary

	Existing Condition	Future OCP Population
Velocity	**Length (m)**	**Length (m)**
0 – 2 m/s	4,219	4,219
2.0 – 3.0 m/s	105	105
>3.0 m/s	87	87

Table 5.3 indicates that 87 m of the pressured pipe has a maximum velocity exceeding the 3.0 m/s criteria. This is a section of the siphon along Marine Drive, between Maple Street and Finlay Street. The high velocity in the siphon is due to partial depth flow in a steep pipe (7% grade).

Figure 5.3 shows the profile and HGL in the siphon between Maple Street and Cypress Street, and in the gravity sewer on Columbia Avenue upstream of the siphon. The change is siphon size from 525 mm to 450 mm at P-8049 is causing backwater that impacts the upstream residential area and the HGL is close to ground level at nodes 4760 and 1561 (upstream of P-8049).

Table 5.4: Inflows to Pump Stations

		Existing Condition	Future OCP Population	
Pump Station	**Catchment Area (Ha)**	**Modelled Inflows (L/s)**	**Modelled Inflows (L/s)**	**Estimated Pump Capacity Range**
Ash	8.2	27	28	49-52 L/s
Bergstrom	19.4	31	31	20-24 L/s
Keil	8.0	24	25	52-63 L/s
Metro Vancouver	356.2	500	506	370 L/s

It was identified that the MV Pump Station is currently undersized under the 50-Year peak hour event, since the previously proposed sanitary diversions and new pump station to reduce inflows to the MV Pump Station were no longer being constructed. The MV Pump Station cannot convey the flows under peak storm events and is causing backwater flows in the sewers upstream. It was noted that the bypass sewer data in the 2013 model was inconsistent with the data provided in the Metro Vancouver's online GIS database. The bypass sewer in the Metro Vancouver's online GIS database is at a lower elevation and is a larger pipe that is sloped towards the outfall. In comparison, the 2013 model data has a smaller pipe with zero grade and the pipe is at a higher elevation. The configuration of the bypass sewer will affect how much the upstream sewers will be impacted when the pump station is under capacity.

Among the three City owned and operated pump stations, the Bergstrom Pump Station is currently under capacity to service the design catchment area. This was also identified in the 2013 study.

5.3 Condition Assessment

In addition to the system capacity assessment, ISL had also reviewed CCTV data compiled by AECOM and Binnie. The reports include:

- AECOM Area C Spring Flushing CCTV Memo, 2017
- Binnie Area D & E CCTV Inspection Program Assessment and Evaluation Report, 2017
- AECOM Area B Spring Flushing CCTV Memo, 2016
- AECOM Area A Spring Flushing CCTV Memo, 2015

These reports cover Areas A, B, C, D and E of the City (shown in Figure 5.4). In these reports, AECOM and Binnie reviewed CCTV inspections and provided conditional assessments that ranked storm and sanitary lines on a scale of 1.0 (Best/Very Good) to 5.0 (Worst/Very Poor). As part of the condition assessments, sanitary lines were reviewed for surface defects such as longitudinal and circumferential cracking, deformities, offset joins, and broken pipe. In addition, the sanitary lines were also reviewed for operational and maintenance defects which included debris obstruction and root intrusion.

A summary of the location of sanitary pipes with structural and service defects as per the CCTV assessments is shown in Figure 5.4. ISL has chosen to only show pipes that had a rating of 'Fair', 'Poor', or 'Very Poor'. These are the pipes in the direst condition according to the CCTV conditional assessments and can be viewed in conjunction with the system capacity assessment in developing the Capital Plan.

In 2018, Binnie completed additional condition assessment on Area 2 for the City. ISL received the additional assessment data from the City following the September 2018 submission of this report.
A summary of the location of sewers with structural and maintenance condition rating of 3 or higher are shown in Appendix B. Condition upgrades required as a result of the additional assessment were not included under Section 6.0 Recommended Capital Work. The City should consider the additional upgrades that may be required during capital planning.

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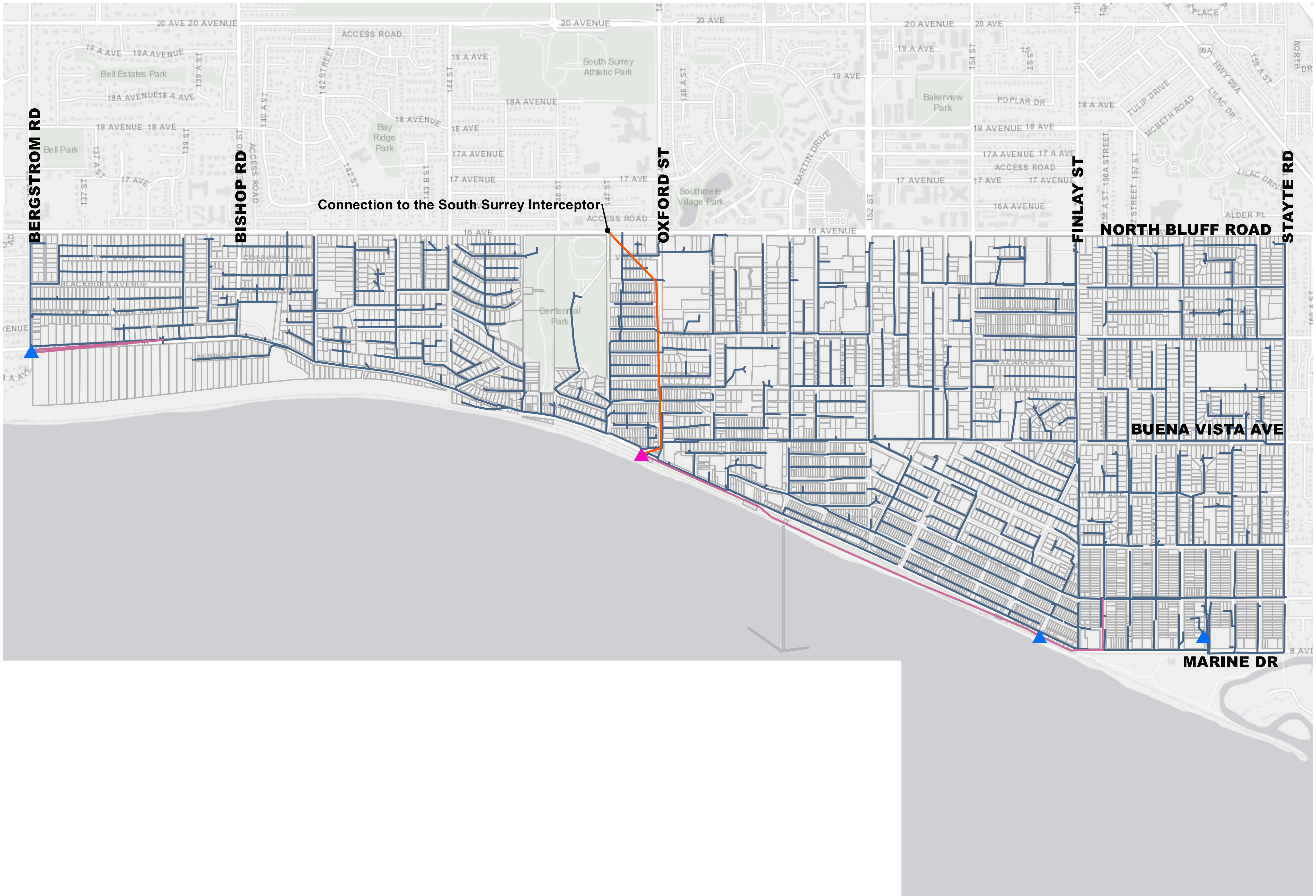
FIGURE 1.5



Legend

- MV Pump Station
- City Pump Stations
- MV Forcemain
- Forcemains & Siphon
- Gravity Sewers

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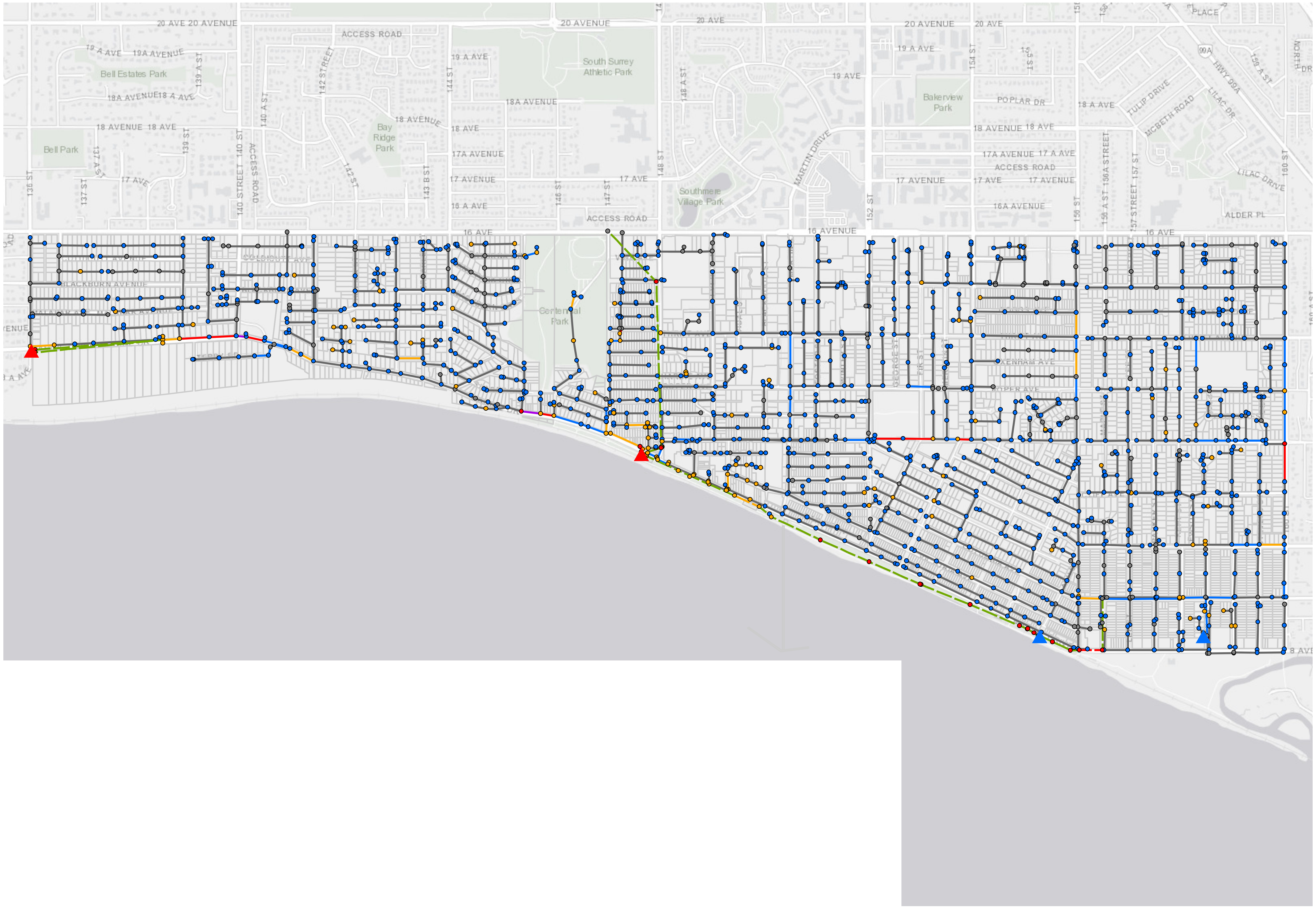
Sewer Master Plan Update 2019

Existing Sanitary Sewer System

ISL Engineering and Land Services

FIGURE 5.2



Legend

Pump Stations

- PWWF > Capacity
- PWWF < Capacity

Node Freeboard (HGL)

- above ground
- <1.2m below ground
- 1.2m - 2.4m below ground
- >2.4m below ground

Max Flow/Design Flow

- <50%
- 50% - 80%
- 80% - 100%
- 100% - 150%
- >150%

Forcemain/Siphon Max Velocity

- 0 - 2 m/s
- 2 - 3 m/s
- >3.0 m/s

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0 0.1 0.2 0.4 0.6 0.8 km

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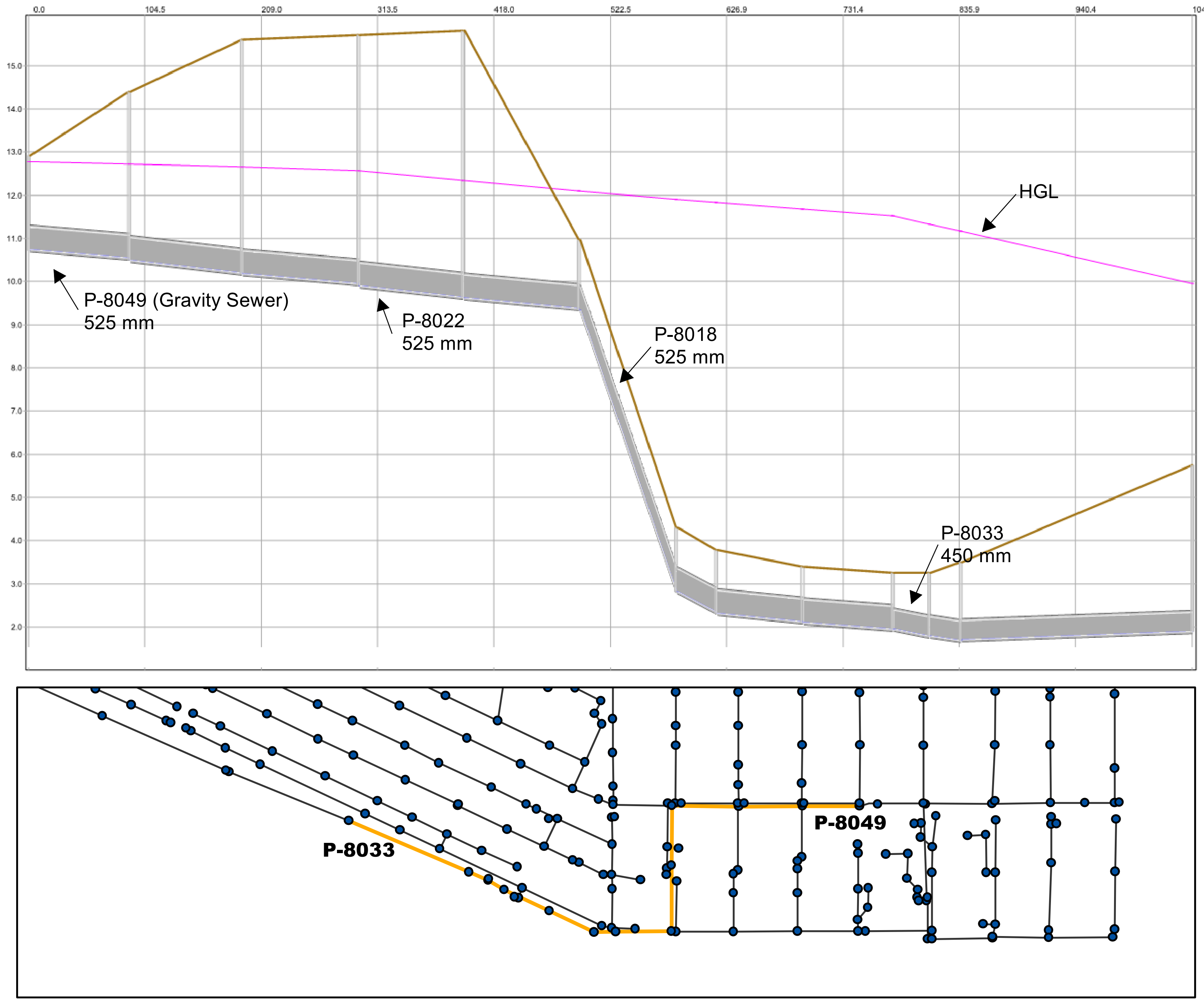
Sanitary Model
Under Future OCP Condition
50 yr - 2 hr

ISL Engineering and Land Services

FIGURE 5.3



Legend

Pipes shown in profile

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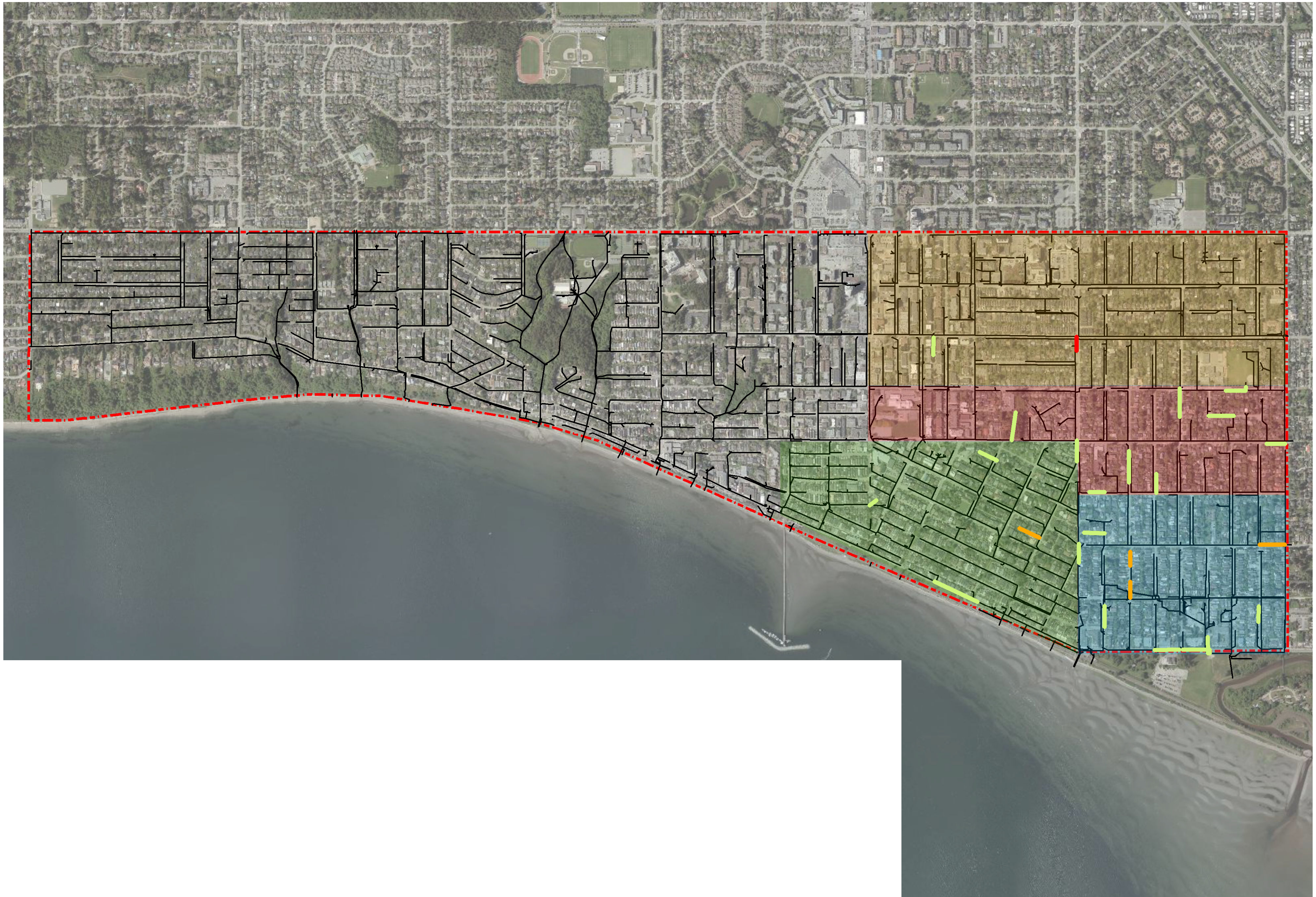
Sewer Master Plan Update 2019

HGL in Siphon
and Upstream Gravity Sewers
(P-8049 to P-8033)

ISL Engineering and Land Services

FIGURE 5.4



Legend

Study Area

CCTV Program Areas

AREA A
AREA B
AREA C
AREA D&E

Condition Rating

3 (Fair)
4 (Poor)
5 (Very Poor)

Note: Only pipes in Fair to Very Poor condition are identified on this map

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0 0.1 0.2 0.4 0.6 0.8 km

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Sewer Master Plan Update 2019

2016-2017 CCTV Programs Pipe Condition Summary

ISL Engineering and Land Services

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEROGRID, IGN, and the GIS User Community

6.0 Recommended Capital Work

6.1 Sanitary Sewer Upgrades

Figure 6.1 identifies locations of upgrades with the proposed sewer size. These upgrades were identified based on the assessment criteria described earlier and the condition assessments. The proposed pipe sizes reduce the peak flow rate to below 100% of the pipe capacity and lowers the HGL below pipe obvert under the 50-Year 2-Hour event under both existing and future OCP conditions. It is recommended that the City should put priority on upgrading pipes currently exceeding 100% of their capacity under the design storm event. Where budget allows, the City can choose to upgrade additional pipes exceeding 80% of their capacity to further reduce potential surcharging or flooding risks. Alternatively, the pipes at lower upgrading priorities can be upgraded as part of future development, under the Local Government Act.

Marine Drive, from Terry Road to east of Nichol Road

This was identified as one of the upgrades in the 2013 study. However, the length of pipe that needs to be upgraded to protect against the 50-Year storm may be reduced, which will lower costs and reduce construction disturbance. Approximately 330 m of 200 mm pipe is proposed to be upgraded to 300 mm compared to the 600 m that was proposed in the 2013 master plan. The Qpeak/Qcapacity of the pipe segment immediately upstream of the upgrade (Pipe ID: P-1267) dropped after the downstream is upsized (as flows are no longer backing up to it), therefore the upgrade in this segment is no longer required. Two pipe segments downstream of the proposed upgrades currently have Qpeak/Qcapacity greater than 80%, however upgrade is not considered necessary since the HGL is below pipe obvert. A larger size of 300 mm is selected for these upgrades as opposed to the 250 mm that was previously proposed, since the increase in material cost from 250 mm to 300 mm is minimal compared to the construction cost. The larger size was selected to provide additional capacity for potential backwater flows from the downstream pipes in a larger storm event (i.e. greater than 50-Year). Note that a few sections of pipes downstream of the upgrade will be smaller (200 mm) compared to the upgraded pipes (300 mm). These smaller pipes were built at a steeper grade which gave them higher capacity. During design and construction of these upgrades, the manhole at the very downstream end of the pipes can be oversized to accommodate for a custom flume benching to prevent hydraulic instability from the drop in pipe size.

Marine Drive, from High Street to west of Oxford Street

The upgrades proposed in this area are generally similar to the 2013 study with some differences in pipe size at Anderson Street. It is recommended that the City verify the configuration of pipes at the downstream end of the proposed upgrades, upstream of MV Pump Station (specifically, P-715 and P-1363). The current configuration based on available data shows that some pipes have a negative slope, which would affect the design of the pipe upgrade if the data in the model is incorrect. If the pipe configuration shown in the model is correct, then these pipes should be reconstructed to eliminate the reverse grade in order to prevent backwater flows and deposition in the pipe. Additionally, the City should coordinate with Metro Vancouver to upgrade the MV Pump Station as soon as possible as the MV Pump Station's limited capacity is currently causing back flows to the upstream sewers. Thus upgrading these sewers will not completely address flooding risks in these areas without upgrading the MV Pump Station.

Buena Vista Avenue, between Best Street and Johnston Road

Three pipe segments on Buena Vista need to be upgraded. In the 2013 study, two of the segments had a proposed size of 375 mm, it is noted that upgrading to a 300 mm pipe is sufficient in reducing the peak flows to just below 80% of design capacity. The 2013 study also proposed to upgrade a short section of pipe at Johnston Road. Upon further assessment on the surrounding conditions and the HGL, ISL determined that this upgrade is not necessary. The high Qpeak/Qcapacity rate is caused by the pipe's flat grade compared to the upstream, giving it a lower design flow. Since the HGL is near the centre of the pipe and the upstream/downstream pipes have good capacity, the surcharging risk in this pipe is low.

Statye Road, south of Buena Vista Avenue

The pipe just south of Buena Vista Avenue is undersized and needs to be upgraded from 200 mm to 300 mm. This upgrade was also proposed in the 2013 study.

Condition Upgrades

Pipes that were rated as “Poor” or “Very Poor” should be upgraded to ensure the pipes will have capacity under peak flow conditions. Since the pipes rated as “Poor” or “Very Poor” are all located closer to the upstream end of the system, trenchless pipe reline can be completed instead of open cut pipe replacement. As shown under the “q/Q (Improved)” in Table 6.4, the relined pipes with a smoother internal wall will still have enough capacity to carry the peak flows despite having a smaller inner diameter.

To address structural defects in pipes that received better ratings, point repairs can be completed. Pipes rated as “Fair” and were recommended in the CCTV memos to have trenchless point repair (TPR) or external point repair (EPR) are listed in Table 6.1. Additional repairs recommended in the CCTV memos can be completed if the City has available budget.

Table 6.1: Point Repairs for Pipes with "Fair" Ratings

Location	Pipe ID	Diam (mm)	Defect	Proposed Rehab
Propect Cres	83	200	Hole	TPR
Buena Vista Ave	86	250	Separated joint	TPR
Marine Dr	108	250	Loss of level	EPR
Roper Ave	498	200	Broken at service lateral	TPR
Kent St	583	200	Hole	TPR
Finlay St	733	300	Broken	TRP
Lee St	779	200	Fracture	TPR
Cliff Ave	796	200	Joint offset	EPR
Parker St	799	200	Fracture, join separation, cracks	TPR
Moffat Lane	878	200	Cracks, broken	EPR
Marine Dr	988	200	Hole (in liner)	TRP
Stevens St	1014	200	Hole	EPR
Marine Dr	1043	250	Loss of level	EPR
Buena Vista Ave	1357	200	Broken	EPR

Note: TPR – Trenchless Point Repair; EPR – External Point Repair

6.2 Siphon Upgrades

The siphon along Marine Drive and on Maple Street is currently causing sanitary flows to back up to the gravity sewers on Columbia Avenue. When the siphon was replaced in 2000, it appears (based on GIS and 2012 model data) that some intermediate sections of the siphon on Marine Drive had a smaller size of 450 mm compared to the rest of the siphon at 525 mm. The reduction in siphon size and addition of flows from the Ash Pump Station is causing a steeper HGL in the system during the 50-Year storm and backing up flows to the residential area on Columbia Avenue. Upgrading the siphon where it is currently 450 mm to 525 mm will lower the HGL in the gravity sewers on Columbia Avenue, thus reducing flooding risks in the upstream residential area. This upgrade was proposed based on the older, 300 mm siphon not in service as per the 2013 sewer master plan update.

ISL recommends the City to investigate the feasibility and cost associated with other upgrade options to address the backwater effects from the siphon. Alternatives to upgrading the siphon may include construction of underground storage next to the Ash Pump Station, or diverting the Ash Pump Station forcemain away from the existing siphon.

A section of the siphon on Marine Drive, between Maple Street and Finlay Street, has a maximum simulated velocity greater than 3.0 m/s due to the top end of the siphon operating under partial depth in a section of steep pipe. The velocity cannot be reduced by changing the pipe size as it is caused by the steep slope of the siphon pipe. This is not considered to be a concern within the siphon.

6.3 Pump Station Upgrades

MV Pump Station

As indicated in Table 5.4, the MV Pump Station is currently under capacity to service the City under the 50-Year 2-Hour storm. The pump station needs to be upgraded with higher pumping capacity to convey the peak 50-Year flows. Lowering the bypass sewer to prevent backwater flows is not an option as it will introduce additional sanitary overflows to the ocean. As mentioned earlier, the bypass sewer data is inconsistent with Metro Vancouver's online GIS data. The bypass sewer configuration should be verified.

An alternative to upgrading the pumping capacity is to construct an underground storage to temporarily store the inflows that exceed the MV Pump Station capacity. This option should be investigated if Metro Vancouver does not plan on upgrading its pump station in the near future, or if Metro Vancouver requires the City to reduce its peak wet weather flows to the MV Pump Station. The order of magnitude of the storage volume was estimated for the 50-Year storm by comparing the inflows to the MV Pump Station and the outflows based on MV pumping rates. The estimated volume was in the order of 5000 m^3, which would equate to an order of magnitude cost of $5 million. The storage elevation is critical for the storage facility to be reserved until all MV pumps are operating and the flows start backing up and surcharging the City's trunk sewers. Further study on this option would be needed to refine the storage volume, determine the optional location and provide more detailed cost estimates.

Bergstrom Pump Station

Bergstrom Pump Station is under capacity based on the estimated design capacity from the 2013 study. Since the pump station cannot convey the inflows under the 50-Year storm event, it is causing flows to back up to residential areas on Marine Drive and on 136 Street (the west boundary of the City). The City should confirm the pump station capacity and design upgrades as necessary.

Additionally, the City should confirm the capacity of all pump stations in the City to ensure the design capacities in the model were not overestimated.

6.4 Capital Plan

The proposed sanitary improvements were prioritized into a 5-Year Capital Plan based on the City's annual capital budget for the next 5 years. The Terms of Condition originally requested a 10-Year Capital Plan but with the City's budget, the proposed sanitary improvements can be completed within 5 years. The City's available capital budget for sanitary improvements between 2018 and 2022 are provided in Table 6.2 below. The improvements were prioritized based on pipe condition, surcharging risk (level of HGL), and the available budget each year.

Table 6.2: City of White Rock's Available Capital Budget for Sanitary Works (2018-2022)

Year	Budget
2018	$3.20M
2019	$3.03M
2020	$1.52M
2021	$1.17M
2022	$1.38M

The unit cost used in the Capital Plan are provided in Table 6.3. An additional 10% engineering fee and 25% contingency allowance were added to the total of each year's upgrade costs.

Table 6.3: Unit Cost of Upgrades

Pipe Size/Type of Repair	Unit	Unit Cost
200 mm	m	$770
250 mm	m	$817
300 mm	m	$878
375 mm	m	$975
450 mm	m	$1,059
525 mm	m	$1,234
600 mm	m	$1,319
675 mm	m	$1,500
TPR/EPR	ea	$500
Reline	m	$500

The proposed capital projects are summarized into Table 6.4 based on the following prioritization:

1. Pipes that need rehabilitation: "Poor" and "Very Poor" rated pipes, and point repairs for "Fair" rated pipes.
2. Pipes with the highest surcharging risks: Marine Drive, High Street to Oxford Street
3. Pipes at the next highest risks: Siphon along Marine Drive
4. Remaining upgrades prioritized based on decreasing risks and available budget

The location of the proposed capital projects are shown in Figure 6.2. A summary of the total estimated expenditure on capital improvements per year is provided in Table 6.5. At the time of development of the final report, some upgrades that were planned to be completed in 2017/2018 were deferred to 2019. These deferred upgrades are identified in Figure 6.2, although the sanitary model was developed assuming upgraded conditions.

Table 6.5: Summary of Proposed Upgrades and Cost Estimate

Year	Approximate Length of Repair/Upgrades (m)	Cost Estimate
2018	234	$262,049
2019	681	$922,105
2020	1,239	$2,063,884
2021	609 (& pump station)	$2,067,151
2022*	--	--
Total	**2,963**	**$5,315,187**

*Note: Cost of optional repairs are not included in the table. Additional optional repairs can be completed in 2022.

6.5 Development Contribution Requirements

Under the Local Government Act, developers are required to contribute to a portion of the capital expenditure costs necessary to service growth. This development contribution is considered standard practice among most municipalities.

The City may require developers to provide excess or extended services under Section 507 of the Local Government Act. Excess or extended services include a portion of the sewage system that will serve land other than the land being developed. Typically, this can be upgrading of sewers downstream of the development as the downstream system is close to or exceeding capacity. Additional flows from the development will worsen the downstream system capacity. The developer is considered to be “advancing history” by completing development before the City has the necessary sewer capacity. In this case, the developer would either complete the excess or extended services or pay cash in lieu to the City as per the Development Agreement.

If a developer is required to complete or pay for excess or extended services, the developer may apply to enter into a Latecomer Agreement with the City. This allows the developer to administer cost recovery from latecomer properties. Under the Latecomer Agreement, the City can impose charges on subsequent, eligible latecomer developers or owners who benefit and connect to the works.

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FIGURE 6.1

Legend

- Pipe Upgrades
- Condition Upgrades
- PumpStations

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0 0.1 0.2 0.4 0.6 0.8 km

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Proposed Upgrades

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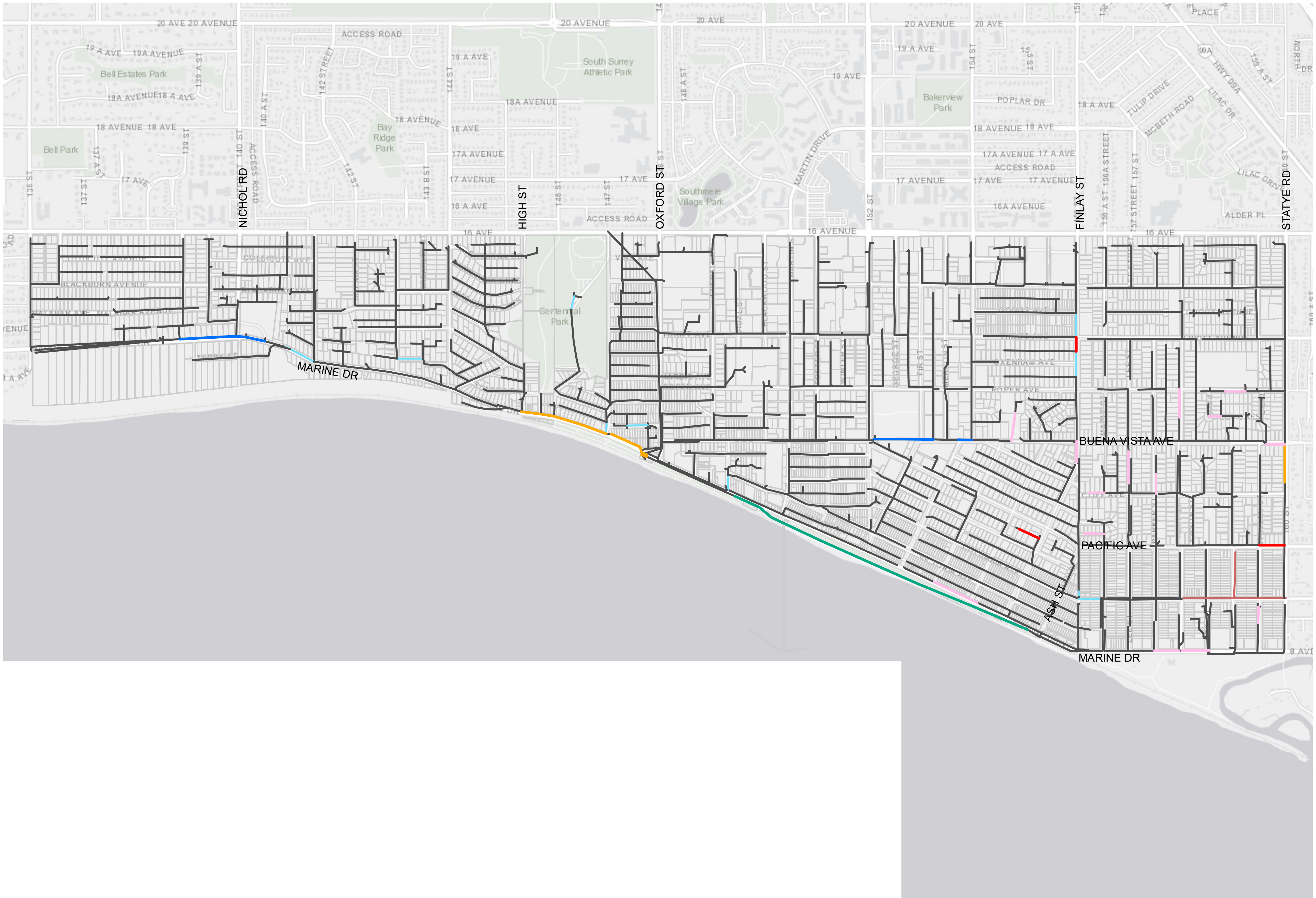
FIGURE 6.2



Legend

Proposed Upgrades

- 2018
- 2019
- 2020
- 2021
- Pipes with Point Repairs (2018)
- Optional Upgrades - May Complete with Future Development
- 2017/2018 Upgrades Deferred to 2019
- Existing Sewers

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Sewer Master Plan Update 2019

Capital Projects

ISL Engineering and Land Services

Document: J:\30000\31950\31939_White_Rock_Drainage-Sewer_MP\02_CADD\25_GIS\251_Figures\Sanitary\Figure 6.2 - Capital Projects.mxd

Date: 2018-12-05

7.0 Conclusion

Based on the assessment of the City's sanitary system under existing and future OCP land use conditions, a number of upgrades were identified. The majority of these upgrades are located along Marine Drive with a few other capacity and condition upgrades scattered throughout the east side of the City. Most of the capacity upgrades were identified in the previous study with the exception of the siphon upgrade. The siphon upgrade would be required under the assumption that only the 525 mm siphon (constructed in 2000) is in service. This upgrade is relatively more costly compared to the other upgrades required in the City. ISL recommends the City to explore other options, such as underground storage, when designing the upgrade.

The MV Pump Station upgrade is critical in mitigating flooding risks in the upstream sewers. The City should coordinate with Metro Vancouver to determine a timeline and strategy in upgrading the pump station. As discussed in the report, the City can explore the cost and feasibility of alternate options to upgrading the pump station.

As mentioned in the report, some pipes in the model appears to have unusual grades or elevation (e.g. reverse grade). ISL recommends to the City to complete field survey to verify pipe configuration prior to designing any sewer upgrades.

It was noted that the Qpeak/Qcapacity of the system did not increase significantly between the existing and future conditions. This is because a significant portion of the peak flows in the system is contributed by wet weather flows (i.e. RDII). ISL recommends the City to investigate into establishing an I&I program to reduce I&I in the sanitary system and prevent unnecessary wet weather related upgrades in the future. Additionally, the City should continue to complete regular CCTV assessments to address structural or service defects in the system on a timely manner. Doing so would also help in reducing I&I in the system.

Appendix A

2013 Sewer Master Plan Update by AECOM

Water Resources

City of White Rock

Sewer Master Plan Update

Final Report

Prepared by:

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www.aecom.com

Project Number:

60238740

Date:

January 7, 2013

Statement of Qualifications and Limitations

The attached Report (the “Report”) has been prepared by AECOM Canada Ltd. (“Consultant”) for the benefit of The City of White Rock (“Client”) in accordance with the agreement between Consultant and Client, including the scope of work detailed therein (the “Agreement”).

The information, data, recommendations and conclusions contained in the Report (collectively, the “Information”):

- is subject to the scope, schedule, and other constraints and limitations in the Agreement and the qualifications contained in the Report (the “Limitations”);
- represents Consultant’s professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to Consultant which has not been independently verified;
- has not been updated since the date of issuance of the Report and its accuracy is limited to the time period and circumstances in which it was collected, processed, made or issued;
- must be read as a whole and sections thereof should not be read out of such context;
- was prepared for the specific purposes described in the Report and the Agreement; and
- in the case of subsurface, environmental or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time.

Consultant shall be entitled to rely upon the accuracy and completeness of information that was provided to it and has no obligation to update such information. Consultant accepts no responsibility for any events or circumstances that may have occurred since the date on which the Report was prepared and, in the case of subsurface, environmental or geotechnical conditions, is not responsible for any variability in such conditions, geographically or over time.

Consultant agrees that the Report represents its professional judgement as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement, but Consultant makes no other representations, or any guarantees or warranties whatsoever, whether express or implied, with respect to the Report, the Information or any part thereof.

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AECOM: 2012-01-06

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January 7, 2013

Mr. Greg St. Louis, P.Eng.
Director of Engineering and Municipal Operations
City of White Rock
Operations Department
877 Keil Street
White Rock, BC
V4B 4V6

Dear Mr. St. Louis:

Project No: 60238740

Regarding: City of White Rock Sewer Master Plan Update
Revised Final Report Submission

Sincerely,
AECOM Canada Ltd.

Please find enclosed our Final Report for the Sewer Master Plan Update and updated 10-Year Capital Plan for the City of White Rock. This report includes all comments received from the City on September 17, 2012 by e-mail as well as additional comments from our meeting with City staff on October 9.

We have enjoyed working with City Staff on this project and we look forward to providing continued services to the City of White Rock. If you have any questions or concerns please don't hesitate to contact us at 604.444.6400.

Sincerely,
AECOM Canada Ltd.

Stephen Bridger, P.Eng.
Project Manager

Encl.
cc:

Distribution List

# of Hard Copies	PDF Required	Association / Company Name
3	Yes	City of White Rock

Revision Log

Revision #	Revised By	Date	Issue / Revision Description
0	Stephen Bridger	June 27, 2012	Draft Report
1	Stephen Bridger	Aug 10, 2012	Final Report
2	Stephen Bridger	Jan 7, 2013	Revised Final Report

AECOM Signatures

Report Prepared By:

Chris O'Donnell, EIT
Project Engineer

Report Reviewed By:

Stephen Bridger, P. Eng.
Project Manager

Executive Summary

The City of White Rock retained AECOM to complete a Sewer Master Plan (SMP) Update including a 10-Year Capital Plan for planning and budgeting purposes. The City is home to approximately 19,240 people and covers an area of approximately 473 hectares. Existing land use in the City is predominantly low density residential with pockets of medium density residential land use and the commercial core is the Town Centre Area. Development in the form of densification is occurring as now townhouses and apartments are planned for construction, with the bulk of the development occurring in the Town Centre and North Bluff Areas. The future 2031 population is projected to be 23,500 based on the 2008 OCP and timing of development activity is dependent on market conditions. In addition, commercial development is anticipated to increase as opportunities arise in the Town Centre and the residential population increases. There is also incremental redevelopment and infill activity in areas outside of the Town Centre that will be ongoing.

Prior to this document, the City had completed an assessment and Capital Plan for the sanitary sewer system in 2005 (*Sanitary Sewer Model, Update and Capital Plan* report by KWL) that was updated in 2010. The 2005 & 2010 reports were based on the 100-Year peak hour inflow and infiltration (I&I) rates developed using the Envelope Method that result in conservative flow estimates and subsequent recommended upgrades. In development of the new SMP, our analysis focused on a review of the I&I rates and recommendation of new rates to be applied for determination of system upgrades to meet existing system requirements and future population growth. In addition, a new hydraulic model was developed using XPSWMM that reflects current land use conditions and the future build out of the OCP. The hydraulic model incorporates all new infrastructure and was used to assess future land use changes for development of the updated Capital Plan.

In terms of existing sanitary sewer infrastructure there are approximately 82 km of sewers that collect sewage from 10 major catchment areas and three City sanitary pump stations (Keil, Ash and Bergstrom) that pump to the Metro Vancouver Pump Station at the foot of Oxford Street. Linear sewer infrastructure includes gravity pipes ranging in diameter from 150 to 600 mm, forcemain ranging from 100 to 150 mm and the siphon ranging from 450 to 525 mm. Sewer pipe materials include asbestos cement, PVC, HDPE and clay pipe. The SMP report includes a summary of the existing information reviewed, details on the model development, model calibration and validation, I&I analysis, sanitary infrastructure assessment, and recommended improvements included in the City's Capital Plan.

As noted above, the major sanitary sewer upgrades included in the previous Capital Plan were designed for conveyance of the 100-Year Peak Hour I&I rates and to reduce the potential for sanitary sewer overflow (SSO) occurrence at the Metro Vancouver PS and elsewhere in the collection system. One of the most significant changes for the updated Master Plan is the use of a 50-Year event as opposed to 100-Year event that was previously used. Although the 50-Year event is less conservative, it was determined to be more realistic and achievable for White Rock given the limited tax base available for implementation or large scale capital projects. The 50-Year return period was also determined to be an acceptable frequency that one may anticipate localized sewer backup to occur balanced with financial capital cost for system upgrades compared to larger events. This change is further rationalized given that Metro Vancouver's documented I&I target is for the 5-Year return period event (or 11,200 L/Ha/day) and there was no anecdotal evidence of reported overflows within the last 5 years from either the City or Metro Vancouver.

A major upgrade that was proposed in the previous Capital Plan included a new diversion (from Johnston Street to Thrift Avenue) and pump station at Oxford Street and Thrift Avenue that would be tied into the Metro Vancouver forcemain on Oxford Street. The diversion and pump station were intended to reduce flows to the Metro Vancouver Pump Station as it is currently undersized for future flow conditions. Several challenges were presented with siting of a new pump station at Oxford and Thrift included the tie-in to an aging AC MV forcemain, utility relocations, and provisions for overflows requiring connections to the gravity sewer system. The forcemain connection issue also introduced the need to construct a new twin forcemain to convey flows up to the MV interceptor sewer at North Bluff Road and the associated costs for this infrastructure.

Upon further review of the proposed diversion and pump station at Oxford Street we confirmed that the major benefit was a reduction in flows at the MV Pump Station and there were limited improvements to the City's collection system. It was also determined that under the future conditions the MV Pump Station is still in need of upgrading even with a new Oxford Street Pump Station. In such case, we have not recommended that the City proceed with the new pump station or associated diversion as this would result is significant cost to the City that could be avoided by upgrading the MV Pump Station. It should also be noted that upgrades to the MV Pump Station would be funded by the Fraser Sewerage Area managed by Metro Vancouver and not directly by the City.

Another item is the temporary diversion of a sewer on Thrift Ave west of Oxford due to slope stability concerns in the Anderson Ravine. We recommend that the City purchase the land where the temporary sewer bypass is located west of Oxford between Thrift and an easement to the south to alleviate risk of sewer failure in Anderson Ravine. The current ROW agreement expires in February 2013 and the City has noted that an agreement for purchase of the ROW could be reached.

In summary, the updated proposed recommendations and collection system upgrades are discussed below and shown in ***Figure 5.1***. The proposed upgrades are based on the capacity assessment criteria and condition data discussed in ***Sections 3*** and ***4***.

- There are upgrades noted in the vicinity of Marine Drive and Oxford Street and it is recommended that the sewer and manhole inverts at this location be verified. During the model development there were four different sources of information (Metro Vancouver drawing, White Rock Siphon record drawings, City' GIS data and Hydra model information) that all provided differing data for the pipe/manhole inverts leading up to the connection with Metro's Pump Station. Ultimately the GIS and record drawing information was used for this Capital Plan Update but results in several reverse graded pipes at this location. Although unusual, this may be how the system is configured in reality and should be field verified by topographic survey of the sewers in this area.
- As noted above the City should purchase a ROW where the temporary sewer bypass is located west of Oxford, between Thrift and the easement to the south, to make this a permanent sewer and alleviate the risk of slope failure in Anderson Ravine.
- A review of the pump station capacities was completed based on a comparison of model predicted PWWF and estimated pump station capacity. The capacity analysis shows that Bergstrom Pump Station is undersized. Prior to initiating any works further assessment of the pump station capacity is required along with a pump station condition assessment for all three sanitary pump stations.
- City operations staff noted routine maintenance issues at the pump stations (particularly Ash Pump Station) due to grease build-up that should be addressed with a Sewer Bylaw amendment for source control including use of garburators and grease traps. A complete review of the Sewer Bylaw and development of a means to reduce grease build up in sewers at sources thus reducing overall maintenance requirements and likelihood of sewer blockages is required.
- Further to the recommendation above, the Sewer Bylaw is in need of updating to include current I&I rates as well as an updated list of approved pipe materials.
- Although the City has completed CCTV assessment on the majority of the system, there is a significant length of sewer pipe where the surveys were either abandoned or incomplete. The reasons for the incomplete sewer inspections (extracted from the GIS data) are listed in ***Section 4.2*** and these locations should be revisited.

- There are a significant number of sewers with an internal condition grade (ICG) of 3 or 4 and most of these have been highlighted for rehabilitation in this SMP report along with several point repairs for sewers with holes or major joint displacements. The City should re-CCTV all proposed locations prior to any works.
- The City should conduct condition assessment on approximately 10% or 8 km of sewers each year. The results of the CCTV inspections should be reviewed by a qualified consultant to determine the rehabilitation works required and priority that they should be completed in. In addition to sewers, the manholes should also be inspected for both structural and service defects.
- Operation and maintenance of the Siphon could be enhanced by reinstating the water level monitor for the siphon at Maple Street and Victoria Ave. An operations manual for the Siphon was completed back in 2000 when the project was completed and this document should be reviewed to assess what maintenance measures are in place and could be improved upon.

A summary of the total expenditure for capital improvements per year is provided in **Table E.1** below.

Table E.1 Summary of Capital Plan

Year	Approximate Length to be Replaced (m)	Cost Estimate
2013*	1,349	$ 1,645,800
2014	685	$ 878,400
2015	406	$ 588,000
2016	612	$ 820,900
2017	602	$ 684,500
2018 - 2023	854	$ 2,509,300
Total	**4,507**	**$ 7,126,900**

* 2013 upgrades include works on Marine Drive from High Street to Bishop Road

Figure 6.1 shows the proposed sanitary upgrades based on the year required and a detail breakdown of the proposed capital improvements for each year is shown in ***Table 6.3***.

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1 Introduction

1.1 Overview

The City of White Rock retained AECOM to complete a Sewer Master Plan (SMP) including an updated 10-Year Capital Plan for budgeting purposes. Prior to this document, the City had completed an assessment and Capital Plan for the sanitary sewer system in 2005 (*Sanitary Sewer Model, Update and Capital Plan* report by KWL) that was updated in 2010. The 2005 & 2010 reports were based on the 100-Year peak hour inflow and infiltration (I&I) rates developed using the Envelope Method which results in conservative flow estimates and subsequent recommended upgrades. In development of the new SMP, our analysis focused on a review of the I&I rates and recommendation of new rates to be applied for determination of the updated Capital Plan to meet existing system requirements and future population growth.

According to the 2011 Census data and the current Official Community Plan (OCP), White Rock is home to approximately 19,240 people and expected to grow to 23,500 people by 2031. The City covers an area of approximately 473 hectares and is bound by Semiahmoo Bay to the south and the City of Surrey to the west, north and east as shown in **Figure 1.1**. In terms of sanitary sewer infrastructure there are approximately 82 km of sanitary sewers that collect sewage from 10 major catchment areas and three City sanitary pump stations (Keil, Ash and Bergstrom) that pump to the Metro Vancouver Pump Station at the foot of Oxford Street. Linear sewer infrastructure includes gravity pipes ranging in diameter from 150 to 600 mm, forcemain ranging from 100 to 150 mm and the siphon ranging from 450 to 525 mm. Sewer pipe materials include asbestos cement, PVC, HDPE and clay pipe.

Over the last 10 years, the City has completed a number of sewer condition assessment and rehabilitation projects including CCTV investigations, smoke testing, point repairs, pipe lining, lateral connection investigation, and sewer replacement. A review of all the sewer rehabilitation works completed since 2001 is presented in the *Audit of Sanitary Rehabilitation Program* (KWL 2011) report. In addition to providing a comprehensive summary of the rehabilitation projects and I&I analysis already completed, one of the primary objectives for the audit of the sewer rehabilitation program was to provide documentation for meeting Metro Vancouver's *Integrated Liquid Waste and Resource Management Plan* (ILWRMP) requirements.

The updated SMP was required to review recommendations provided in the previous studies and convert the hydraulic model from HYDRA to a new software package. A review of available hydraulic modelling software was completed and XPSWMM was selected for use based on its capabilities and for consistency with the current drainage model. In such case, a city-wide hydro-dynamic XPSWMM model was developed to reflect current land use conditions and full build out of the OCP. The new hydraulic model incorporates newly constructed infrastructure and was used to assess future land use changes to assist with development of a new Capital Plan.

The SMP includes a summary of the existing information reviewed, details on the model development, model calibration and validation, I&I analysis, sanitary infrastructure assessment, and recommended improvements to be included in the City's Capital Plan.

The City of White Rock

Sewer Master Plan Update

Legend

City Boundary

0 100 200 400 600 800
Meters

Project No.	Date
60238740	June 2012

City of White Rock Study Area

Figure 1.1

1.2 Key Issues & Objectives

This SMP is in-line with the City's OCP goals for achieving appropriate levels of services and infrastructure improvements to accommodate growth and preventing adverse impacts on health, property and the environment. To achieve these goals, the SMP outlines a phased Capital Plan for sanitary sewer infrastructure improvements and replacement that is within the current budget expenditures.

Current development within the City includes densification of mixed use residential and commercial areas in and around the Town Centre Area (including North Bluff East and West areas and the Johnston Road area) as well as lot subdivision or amalgamation for re-development into smaller single family homes or townhouse infill type developments. The 2031 population is predicted to be 23,500 which is an increase of 22%.

In summary, the main objectives of this Sanitary Master Plan are as follows:

- Review existing and background information;
- Develop an updated hydraulic model of the City's entire sanitary sewer system using XPSWMM;
- Calibrate the model to "existing conditions/populations" using dry and wet weather flow data, then validate the model;
- Develop sanitary flow projections for the OCP land use and population;
- Identify I&I rates across the City in accordance with the ILWRMP;
- Assess the hydraulic capacity of the sewer system for future conditions;
- Assess condition of existing sanitary infrastructure and provide a plan for a continued CCTV condition assessment program; and
- Develop a Capital Plan for the City using a phased approach for short, medium, and long term projects.

1.3 Key Terms and Abbreviations

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

Table 1.1 – Key Terms/Abbreviations

Term	Definition
Average Dry Weather Flow (ADWF)	Lowest 24-hour average sanitary flow during a 7-day period of dry weather. ADWF is 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.
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.
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.
Internal Condition Grade (ICG)	The internal condition grade ranges from 1 (good) to 5 (bad) for structural or service condition of sewers
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 XPSWMM to quantify 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.

2 Model Development and Calibration

This section provides a summary of the existing sanitary system, GIS data review, model software review and selection, flow monitoring data, sewage flow generation, model calibration and validation and design criteria (including I&I rates). GIS data was provided by the City's GIS department and included shapefiles for the sanitary system, cadastral information for the lots and streets, as well as existing and future land use information. The previous sanitary sewer model was also provided but was developed using HYDRA software which is not well supported any longer nor commonly used by Municipalities in the Lower Mainland. Information extracted from the old model was limited to the pipe/MH network information as the sewer loading information from the old model was out of date given that it was based on 2001 Census information.

2.1 GIS Data Review

A review of the sanitary system GIS data was completed to identify any "data gaps" that needed to be rectified prior to development of the XPSWMM hydraulic model. A few of the GIS data gaps that were noted include the following items:

- 0.3 % of pipes were listed in GIS with no diameters and for these pipes we obtained information from the old HYDRA model data or inferred from upstream and downstream sewers,
- 4.3 % the pipes do not have an entry for the pipe age or installation date which was assumed to similar to adjacent sewers, and
- pipe inverts were incorrect for a few pipes and new inverts were inferred from connecting sewers or ground elevations.

As-built drawings were also reviewed to collect information for pipes recently constructed and not yet in the GIS database and this information was entered into the hydraulic model.

Model connectivity gaps were reviewed using connectivity tools available in XPSWMM to ensure all pipes are connected to manholes. For all new conduits the pipe diameters, lengths, and slopes were reviewed in GIS prior to importing into the model as well as any connectivity gaps such as:

- Upstream and downstream pipe inverts being cross-referenced or switched;
- Manholes not connected to any pipe or pipe not having an assigned upstream/downstream manhole ID;
- Manhole inverts above the pipe invert; and
- Manhole inverts above the ground elevation.

Once the GIS data gaps were resolved, the sanitary network attributes were imported into XPSWMM. Data imported into the model included pipe ID, diameter, pipe inverts, length and material type, manhole ID, inverts, rim elevations and "X-Y" coordinates.

2.2 Model Software Review and Selection

The City of White Rock was previously using HYDRA software for the sanitary sewer model and is currently using an XPSWMM model for drainage master planning. With the development of the SWMM 5.0 engine by the USEPA there are a number of SWMM based programs now on the market (EPA SWMM, XP SWMM, PCSWMM, InfoSWMM) that offer more flexibility and superior hydraulic capabilities and accuracy over HYDRA. A SWMM based model is more applicable for the City of White Rock when considering the complex hydraulics resulting from steep sewers draining to flat sewers on Marine Drive, the Marine Drive siphon, pump stations, diversion manholes and overflows.

The software review and evaluation process is an informative process that includes input from our hydraulic engineering specialists and City Staff to ensure the selected software meets the City's immediate and long-term requirements. The software review considered the following fundamental model elements:

- hydraulic capability and accuracy;
- graphics (interface and result output);
- GIS integration;
- cost (initial purchase & annual support);
- user friendliness (how easy is it to use); and
- what other local Cities are using.

During the evaluation each software platform was ranked based on the various categories and assigned weighting parameters depending on the importance of each one. The four main software vendors in the North American market are XP-Software, Innovyze, Bently and DHI (excluding the freeware EPA-SWMM) and there are 7 sanitary sewer software packages as listed shown in **Table 2.1**.

InfoSEWER and HYDRA have limited hydraulic capability as they have difficulty in modelling hydraulics at pump stations, forcemains, flow diversions and storage systems, therefore these two are not recommended for the City as they move forward and continue to experience growth and development. The remaining 5 packages are all "equally proficient" however they vary in cost, GIS compatibility, user friendliness, run-time speed, hydraulic accuracy and flow generation. **Table 2.1** is a summary of the key software selection findings.

Although DHI's Mike Urban is ranked No. 1 overall we are recommending the City continue to use XPSWMM. There are several reasons for this recommendation:

- The drainage model is already in XPSWMM and operates well;
- XPSWMM is commonly used by City's and consultants in the Lower Mainland area, and there are numerous land development consultants who perform work in the City;
- Hydraulic engine can be run with either EPA SWMM5 or the full St. Venant dynamic equations that are applicable to conditions in White Rock for effects of storage and backwater in conduits; and
- Relatively low cost compared to its peers for initial license cost as well as annual maintenance.

There are limitations to XPSWMM as noted in the disadvantages which can be overcome. The GIS integration can be achieved using the ODCB database import function and with XPSWMM 2011 they have increased the GIS capability. Simulation run times are long but not unreasonable compared to its peers. Similarly for exporting output data, inevitably there is some data manipulation required in Excel and GIS to generate presentable figures; however, this is not usually completed by City staff as it is generally prepared by consultants when updating the Master Plan document.

Table 2.1 – Software Review Key Findings

Vendor	Software	Rank	Key Advantages	Key Disadvantages
Innovyze	InfoSEWER	9	Easy to use	Poor hydraulics (cannot model backwater)
			Good integration with GIS	Poor sanitary flow generation (i.e. Excel & imported)
			Moderate cost	Not Standalone (requires ArcGIS to run)
	InfoSWMM	3	Good hydraulic engine (SWMM 5)	Restricted or limited sanitary flow generation (i.e. Excel & imported)
			Good integration with GIS	Not Standalone (requires ArcGIS to run)
			Moderate cost	More applicable for drainage
	Info Works	4	Excellent hydraulic engine	Expensive (double other software costs)
			Excellent flow generation	Steep Learning Curve
			Good scenario manager	Not widely used in North America
XP-SOFTWARE	Hydra	8	Good flow generation	Poor hydraulics (particularly pump stations)
			Easy to use	Limited GIS integration
			Low cost	Poor Software Support
	XP-SWMM	2	Good hydraulic engine (SWMM 5)	Limited GIS integration
			Good flow generation (L/cap/day)	Long simulation run-times (two modules)
			Widely used across North America	Not easy to export results
			Same Software STM & SAN	
Bentley	SewerGEMS	5	Good hydraulics	Long simulation run-times
			Good flow allocation	Not widely used in Canada
			Easy to use	Moderate Software Support
DHI	Mike Urban (MOUSE)	1	Excellent hydraulic engine	Can be unstable for large models
			Good integration with GIS	Moderate-steep learning curve
			Good flow generation	

2.3 Existing Sewer Infrastructure

There are 10 major sanitary catchment areas that make up the 473 Ha within the City limits and are shown in **Figure 2.1**. The naming convention for the sewer catchment areas was maintained from the previous study for continuity in reviewing historical monitoring data and I&I information.

The entire sanitary sewer system drains to the MV Pump Station at the foot of Oxford Street which is pumped up Oxford Street in a 600mm diameter forcemain to North Bluff Road to the connection with the South Surrey Interceptor. There are approximately 81.6 km of gravity sewers, 700 m of forcemain and a 2.3 km siphon owned and operated by the City. An overview of the sewer system and locations of forcemains and pump stations is shown in **Figure 2.2.** A breakdown of the GIS pipe data showing the diameter size range and material is listed in **Tables 2.2 and 2.3** below.

Table 2.2 Sanitary Sewer Diameters

Diameter	Length (m)
100mm to 200mm	68,011
250mm to 300mm	10,759
375mm to 600mm	2,859
Total Length =	**81,629**

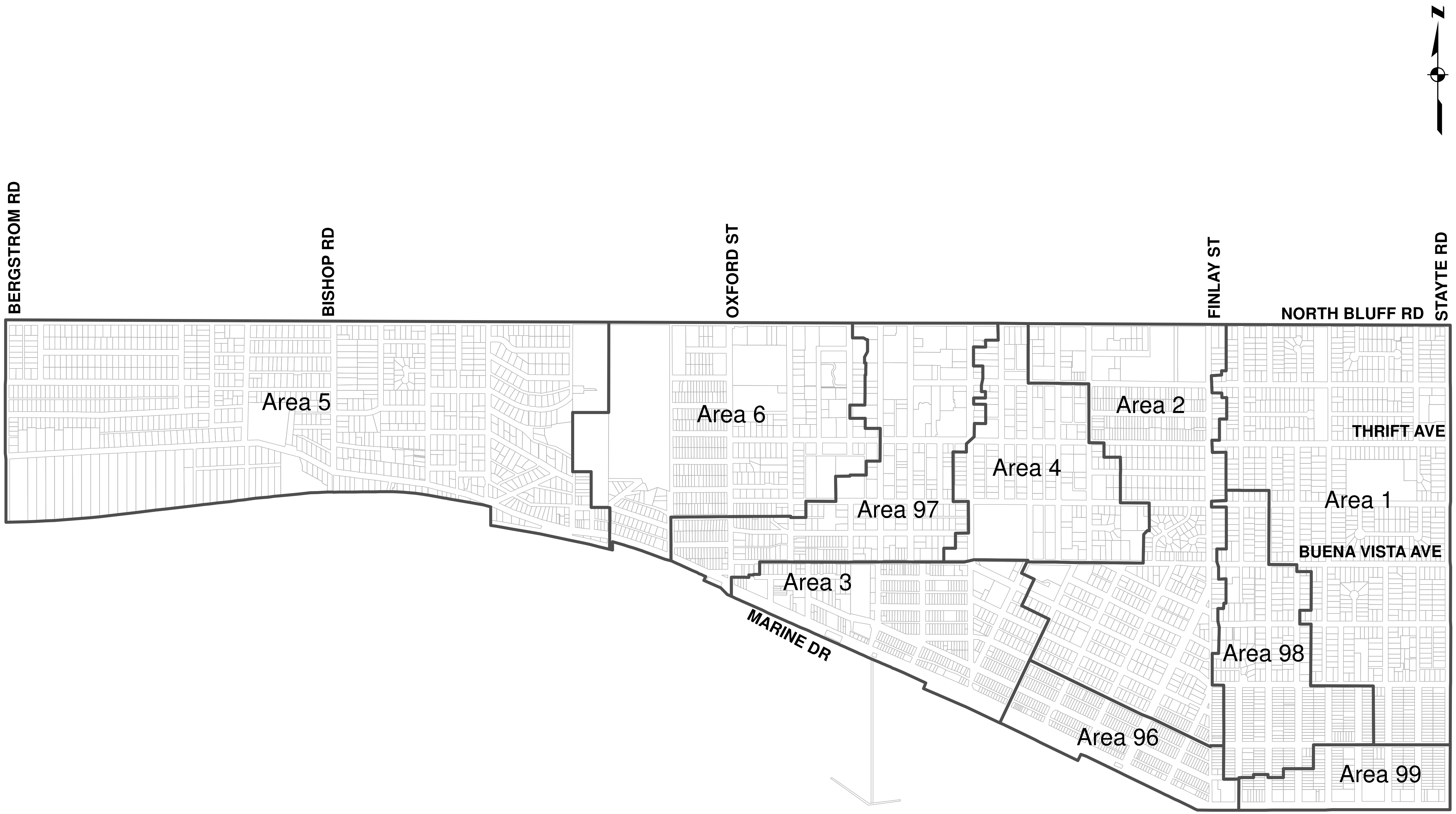
Table 2.3 Sanitary Sewer Materials

Material	Length
Concrete	352
PVC	24,571
Vitrified Clay	19,265
Asbestos Cement	35,763
HDPE	1,322
CIPP (Liner)	108
Unknown	248
Total Length =	**81,629**

There are three pump stations that are operated by the City: Keil PS, Ash PS and Bergstrom PS. Detailed information for the pump stations was provided by the City and from Xylem (formerly Flygt ITT) and is presented in **Table 2.4**. The pump stations are currently equipped with digital level sensors and control panels for operation's use.

Table 2.4 Sanitary Pump Stations

Pump Station	Catchment Area (Ha)	Pump Type	Primary Pump ON/OFF Levels (from approx geodetic)	Secondary Pump ON/OFF Levels (from approx geodetic)
Keil	8.0	2 x 20 Hp MT CP3152	-2.180m / -2.430m	-2.145m / -2.430m
Ash	8.2	2 x 15 Hp MT CP3102.181	1.431m / 1.050m	1.462m / 1.050m
Bergstrom	19.4	2 x 5 Hp MT CP3102.181	64.180m / 63.720m	64.180m / 63.720m



THE CITY OF WHITE ROCK

Sewer Master Plan Update

Legend

Catchment Areas

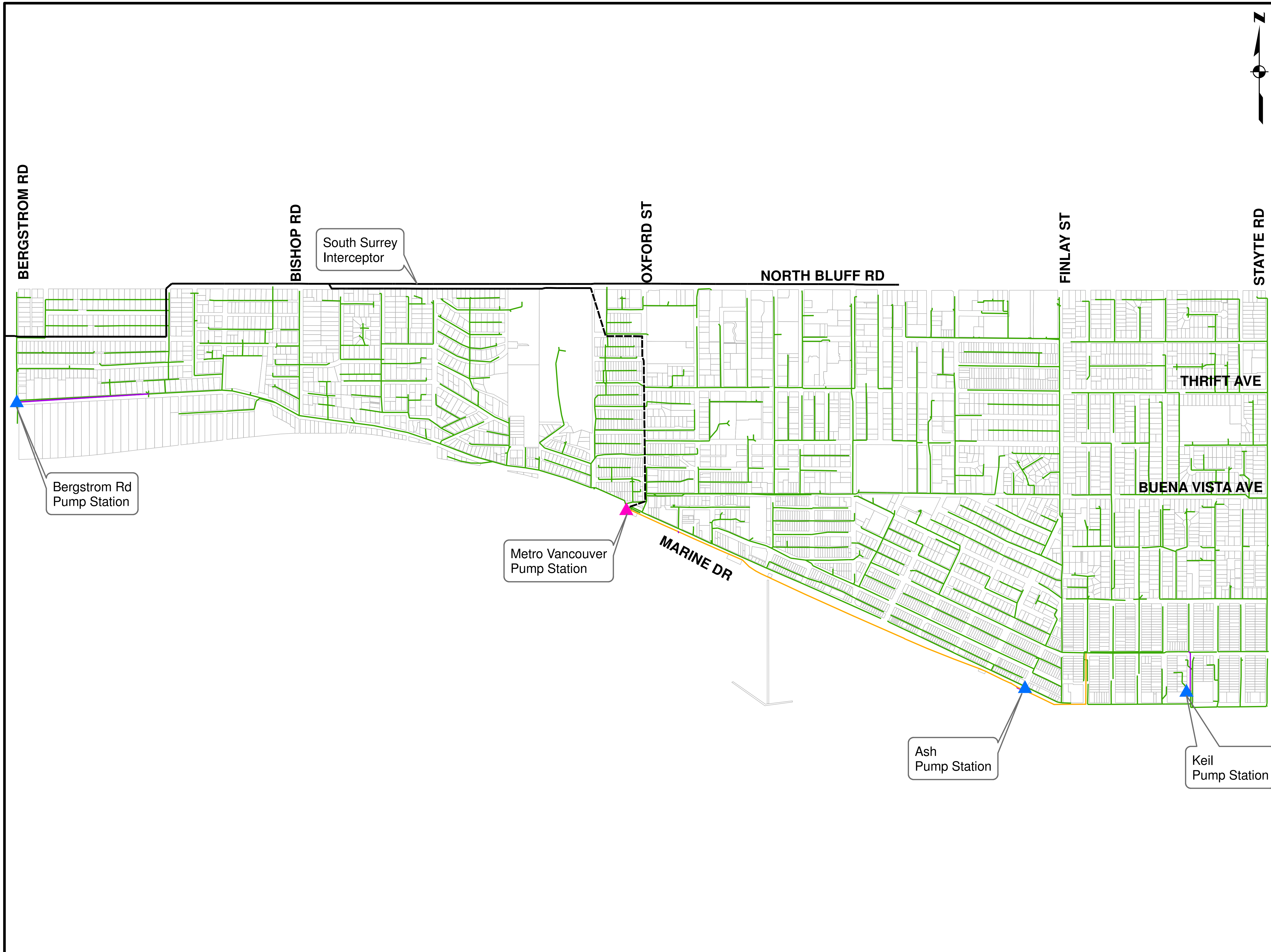
AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	July 2012

Sanitary Sewer Catchment Areas

Figure 2.1



The City of White Rock

Sewer Master Plan Update

Legend

- City Pump Stations
- MV Pump Station
- Gravity Main
- Forcemain
- Siphon
- MV Interceptor
- MV Force Main

0 100 200 400 600 800
Meters

Project No.	Date
60238740	August 2012

Existing Sanitary Sewer System

Figure 2.2

In addition, there is a siphon that conveys sewage from the eastern portion of the City at Keil Street to the MV Pump Station at Oxford Street. The siphon is 2.3 Km in length and was replaced in year 2000. It was designed for a peak wet weather flow (PWWF) capacity of 189 L/s based on 2020 population (from the previous OCP) and an I&I allowance of 22,400 L/Ha/day.

As noted above, all sewage flows in White Rock drain to the Metro Vancouver Pump Station at the foot of Oxford Street. The current White Rock Pump Station has a capacity of approximately 373 L/s. Data for the MV forcemain was used to assist with the model calibration and validation along with the sewer temporary flow monitoring data discussed in the next section.

2.4 Flow Monitoring Data

In order to calibrate and validate the updated model, SFE Global was retained to install four flow monitoring stations at predetermined locations in the City from January to March of 2012. Corresponding rainfall data was obtained from the White Rock STP rain gauge. The flow monitor and rainfall gauge locations are shown in **Figure 2.3** and details of each site are summarized in **Table 2.5** below. **Figure 2.3** also includes the locations of the historical flow monitoring gauges used for the previous studies for reference purposes.

Table 2.5 Summary of Temporary Flow Meter Sites

Site No.	Location	Catchment Area (Ha)	Flow Meter Type	Inlet Pipe Dia. (mm)	Outlet Pipe Dia. (mm)
1	Magdalen Cr at Marine Dr	16.9	Weir with Area/Velocity	250	250
2	Anderson St at W. Beach Ave	43.1	Area/Velocity	250	250
3	Finlay St at Buena Vista Ave	26.4	Weir with Area/Velocity	250	300
4a	Martin St at Prospect St	16.5	Area/Velocity	200	200
4b*	Buena Vista at Martin St	18.9	Area/Velocity	200	200

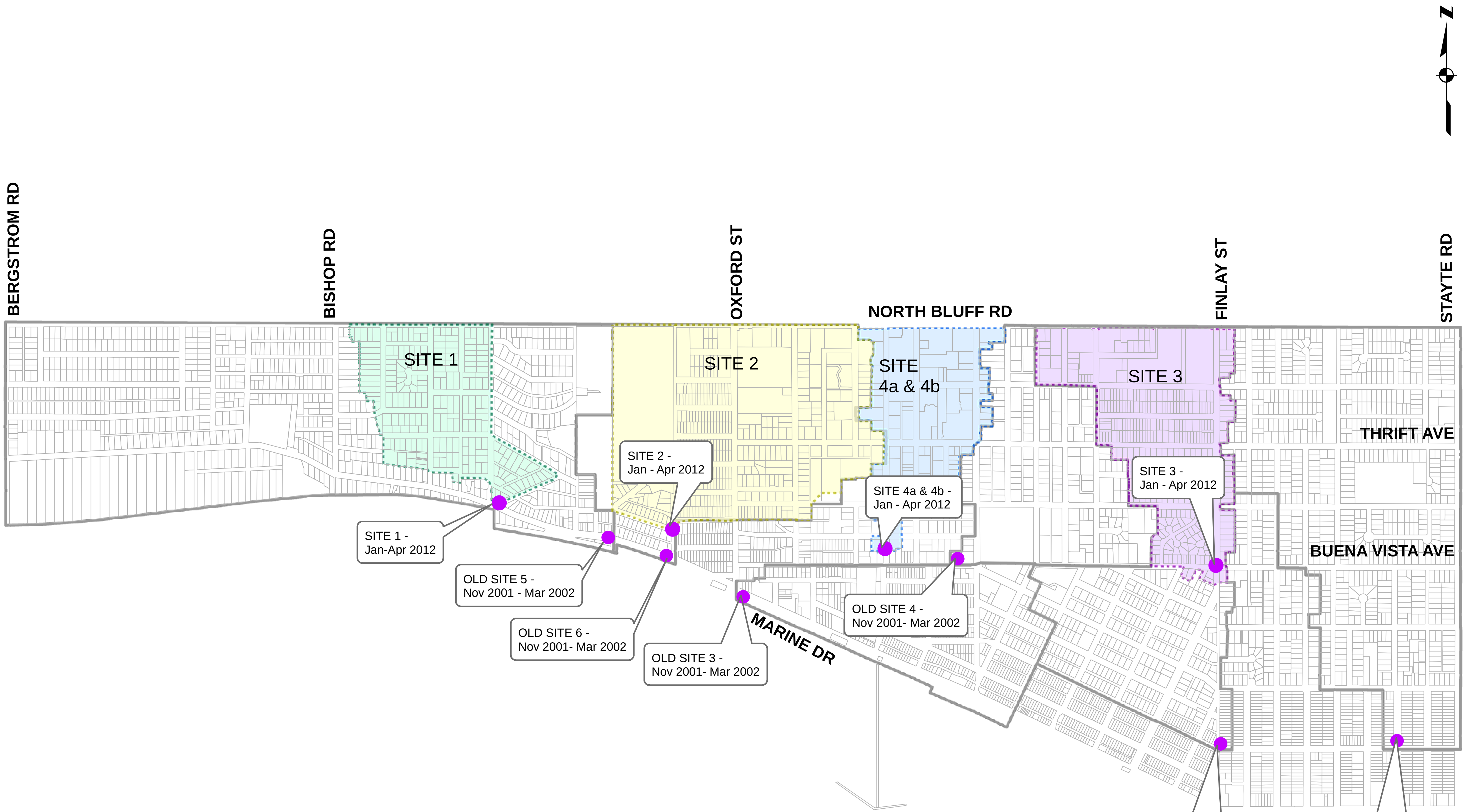
* 4b installed as backup for 4a

Flow data was received for the months of January, February, and March of 2012 and data quality was reviewed at the end of each month by both SFE and AECOM. At Sites 1, 2 and 3 the data was of good quality while at Sites 4a and 4b the data recorded was quite variable. At Site 4a the dry weather data was of adequate quality but the wet weather data was deemed unusable. Although Site 4b was installed as a backup meter for Site 4a the dry weather flow rate at 4b was an order of magnitude greater than the more realistic values recorded at 4a such that this data was also deemed unusable. Reasons for the poor data at sites 4a and 4b are due to the steep pipe slope and turbulent flow conditions particularly at the 4b manhole site. In such case, the dry weather calibration efforts focused on using data from all four sites while the wet weather calibration and validation was completed using data from Sites 1, 2 and 3 only. Detailed results from the flow monitoring program are discussed in **Sections 2.5** and **2.6**.

Historical flow monitoring data was also available for six locations in the City dating back to 2001. Summary information for these sites is provided in the *Sanitary Sewer Rehabilitation Program Audit Update – 2011 Draft Report* by KWL. Historical flow monitoring data from "Old Site 1" for 2005 was used to further refine the dry and wet weather calibration. Data from the remainder of the sites was not used for calibration purposes, but available wet weather data

The City of White Rock

Sewer Master Plan Update



Legend

- Old Flow Monitoring Sites
- Flow Monitor Location
- Site 1 - Magdalen Cres at Marine Dr
- Site 2 - West Beach Ave & Anderson St
- Site 3 - Buena Vista Ave & Finlay St
- Site 4a - Martin St north of Buena Vista

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	July 2012

Flow Monitor Locations

Figure 2.3

from four of the historical sites (Old Sites 1, 2, 4 and 5) was reviewed and incorporated into the I&I analysis for this study and is presented in **Section 2.7**.

Flow data for the MV forcemain from a gauge located at Anderson Street and North Bluff Road was also reviewed and used for the model calibration and determination of average dry weather flows, GWI and I&I rates. This data provided an overall check for the entire City as all sewage within the City limits is pumped by MV.

2.5 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 and future scenarios.

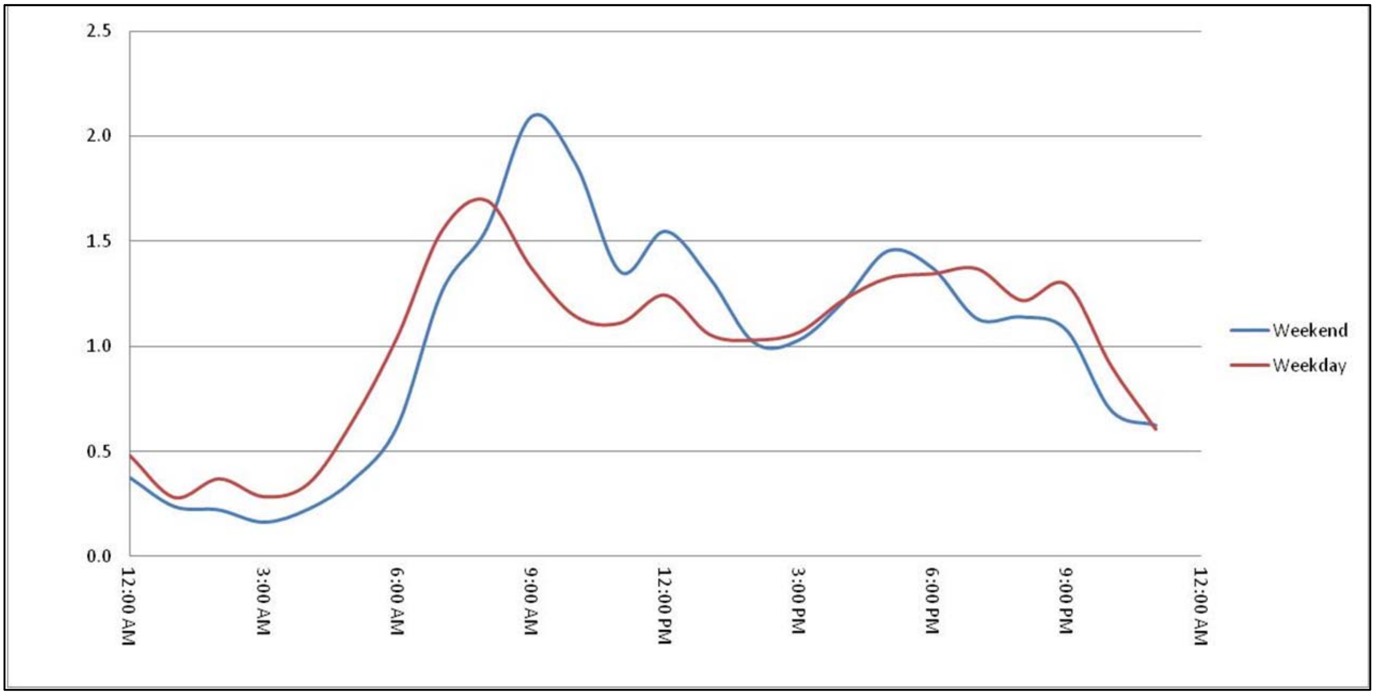
Sewage flows were generated on a parcel by parcel basis using the following key information:

- a property identification number for each parcel;
- populations derived using 2011 Census Block information and assigned to parcels based on land use; and
- water meter records from EPCOR for 2009 used for the institutional, commercial and industrial sewer loading.

Once the residential population was assigned to each parcel an estimate for sanitary flow was generated based on the dry weather flow monitoring data discussed below. The population and sanitary loading were then allocated to the nearest fronting sewer manhole using spatial join tools in GIS.

The diurnal curve for residential land use was developed using the flow meter data from Site 1 which is primarily a residential area. As shown in **Figure 2.4** the weekend pattern is has a higher peaking factor than the weekday pattern, in such case we have used the weekend pattern for the system capacity analysis.

Figure 2.4 Diurnal Pattern - Residential



A separate diurnal pattern was applied to the institutional and commercial land uses which was derived from previous sanitary modelling studies for Cities in the Lower Mainland.

The per capita sanitary flow rate for residential use was estimated using the dry weather flow data recorded during the 2012 monitoring period for Sites 1 to 4a (data from site 4b did not yield reasonable values and has not been included), flow data from “Old Site 1” and citywide flow data from the Metro Vancouver pump station (metered on the forcemain at North Bluff Road and Anderson Street). Using flow from consecutive dry days at each location, we were able to generate flow rates for base sanitary flows, night time low flows and groundwater infiltration (GWI) to estimate a unit rate for the sanitary flow. The GWI was initially estimated to be 80% of the night time low flow, and refined during calibration. The institutional/commercial/industrial (ICI) sanitary flow was estimated to be 85% of the ICI water meter records.

As shown in **Table 2.6** below, the average sanitary flow rate ranged from 115 to 458 L/cap/day which is within the typical expected range with the overall average of 210 L/cap/day at the MV forcemain. For comparison, the average water consumption based on the 2009 data from EPCOR is approximately 430 L/cap/day (including all meters regardless of land use type), and our recent work in North Vancouver and Surrey found rates in the range of 220-260 L/cap/day.

Table 2.6 Dry Weather Flow Summary

Site	Location	Area (ha)	2011 Population	ADWF (L/s)	GWI (L/s)	GWI (L/Ha/day)	ICI flow (L/s)	Residential Flow (L/s)	Sanitary Flow (L/cap/day)
Site 1	Magdalen/ Marine	16.9	552	3.3	1.5	7,692	0	1.8	283
Site 2	Anderson/ W. Beach	43.1	2,566	21.2	6.1	12,185	1.5	13.6	458
Site 3	Finlay/ Buena Vista	26.4	1,893	14.1	4.4	14,368	2.2	7.5	343
Site 4a	Martin/ Prospect	16.5	1,840	6.6	2.7	13,961	1.5	2.5	115
Old Site 1	Columbia/ Habgood	61.0	2,479	10.2	3.7	5,259	0.7	5.8	204
GVRD Forcemain		356.2	19,238	75.1	16.3	3,960	12.7	46.2	210

The GWI estimate for each monitored area is shown in **Table 2.6**. GWI is 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;
- 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;
- number of service connections, as more connections result 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.

GWI also varies during periods of dry weather and wet weather as soil conditions tend to be saturated during wet-weather months, even during periods of little to no precipitation. The values summarized above represent wet weather GWI as the monitoring period was during winter months.

The calculated GWI ranges from 5,259 to 14,368 L/Ha/day for a weighted average value of 9,395 L/Ha/day across the 5 monitoring catchments (including Old Site #1). This is a relatively high value for GWI and higher than the previously reported values in the 2005 KWL report that averaged approximately 5,200 L/Ha/day. The elevated values recorded during the 2012 flow monitoring period may be a result of meter error or increased GWI during wet months. As a further check we reviewed the MV forcemain data to determine a City-wide average for GWI that resulted in 3,960 L/Ha/day which was estimated using data from August 2011.

Ultimately for sewer catchments that did not have flow data, the per capita rate and GWI rate applied to the model was based on the city-wide MV forcemain data.

2.6 Model Calibration and Validation

The hydraulic model of the City's sewer system was calibrated for both dry and wet weather flow conditions and an additional wet weather event was modeled as the validation event. Time series data for calibration and validation results are shown graphically in **Appendix A** and summarized in **Table 2.7** for four of the new flow monitoring sites, one historical monitoring site and the Metro Vancouver forcemain.

Table 2.7 Calibration and Validation Summary

Location		Dry Weather Calibration		Wet Weather Calibration			Wet Weather Validation		
		Average Flow (L/s)	Total Event Volume (m^3)	Peak Flow (L/s)	Average Flow (L/s)	Total Event Volume (m^3)	Peak Flow (L/s)	Average Flow (L/s)	Total Event Volume (m^3)
Site 1	Date	Mar 21-25th, 2012		Feb 20-26th, 2012 - 55.4mm			March 2-6th, 2012 - 31.8mm		
	Monitor	3.6	309	11.4	6.0	3,089	10.4	5.7	2,465
	Model	3.5	299	9.7	5.3	2,767	10.0	4.8	2,056
	% Error	-3.0%	-3.2%	-17.1%	-11.6%	-11.6%	-3.6%	-19.8%	-19.9%
Site 2	Date	Mar 21-25th, 2012		Feb 20-26th, 2012 - 55.4mm			March 2-6th, 2012 - 31.8mm		
	Monitor	21.8	1,884	50.5	24.3	12,603	43.1	22.4	9,666
	Model	22.1	1,911	43.8	25.0	12,976	49.2	24.0	10,385
	% Error	1.5%	1.4%	-15.3%	2.9%	2.9%	12.4%	7.0%	6.9%
Site 3	Date	Feb 3-6th, 2012		Feb 20-26th, 2012 - 55.4mm			March 2-6th, 2012 - 31.8mm		
	Monitor	12.9	1,111	31.6	13.2	6,830	32.3	13.1	5,683
	Model	13.0	1,118	24.5	14.0	7,259	26.7	13.7	5,906
	% Error	0.7%	0.6%	-28.7%	5.9%	5.9%	-20.6%	3.8%	3.8%
Site 4a	Date	Feb 7 -11th, 2012		--			--		
	Monitor	6.1	529	--	--	--	--	--	--
	Model	6.4	554	--	--	--	--	--	--
	% Error	4.6%	4.5%	--	--	--	--	--	--
Old Site 1	Date	Jan 10-14th, 2005		Jan 15-19th, 2005 - 128.2mm			Dec 24-27th, 2004 - 45.0mm		
	Monitor	10.7	926	75.2	29.1	8,790	31.6	15.6	4,042
	Model	10.8	932	68.2	31.0	9,363	32.4	17.8	5,389
	% Error	0.6%	0.6%	-10.1%	6.2%	6.1%	2.6%	12.5%	25.0%
MV FM	Date	March 4-6th , 2010		Dec 1-7th, 2007 - 117.4mm			Jan 7-14th, 2010 - 121.6mm		
	Monitor	75.1	6,583	366.4	185.1	47,987	372.3	126.0	87,122
	Model	88.4	7,634	365.2	197.2	51,119	361.1	134.4	92,871
	% Error	14.9%	13.8%	-0.3%	6.1%	6.1%	-3.1%	6.2%	6.2%

Dry weather flow for the meter data and model predicted values are compared using the average flow for a 5 day period. As noted in the table above, the model predicted values are within 10% of the meter data for all sites with exception of the MV forcemain. The model predicted MV forcemain total volumes are higher than metered because we have assumed a higher GWI than the overall City-wide average as we blended the this value with the data from the temporary flow monitoring sites.

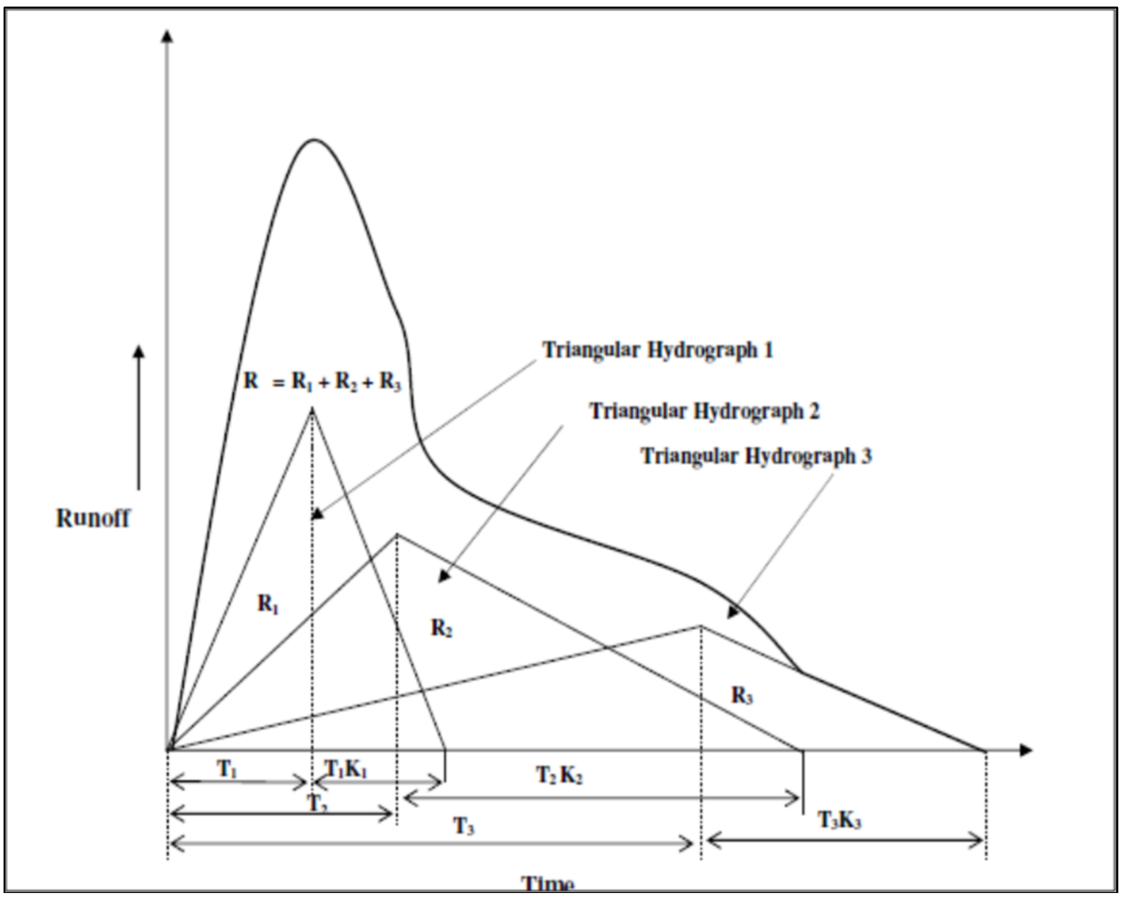
For the wet weather calibration and validation a comparison of the peak flow and volume for the metered data versus the modelled data for each flow monitoring site is provided. Peak flow values were difficult to compare for some sites due to spikes in the data. Overall, the wet weather calibration results are acceptable.

To complete the wet weather model calibration values were input for rainfall-derived infiltration and inflow (RDII). XPSWMM uses the RTK approach, whereby:

- “R” represents the percentage of rainfall that enters the sanitary sewer system;
- “T” represents the time from the onset of rainfall to the peak of the unit hydrograph in hours; and
- “K” represents the ratio of time to recession of the unit hydrograph to the time to peak.

The RTK approach has three such unit hydrographs with R1, R2 and R3 being the relative amounts that occur at various stages (i.e. fast, medium and slow response). The RTK technique is illustrated in **Figure 2.5.**

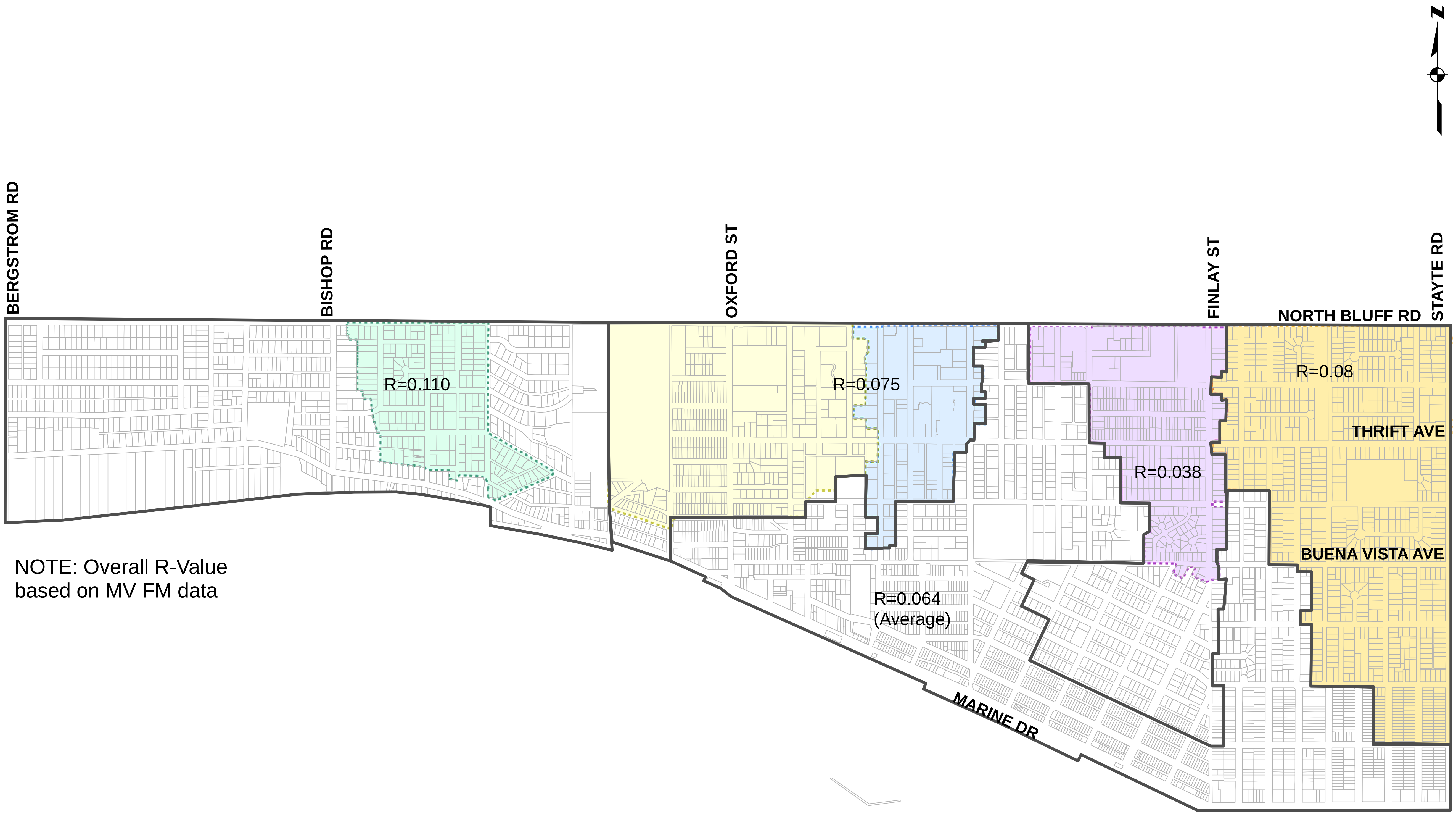
Figure 2.5 – RTK Parameter Description



The "R" values derived from the temporary flow monitoring sites are representative of events in the range of a 6-month to 5-Year return period while the overall "R" values derived from the White Rock Pump Station data are representative of a 50-Year return period event because the City-wide modelled was calibrated to the December 2007 storm event which was approximately a 50-Year 24-Hour storm event. Typically, as the return period increases so should the "R" value, and ultimately the "R" value will plateau once the return period becomes too large / infrequent.

A summary of the total "R" values derived for each flow monitor area and applied in the model is shown in **Figure 2.6**. Calibrated total "R" values were applied to each flow monitoring catchment area and the overall calibrated "R" value for the White Rock Pump Station was applied to the remainder of the City. Other factors that can impact the "R" values include pipe age, material and condition information (i.e. whether rehabilitation had occurred). Further refining of the calibration could be completed with additional flow monitoring data in subsequent years as required.

With the calibrated XPSWMM model, the City has the ability to input various rainfall events (historical and design storms) to assess peak flows and RDII. RDII in sanitary sewer systems is a major source of operating problems, causing poor performance of many sewer systems and is often the main cause of SSOs (Sanitary Sewer Overflows) to customer basements, streets, or nearby streams.



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Legend

- R-Value Areas Total R = (R1+R2+R3)
- Site 1 - Magdalen Cres at Marine Dr
- Site 2 - West Beach Ave & Anderson St
- Site 3 - Buena Vista Ave & Finlay St
- Site 4a - Martin St north of Buena Vista
- Old Site 1 - Columbia & Hasgood

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	August 2012

Model Calibration R-Values

Figure 2.6

2.7 Inflow and Infiltration Analysis

In order to evaluate the RDII rates for the City, we reviewed the historical monitoring data for Old Sites 1, 2, 4, and 5 (shown in **Figure 2.3**) as the storm events recorded at these sites were significantly greater than those recorded during the recent flow monitoring period from January to April 2012. Data from the Metro Vancouver forcemain was also reviewed to estimate I&I rates for the City. The resulting I&I rates are summarised in **Table 2.8** and the graphs used to generate the table are provided in **Appendix B**.

For this analysis, we determined that a logarithmic projection for RDII rates produced the “best fit” curve, with the least “R- squared” value, for the temporary flow monitoring data. This theory correlates with the fact that as the return period increases so should the RDII value and ultimately the RDII value will plateau once the return period becomes too large / infrequent. This methodology is different from the 2005 KWL Study where the system assessment was completed using a linear regression and extrapolated to a 100-Year event. While the linear regression and Envelope Method is widely used it can often result in elevated projections of RDII rates for 50-Year and 100-Year return periods.

Included in the table are the RDII rates for the 5-Year and 50-Year events. We have selected these return periods as they provide a range of values from the previous Metro Vancouver I&I target for the 5-Year return period of 11,200 L/Ha/day, and the 50-Year return period was determined to be an acceptable frequency that one may anticipate localized sewer backup to occur balanced with financial capital cost for system upgrades compared to larger events.

The GWI rate applied is the same as previously calculated in the 2005 KWL Study. These GWI rates are greater than the overall City-wide rate of 3,960 L/Ha/day determined from the White Rock Pump Station data but are less than the GWI rates estimated from the recent flow monitoring program as shown in **Table 2.6**.

Table 2.8 Summary of RDII rates per Return Period

Site #	GWI (L/Ha/day)	5-Year RDII (L/Ha/day)	50-Year RDII (L/Ha/day)	5-Year Total I&I (L/Ha/day)	50-Year Total I&I (L/Ha/day)
Old Site 1	6,500	30,100	36,500	36,600	43,000
Old Site 2	7,600	66,700	83,500	74,300	91,100
Old Site 4	4,300	44,100	51,900	48,400	56,200
Old Site 5	5,259	31,900	38,000	37,159	43,259
MV FM	3,960	28,687	42,156	32,647	46,116

3 Hydraulic Analysis

3.1 Design Criteria

Sanitary sewer design criteria is based on providing a level of service to the public and the City's current criteria is provided in Subdivision Bylaw # 777 Schedule B. The Bylaw states the ADWF is to be 360 litres per capita per day and the average "infiltration rate" is to be 0.06 L/s/Ha (or 5,184 L/Ha/day).

For the system capacity assessment, we have used 360 L/cap/day for the base sanitary flow loading. The peaking factor is derived from the diurnal curve (rather than the Harmon Peaking factor as per the Bylaw) for use in the hydraulic model. This provides the City with a slightly conservative model, a representative of local sewers, as compared to the "measured" average dry weather flow of 210 L/cap/day.

For the groundwater infiltration criteria, we have applied the GWI rates determined from the flow monitoring data for metered catchments and applied the Bylaw unit rate of 5,184 L/Ha/day for the remainder of the system. An overall GWI rate was determined from the Metro Vancouver forcemain data but was not used as it was slightly lower than the Bylaw rate and significantly lower than the smaller (more localized) metered subcatchment areas.

The peak wet weather criteria was determined to be the 50-Year 2-Hour storm event to simulate the RDII component. The two hour event was selected based on a review of the 1, 2, 6, 12 and 24 hour storm durations as it resulted in the most severe (highest) flows in the sewer system.

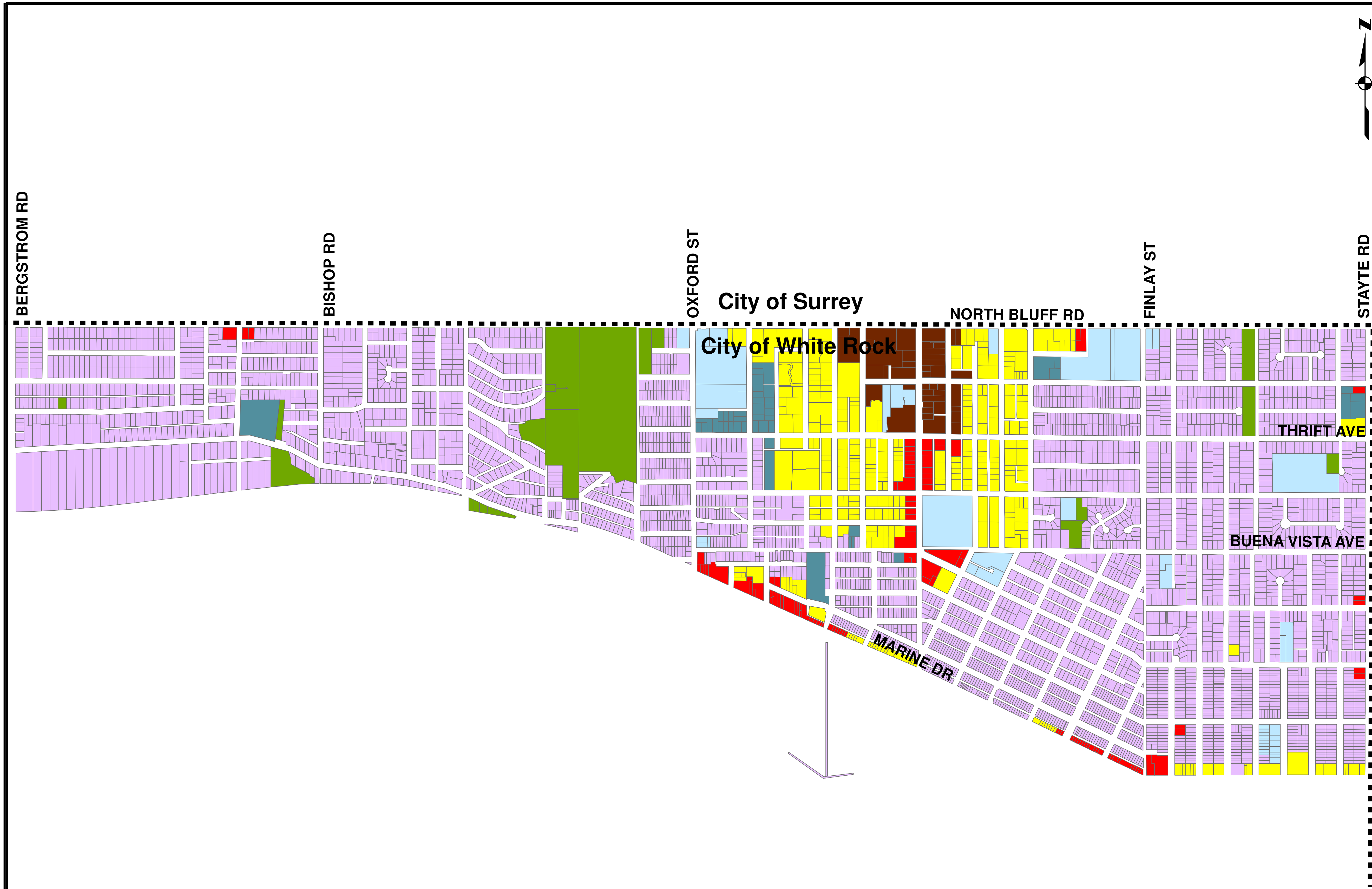
3.2 Existing and Future Land Use

Existing land use within the City is predominantly low density residential with pockets of medium density residential land use and the commercial core area being the Town Centre Area and Marine Drive. As per the OCP, the following land-use types are present within the City:

- Detached or Attached Residential (Low Density)
- Multi-Unit Residential (Low Density)
- Multi-Unit Residential (Medium Density)
- Commercial
- Open Space and Recreation Areas
- Institutional and Utility
- Town Centre Mixed Use

Future residential development will take the form of townhouses and apartments due to restrictions on land availability with the bulk of the development occurring in the Town Centre and North Bluff Areas. The future 2031 population is projected to be 23,500 based on the 2008 OCP and timing of development activity is dependent on market conditions.

In addition, commercial development is anticipated to increase as opportunities arise in the Town Centre and the residential population increases. There is also incremental redevelopment and infill activity in areas outside of the Town Centre that will be ongoing. Future land use is shown in **Figure 3.1**.



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Legend

- Detached/Attached Residential (Low Density)
- Multi-Unit Residential (Low Density)
- Multi-Unit Residential (Med Density)
- Commercial
- Open Space and Recreation Areas
- Institutional and Utility
- Town Centre Mixed Use

0 100 200 400 600 800
Meters

Project No.	Date
60238740	June 2012

OCP Land Use

Figure 3.1

3.3 System Capacity Assessment

The primary goal for the hydraulic capacity assessment is to develop a 10-Year Capital Plan for the City based on the future OCP land-use scenario. In such case, the model was utilized to perform an analysis of the hydraulic capacity for the existing sanitary sewer system under future OCP land use conditions.

The hydraulic analysis of the sewer system was based assuming that the available pipe capacity is 80% of the theoretical pipe full capacity. In such case a ratio of Q max / Q capacity of 0.8 or greater would trigger a pipe upgrade. While the 80% threshold is not particularly conservative, it is acceptable within the industry and was selected to reflect a realistic Capital Plan for the City. The forcemain velocity threshold of 3.0m/s was selected as velocities greater than this value result in increased and undesirable headloss rates in forcemains.

Figure 3.2 shows the existing sanitary sewer system capacity assessment for the PWWF using a 50-Year I&I rate under future OCP population and land use conditions. The sewer system results are highlighted as follows:

- sewer capacity as the ratio of peak flow (Qmax) to pipe capacity (Qcap):
  - Qmax/Qcap <0.5
  - Qmax/Qcap >=0.5 to <=0.8
  - Qmax/Qcap greater than 0.8
- hydraulic grade line (HGL) at manholes or model nodes includes in indication of whether the manhole is surcharging to surface
- forcemains and siphon velocity:
  - max velocity < 3.0 m/s
  - max velocity > 3.0 m/s

Summary tables for the length of sewers under capacity, number of surcharged MHs and velocity of forcemain/siphon are provided below.

Table 3.1 Sewer Capacity Summary

Qmax / Qcap Ratio	# of Sewers	Total length (m)
0-0.5	1,320	69,510
0.5 – 0.8	55	3,269
0.8 – 1.0	30	1,935
Greater than 1.0	28	1,478

As shown in **Table 3.1**, approximately 1,478 m of the City’s 76.2 km (or 1.9%) of sanitary sewers are undersized for the 50-Year peak wet weather flow and are likely to surcharge (i.e. Qmax/Qcap > 1.0). An additional 1,935 m of sewers have a peak wet weather flow resulting in a Qmax/Qcap ratio greater than 0.8 for a 50-Year, 2-Hour event.

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Legend

Pump Stations
- PWWF < Capacity
- PWWF > Capacity

Manhole HGL
- > 1.2 m below ground
- < 1.2 m to Ground
- Above Ground

Pipe Capacity, Qmax/Qcap
- 0 - 50%
- 50 - 80%
- > 80%

Forcemain/Siphon Velocity
- 0.0 - 2.0 m/s
- 2.0 - 3.0 m/s
- > 3.0 m/s

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	August 2012

Existing System with OCP & 50-Year, 2-Hr Event

Figure 3.2

Table 3.2 summarizes the number of manholes where the sewage level is predicted to reach the ground surface. A field review of the manholes predicted to have HGLs above ground should be completed to determine whether there is evidence of the manholes surcharging at these locations.

Table 3.2 Number of Flooded MHs

Depth Below Ground	# of Manholes
>1.2m below ground	1,323
<1.2m to ground	134
Above ground	9

A summary of the model predicted velocities in the forcemains and siphon is provided in **Table 3.3**. Velocity in the siphon ranged from 1.25 to 1.82 m/s depending on the location and corresponding pipe diameter for a PWWF of 290 L/s. The values presented in **Table 3.3** for the forcemain/siphon velocities do not include the Metro Vancouver forcemain.

Table 3.3 Sewer Forcemain Velocity Summary

Velocity Range	Total Length
0 – 2 m/s	5,005
2.0 to 3 m/s	0
>3 m/s	0
Total	**5,005**

A review of the pump station capacity was also completed and is shown in **Table 3.4** below which includes a comparison of model predicted peak wet weather flow (PWWF) versus the estimated pump station capacity. The estimated pump station capacity was determined from field measurements and over laying a system curve for each pump station on the theoretical pump curve.

Table 3.4 Sanitary Pump Station Capacity

Pump Station	Catchment Area (Ha)	Model PWWF (Inflow to Stations)	Estimated Pump Capacity Range
Keil	8.0	31.0 L/s	52 - 63 L/s
Ash	8.2	30.7 L/s	49 - 52 L/s
Bergstrom	*19.4*	*30.4 L/s*	*20 – 24 L/s*
MV PS	*356.2*	*521 L/s*	*370 L/s*

Based on the estimated pump capacity information presented above for the PWWF under the 50-Year event the Bergstrom Pump Station would be undersized. Although this pump station was not identified by City Operations staff as being problematic (beyond routine maintenance issues) it has been flagged as potentially undersized. We have also summarized the Metro Vancouver PS PWWF and capacity which is also undersized as shown in **Table 3.4**.

Further review of the pump station capacities and a detailed condition assessment are recommended and discussed in the capital plan and recommendations section.

Details for pipe capacities and proposed upgrades to meet the future OCP and 50-Year I&I peak wet weather flows are provided in **Section 5.0**.

4 Sewer Condition Assessment

To assist with the long range capital plan, we have performed a review of the available sewer condition assessment information provided by the City and a review of the available GIS pipe age attribute data. A geodatabase of CCTV coding and condition assessment results was analysed that included data compiled since 2001. Over the last 11 years the City has completed a number of sewer condition assessment and rehabilitation projects including CCTV investigations, smoke testing, point repairs, pipe lining, lateral connection grouting and investigation, and sewer replacement with the goal of reducing inflow and infiltration. A summary of the sewer rehabilitation works completed since 2001 is presented in the *Audit of Sanitary Rehabilitation Program* (KWL 2011) report.

4.1 Aging Infrastructure

As future development continues, the need to assess the condition of aging infrastructure becomes more critical. Upon review of the City's GIS data, only 4.3% or approximately 3.5 km of the sanitary sewer collection system did not have an entry for pipe age, and the earliest year of installation is noted as 1928.

A summary of the pipe age or year of installation is shown in **Figure 4.1** and in **Table 4.1**. In general, the following key items are required for a municipality to properly plan for existing infrastructure replacement:

- When was the sewer installed?
- What is the expected life cycle of the sewer?
- Is the asset technologically or commercially obsolete?

From available GIS data, **Table 4.1** categorizes the existing sewers into their corresponding year of installation.

Table 4.1 Summary of Pipe Year of Installation

Year Installed	Length of Pipe (m)
2012-2001	26,301
2000-1991	8,699
1990-1981	8,547
1980-1971	6,852
1970-1961	24,821
1960-1928	2,877
Unspecified	3,533
Total	**81,629**

As noted in the table above, there are approximately 2.9 km of sewer pipe at least 52 years old, and some pipes dating back 84 years which is beyond the life expectancy of most sewer pipe materials. **Table 4.2** below is a reference for typical sewer life cycle expectancy based on material type. A significant portion of the sanitary sewer system is between 42 to 51 years old and is most likely AC pipe. Once the oldest pipes have been replaced, the focus for sewer replacement and upgrades should be on the AC pipe.

One can draw a correlation between the age of pipes as shown in **Figure 4.1** and results of the condition assessment shown in **Figure 4.3** in the next section. A third parameter useful for the correlation of condition vs. age is the material

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Legend

- Unknown
- 1928-1966
- 1967-1980
- 1981-1990
- 1991-2000
- 2001-2008

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	August 2012

Existing Sanitary Sewer Year of Installation

Figure 4.1

types and this parameter can be found in the *Audit of Sanitary Rehabilitation Program* June 2011 Draft Report (KWL) as Figure 2-2 A & B.

Table 4.2 Theoretical Life Cycle of Sewer Based on Material

Material Type	Estimated Life Cycle (Years)
Asbestos Cement	50
Steel	80
Concrete	75
Ductile Iron	80
HDPE	80
PVC	80

4.2 Condition Assessment

Condition assessment data was provided by the City in the form of a geodatabase containing shapefiles compiled by KWL. The data is also presented in the *Audit of Sanitary Rehabilitation Program Draft Report* (KWL 2011). Upon review of the report and GIS information it appears that the majority of the sewers with a structural internal condition grade (ICG) of 5 (i.e. most likely to fail) have been replaced or repaired. A significant number of sewers with an ICG of 3 or 4 remain and have been highlighted for rehabilitation in this SMP report along with several point repairs for sewers with holes or major joint displacements.

The 2011 condition assessment report also highlighted sewers where inspections were either incomplete or abandoned. Reasons for the incomplete/abandoned inspections include the following items several of which are problems that should be rectified:

- debris in sewer,
- pipe too slippery for the camera,
- camera underwater,
- intruding connections,
- major root intrusion,
- large joint deflections,
- unidentified obstructions, and
- collapsed pipes (these appear to have been repaired).

A further review of the sewers identified as having inspections that were incomplete or abandoned, yielded the results shown in **Figure 4.2**. There are a significantly reduced number of incomplete sewer inspections in the current figure than those shown in the 2011 KWL report which is suspect. Further review of the CCTV data and sewer condition assessment results is required to determine where the discrepancies are.

A list of sewer rehabilitation projects including point repairs (external and internal) as well as full segment replacements are included in the Capital Plan and shown in **Figure 4.3**. A summary of the point repairs is shown in **Table 4.3** and is included in the Capital Plan as a line item along with each full pipe segment replacement. It should be noted that additional inspection on any pipes recommended for replacement or rehabilitation should be completed prior to initiating any works.

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Legend

Survey Completion Level

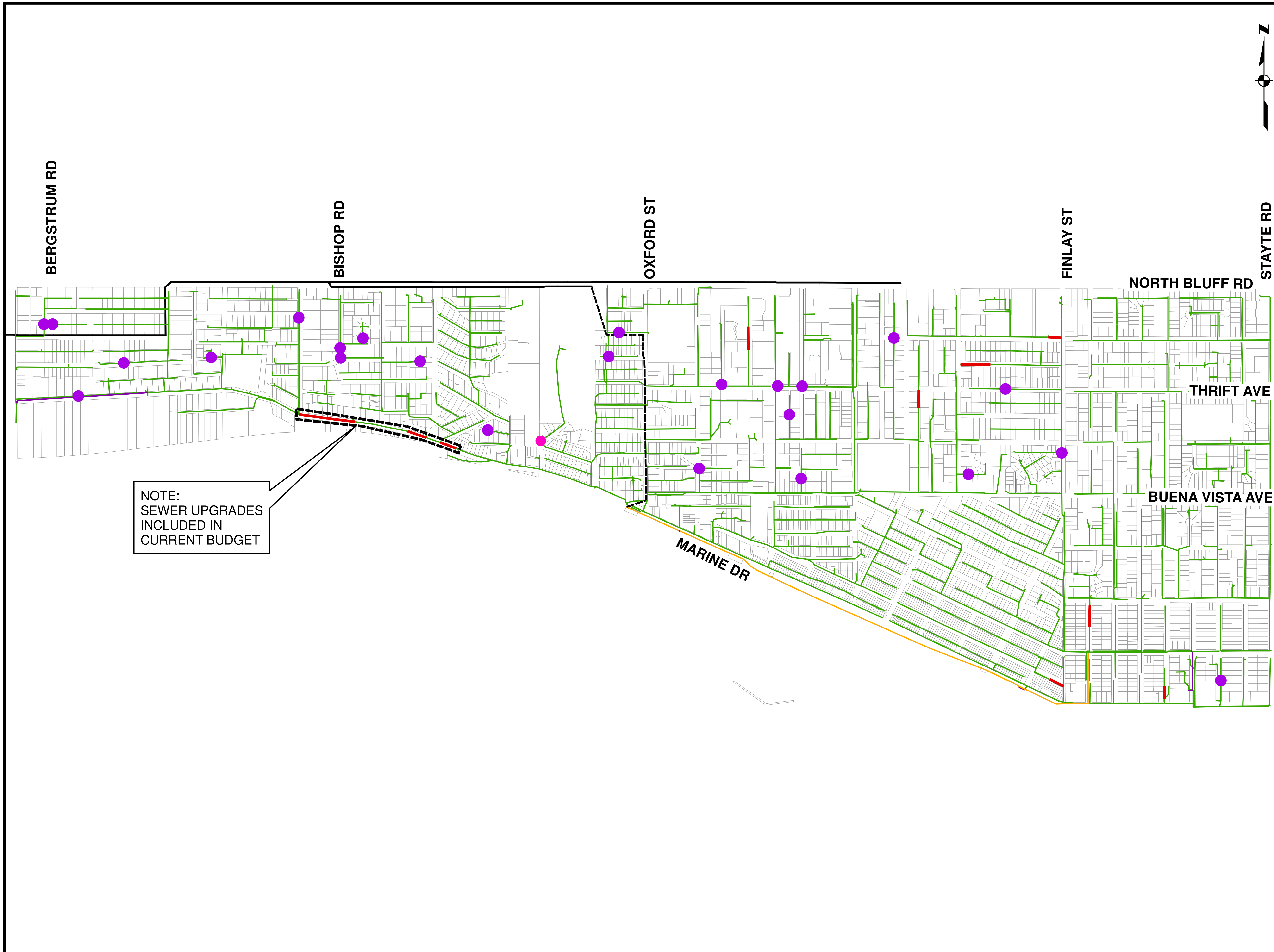
- Survey Abandoned
- Survey Inconclusive
- Survey Completed

0 100 200 400 600 800
Meters

Project No.	Date
60238740	July 2012

Incomplete/Abandoned CCTV Assessments

Figure 4.2



The City of White Rock

Sewer Master Plan Update

Project No.	Date
60238740	July 2012

Sanitary Sewer Recommended Rehabilitation Works

Figure 4.3

Table 4.3 Summary of Point Repairs

Location	Pipe ID	Dia (mm)	Defect	Proposed Rehab
1267 Finlay Street	585	250	Hole	TPR
15495 Thrift Ave	374	200	Joint Displacement / Separated	TPR
15420 Kyle Court	590	150	Hole	TPR
848 Habgood Street	109	200	Hole	TPR
15261 Russell Ave	306	200	Fracture / Joint Displacement	TPR
Intersection of Foster St and Thrift Ave	366	200	Hole	TPR
1424 Martin Street	32	200	Hole	TPR
Lane of 1360 Martin Street	441	200	Hole	TPR
14937 Thrift Ave	359	200	Hole	TPR
15080 Prospect Ave	39	200	Joint Displacement / Separated	TPR
Intersection of Prospect Ave and Everall St	612	200	Break	TPR
14759 Russell Ave	227	200	Hole	TPR
14733 Goggs Ave	300	200	Hole	TPR
14500 Sunset Lane	454	200	Hole	TPR
14366 Blackburn Ave	1151	200	Hole	TPR
14260 Park Ave	1116	200	Hole	TPR
14213 Malabar Ave	1141	200	Hole	TPR
1493 Phoenix Street	1140	200	Hole	TPR
1527 Bishop Road	1099	200	Hole	TPR
13965 Malabar Ave	1139	200	Hole	TPR
13801 Malabar Ave	1158	200	Hole	TPR
13721 Marine Drive	1205	200	Hole	TPR
13685 Blackburn Ave	1089	200	Hole	TPR
1521 Chestnut Street at MH	1090/1104	200	Hole	TPR
14607 West Beach Ave	482	200	Fracture	EPR

Note: TPR - Trenchless Point Repair, EPR - External Point Repair

5 Recommendations

This section summarizes our recommendations for the Sewer Master Plan after completing the hydraulic assessment and review of available condition assessment data. The focus for the recommendations is to meet the City's key issues and objectives outlined in **Section 1.2**.

5.1 Comparison with Previous Recommendations

The 2010 Capital Plan provided by KWL included several recommendations for major sanitary sewer upgrades and a proposed pump station that were designed for conveyance of the 100-Year Peak Hour I&I rates and to reduce the potential for sanitary sewer overflow (SSO) occurrence at the Metro Vancouver PS and elsewhere in the collection system. These previously proposed upgrades are discussed below along with a rationale as to how our analysis differs and why specific upgrades were modified or excluded from this Master Plan.

Reduction from 100-Year to 50-Year I&I Event

One of the most significant changes for the updated Master Plan is the use of a 50-Year event as opposed to 100-Year event that was previously used. Although the 50-Year event is less conservative, it was determined to be more realistic and achievable for White Rock given the limited tax base available for implementation or large scale capital projects. The 50-Year return period was also determined to be an acceptable frequency that one may anticipate localized sewer backup to occur balanced with financial capital cost for system upgrades compared to larger events.

This change is further rationalized given that Metro Vancouver's documented I&I target is for the 5-Year return period event (or 11,200 L/Ha/day) and there was no anecdotal evidence of reported overflows within the last 5 years from either the City or Metro Vancouver.

Oxford Street Pump Station & Diversion

A major upgrade that was proposed in the previous Capital Plan included a new diversion (from Johnston Street to Thrift Avenue) and pump station at Oxford Street and Thrift Avenue that would be tied into the Metro Vancouver forcemain. The diversion and pump station were intended to reduce flows to the Metro Vancouver Pump Station as it is currently undersized for future flow conditions. Several challenges were presented with siting of a new pump station at Oxford and Thrift included the tie-in to an aging AC MV forcemain, utility relocations, and provisions for overflows requiring connections to the gravity sewer system. The forcemain connection issue introduced the need to construct a new twin forcemain to convey flows up to the MV interceptor sewer at North Bluff Road and the associated costs for this infrastructure.

Upon further review of the proposed diversion and pump station at Oxford Street we confirmed that the major benefit was a reduction in flows at the MV Pump Station and there were limited improvements to the City's collection system. It was also determined that under the future conditions the MV Pump Station is still in need of upgrading even with a new Oxford Street Pump Station. In such case, we have not recommended that the City proceed with the new pump station or associated diversion as this would result is significant cost to the City that could be avoided by upgrading the MV Pump Station. It should also be noted that upgrades to the MV Pump Station would be funded by the Fraser Sewerage Area managed by Metro Vancouver and not directly by the City.

Temporary Diversion at Anderson Ravine

Another item is the temporary diversion of a sewer on Thrift Ave west of Oxford due to slope stability concerns in the Anderson Ravine. We recommend that the City purchase the land where the temporary sewer bypass is located west of Oxford between Thrift and an easement to the south to alleviate risk of sewer failure in Anderson Ravine. The current ROW agreement expires in February 2013 and the City has noted that an agreement for purchase of the ROW could be reached and the cost is likely to be approximately $25,000 including the property negotiation fees.

Remaining capital projects on Finlay Street, Columbia Avenue and Marine Drive that were recommended in the previously Capital Plan are present in the current Plan along with new projects that have been identified.

5.2 Summary of Recommendations

The proposed recommendations and collection system upgrades are discussed below and shown in **Figure 5.1**. The proposed upgrades are based on the capacity assessment criteria and condition data discussed in **Sections 3** and **4**.

- There are upgrades noted in the vicinity of Marine Drive and Oxford Street and we recommend that the sewer and manhole inverts at this location be verified. During the model development there were four different sources of information (Metro Vancouver drawing, White Rock Siphon record drawings, City' GIS data and Hydra model information) that all provided differing data for the pipe/manhole inverts leading up to the connection with Metro's Pump Station. Ultimately the GIS and record drawing information was used for this Capital Plan Update but results in several reverse graded pipes at this location. Although unusual, this may be how the system is configured in reality and should be field verified by topographic survey of the sewers in this area.

- As noted above the City should purchase a ROW where the temporary sewer bypass is located west of Oxford, between Thrift and the easement to the south, to make this a permanent sewer and alleviate the risk of slope failure in Anderson Ravine.

- A review of the pump station capacities was completed based on a comparison of model predicted PWWF and estimated pump station capacity. The capacity analysis shows that Bergstrom Pump Station is undersized. Prior to initiating any works further assessment of the pump station capacity is required along with a pump station condition assessment for all three sanitary pump stations.

- City operations staff noted routine maintenance issues at the pump stations (particularly Ash Pump Station) due to grease build-up that should be addressed with a Sewer Bylaw amendment for source control including use of garburators and grease traps. A complete review of the Sewer Bylaw and development of a means to reduce grease build up in sewers at sources thus reducing overall maintenance requirements and likelihood of sewer blockages is required.

- Further to the recommendation above, the Sewer Bylaw is in need of updating to include current I&I rates as well as an updated list of approved pipe materials.

- Although the City has completed CCTV assessment on the majority of the system, there is a significant length of sewer pipe where the surveys were either abandoned or incomplete. The reasons for the incomplete sewer inspections (extracted from the GIS data) are listed in **Section 4.2** and these locations should be revisited.

- There are a significant number of sewers with an ICG of 3 or 4 and most of these have been highlighted for rehabilitation in this SMP report along with several point repairs for sewers with holes or major joint displacements as noted in **Table 4.3**. The City should re-CCTV all proposed locations prior to any works.

- The City should conduct condition assessment on approximately 10% or 8 km of sewers each year. The results of the CCTV inspections should be reviewed by a qualified consultant to determine the rehabilitation works required and priority that they should be completed in. In addition to sewers, the manholes should also be inspected for both structural and service defects.

- Operation and maintenance of the Siphon could be enhanced by reinstating the water level monitor for the siphon at Maple Street and Victoria Ave. We understand there was an Operations Manual for the Siphon that was completed back in 2000 when the project was completed and this document should be reviewed to assess what maintenance measures are in place and could be improved upon.

The City of White Rock

Sewer Master Plan Update

Legend

Prop Dia — Proposed Upgrade

Exist Dia — Existing Sewer

Exist Dia — Upgrade Based On Condition

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	July 2012

Proposed Upgrades

Figure 5.1

6 10-Year Capital Plan

This section details the 10-year capital plan for the City of White Rock to complete the proposed sanitary sewer improvements. The capital plan has also been divided into priority items in consideration of White Rock's allowable annual budget for sewer works. The cost estimates have been prepared based on unit rates and lump sum amounts in our possession and are in Year 2012 dollars. These costs include sanitary sewer and manhole replacement; and asphalt replacement with re-use of road gravel/structural material. Cost excludes service connection replacement to property line. A contingency allowance of 25% and an engineering allowance of 10% have been included for each year while HST is not included. The following table (**Table 6.1**) summarizes the unit cost used, and it is important to note these costs include sewers, manholes and road restoration.

Table 6.1 Unit Costs for Upgrades

Item / Description	Unit	Unit Cost
200mm sewer (c/w backfill & asphalt restoration)	m	$700
250mm sewer (c/w backfill & asphalt restoration)	m	$743
300mm sewer (c/w backfill & asphalt restoration)	m	$798
375mm sewer (c/w backfill & asphalt restoration)	m	$886
450mm sewer (c/w backfill & asphalt restoration)	m	$963
525mm sewer (c/w backfill & asphalt restoration)	m	$1,122
600mm sewer (c/w backfill & asphalt restoration)	m	$1,199
TPR/EPR	ea	$5,000

A summary of the total expenditure for capital improvements per year is provided in **Table 6.2** below.

Table 6.2 Summary of Capital Plan

Year	Approximate Length to be Replaced (m)	Cost Estimate
2013*	1,349	$ 1,645,800
2014	685	$ 878,400
2015	406	$ 588,000
2016	612	$ 820,900
2017	602	$ 684,500
2018 - 2023	854	$ 2,509,300
Total	**4,507**	**$ 7,126,900**

* 2013 upgrades include works on Marine Drive from High Street to Bishop Road

Keeping in mind that The City's allowable annual budget is less than $1M a year for sewer related capital improvements, we have prioritized the upgrades for each year for the first five years. All the remaining upgrades can be completed in the next five years. **Figure 6.1** shows the proposed sanitary upgrades based on the year required. A detail breakdown of the proposed capital improvements for each phase is shown in **Table 6.3.**

Table 6.3 Details of 10-Year Capital Plan

Item #	Project Location	Street/Location	Model ID	Exist Dia (mm)	Prop Dia. (mm)	Length (m)	50-Year Peak Flow (L/s)	Max Flow/Design Flow	Unit Rate ($/m)	Cost Estimate ($)
		Year 2013 - Replacement of Sewers based on Condition Assessment & Capacity								
1.1	Russell Ave at Finlay St	Russell Ave from Finlay St to 15521 Russell Ave	P-56	200	200	50.5	11.0	0.19	$ 700	$ 35,369
1.2	Blackwood Street	Blackwood Street from 1473 Blackwood St to 1521 Blackwood St	P-284	200	200	91.5	7	0.12	$ 700	$ 64,051
1.3	Goggs Ave at Best Street	Goggs Ave from 15460 Goggs Ave to Best St	P-234	200	200	115.5	3	0.06	$ 700	$ 80,857
1.4	Lane East of Fir Street from 1360 Fir St to Thrift Ave	Lane East of Fir Street from 1360 Fir St to Thrift Ave	P-414	200	200	69.0	7	0.11	$ 700	$ 48,300
1.5	Maple St south of Pacific Ave	Maple Street from 976 Maple St to 990 Maple St	P-927	200	200	24.0	28	0.19	$ 700	$ 16,800
		Maple Street from 948 Maple St to 976 Maple St	P-949	200	200	61.0	30	0.24	$ 700	$ 42,700
1.6	Victoria Ave at Finlay St	Victoria Ave from Finlay St to 15574 Victoria Ave	P-1028	200	200	60.0	1	0.01	$ 700	$ 42,008
1.7	Kent Street at Marine Drive	Kent Street from 15791 Marine Dr to 839 Kent St	P-1033	200	200	49.0	3	0.08	$ 700	$ 34,300
1.8	Marine Dr from Bishop to High St	*Marine Dr from High St to 14508 Sunset Dr*	*P-548*	*200*	*300*	*55.8*	*92*	*1.11*	*$ 798*	*$ 44,496*
		Marine Dr frrom 14508 Sunset Dr to 14478 Sunset Dr	*P-540*	*200*	*300*	*77.1*	*106*	*0.94*	*$ 798*	*$ 61,550*
		Marine Dr from 14478 Sunset Dr to 14436 Sunset Dr	*P-531*	*200*	*300*	*70.9*	*105*	*1.00*	*$ 798*	*$ 56,546*
		Marine Drive from 14436 Sunset Dr to Magdalen Cres	*P-506*	*200*	*300*	*66.0*	*105*	*1.31*	*$ 798*	*$ 52,676*
		Marine Dr from Magdalen Cres to 14391 Marine Dr	*P-1427*	*200*	*300*	*56.5*	*79*	*0.98*	*$ 798*	*$ 45,119*
		Marine Drive from 14391 Marine Dr to Brearley Si	*P-455*	*200*	*300*	*77.7*	*79*	*0.93*	*$ 798*	*$ 61,977*
		Marine Dr from Brearley St to 14310 Sunset Dr	*P-1224*	*200*	*300*	*84.4*	*79*	*1.27*	*$ 798*	*$ 67,327*
		Marine Dr from 14310 Sunset Dr to 14283 Marine Dr	*P-1223*	*200*	*300*	*48.8*	*84*	*0.86*	*$ 798*	*$ 38,903*
		Marine Dr from 14283 Marine Dr to 14249 Marine Dr	*P-1222*	*200*	*250*	*73.1*	*83*	*1.08*	*$ 743*	*$ 54,343*
		Marine Drive from 14249 Marine Dr to 14213 Marine Dr	*P-1221*	*200*	*250*	*79.0*	*82*	*0.92*	*$ 743*	*$ 58,705*
		Marine Drive from 14213 Marine Dr to 14205 Marine Dr	*P-1220*	*200*	*250*	*26.0*	*82*	*1.18*	*$ 743*	*$ 19,331*
		Marine Drive from 14205 Marine Dr to Bishop Ra	*P-1218*	*200*	*250*	*112.7*	*79*	*1.00*	*$ 743*	*$ 83,756*
1.9		Trenchless Point Repairs - Various Locations								$ 125,000
1.10		Purchase of ROW for Temporary Sewer Bypass - Anderson Ravine								$ 25,000
1.11		Annual CCTV Inspection (Approx. 8km of Storm Sewers)							$ 7.5	$ 60,000
									Sub-Total	**$ 1,219,115**
									Engineering 10%	**$ 121,911**
									Contingency 25%	**$ 304,779**
									Total	**$ 1,645,805**
		Year 2014 - Replacement of Sanitary Sewers based on Capacity								
2.1	Columbia Ave from Parker to Stayte Road	Columbia Ave from Kent St to 15827 Columbia Ave	P-977	250	375	62.6	116	1.06	$ 886	$ 55,437
		Columbia Ave from 15827 Columbia Ave to Keil St	P-978	200	375	39.2	115	1.05	$ 886	$ 34,740
		Columbia Ave from Keil St to Habgood St	P-8087	300	375	105.1	82	0.82	$ 886	$ 93,083
		Columbia Ave from Habgood St to Stevens St	P-980	200	375	88.5	78	1.32	$ 886	$ 78,438
		Columbia Ave from Stevens St to Stayte Rd	P-112	200	300	101.2	105	1.00	$ 798	$ 80,766
		Intersection of Columbia Ave and Parker St	P-8053	200	300	4.8	24	0.87	$ 798	$ 3,830
2.2	Buena Vista Ave from Johnston Rd to Best Street	Intersection of Buena Vista Ave and Johnston Rd	P-648	300	450	4.6	96	1.73	$ 963	$ 4,449
		Buena Vista Ave from Johnston Rd to 1273 Fir St	P-627	200	375	111.1	75	1.35	$ 886	$ 98,408
		Buena Vista Ave 1273 Fir St to 1225 Merkin St	P-628	200	375	116.1	75	1.41	$ 886	$ 102,882
		Buena Vista Ave from 15367 Buena Vista Ave to Best St	P-637	200	250	52.0	24	1.36	$ 743	$ 38,636
2.3		Annual CCTV Inspection (Approx. 8km of Storm Sewers)							$ 7.5	$ 60,000
									Sub-Total	**$ 650,669**
									Engineering 10%	**$ 65,067**
									Contingency 25%	**$ 162,667**
									Total	**$ 878,403**
		Year 2015 - Replacement of Sanitary Sewers based on Capacity								
3.1	Russell Ave near Finlay St	Intersection of Finlay St and Russell Ave	P-258	200	250	3.5	33	1.15	$ 743	$ 2,623
		Russell Ave at 15521 Russell Ave	P-253	200	250	6.2	26	1.06	$ 743	$ 4,629
3.2	Marine Dr from Anderson St to Oxford St	Marine Dr at 14780 Marine Dr	P-8112	450	600	5.9	262	1.05	$ 1,199	$ 7,074
		Marine Dr at 14780 Marine Dr	P-8111	375	450	23.4	141	3.16	$ 963	$ 22,486
		Marine Dr from 14780 Marine Dr to Oxford St	P-8113	450	600	37.9	262	0.84	$ 1,199	$ 45,454
		Marine Dr from 14780 Marine Dr to 14757 Marine Dr	P-8109	375	450	21.0	139	0.91	$ 963	$ 20,184
		Marine Dr 14757 Marine Dr to Anderson St	P-686	375	450	123.4	191	0.94	$ 963	$ 118,834
		Marine Dr at intersection of Marine Dr and Anderson St	P-622	200	450	17.0	190	0.45	$ 963	$ 16,400
		Anderson St from Marine Dr to 1209 Anderson St	P-8	200	300	44.7	91	0.87	$ 798	$ 35,687
3.3	Oxford St at Buena Vista Ave	Oxford St from Marine Dr to 1184 Oxford St	P-21	300	375	43.2	190	1.30	$ 886	$ 38,249
		Oxford St from 1184 Oxford St to Buena Vista Ave	P-687	200	300	28.5	81	0.91	$ 798	$ 22,743
		Buena Vista Ave from Oxford St to 14811 Buena Vista Ave	P-632	200	300	26.5	75	1.00	$ 798	$ 21,171
		Buena Vista Ave from 14811 Buena Vista Ave to 14831 Buena Vista Ave	P-633	200	300	25.1	75	0.95	$ 798	$ 19,998
3.4		Annual CCTV Inspection (Approx. 8km of Storm Sewers)							$ 7.5	$ 60,000
									Sub-Total	**$ 435,531**
									Engineering 10%	**$ 43,553**
									Contingency 25%	**$ 108,883**
									Total	**$ 587,968**

Table 6.3 Details of 10-Year Capital Plan

Item #	Project Location	Street/Location	Model ID	Exist Dia (mm)	Prop Dia. (mm)	Length (m)	50-Year Peak Flow (L/s)	Max Flow/Design Flow	Unit Rate ($/m)	Cost Estimate ($)
Year 2016 - Replacement of Sanitary Sewers based on Capacity										
4.1	Marine Dr from Oxford St to Vidal St	Marine Dr from Oxford St to Elm St	P-756	300	375	134.3	73	1.05	$ 886	$ 118,990
		Marine Dr from Elm St to 14881 Marine Dr	P-24	300	375	62.0	62	1.19	$ 886	$ 54,941
		Marine Dr frin 14881 Marine Dr to Vidal St	P-27	300	375	76.8	61	1.15	$ 886	$ 68,009
4.2	Marine Dr from High St to Anderson St	Marine Dr from Anderson St to 14661 Marine Dr	P-621	200	450	74.2	115	0.79	$ 963	$ 71,483
		Marine Dr 14661 Marine Dr to 14647 Marine Dr	P-608	200	375	28.9	114	0.73	$ 886	$ 25,588
		Marine Dr from 14647 Marine Dr to Bay St	P-600	200	375	109.9	113	0.79	$ 886	$ 97,327
		Marine Dr from Bay St to Duprez St	P-572	200	375	52.8	113	1.32	$ 886	$ 46,807
		Marine Dr from Duprez St to High St	P-6	200	375	73.3	110	1.65	$ 886	$ 64,944
4.3		Annual CCTV Inspection (Approx. 8km of Storm Sewers)							$ 7.5	$ 60,000
									Sub-Total	**$ 608,089**
									Engineering 10%	**$ 60,809**
									Contingency 25%	**$ 152,022**
									Total	**$ 820,921**
Year 2017 - Replacement of Sanitary Sewers based on Capacity										
5.1	Marine Dr east of Bishop Road	Marine Dr from Bishop Rd to 14123 Marine Dr	P-1217	200	250	56.6	69	0.93	$ 743	$ 42,032
		Marine Dr from 14123 Marine Dr to 14101 Marine Dr	P-8107	200	250	47.0	69	0.96	$ 743	$ 34,921
		Marine Dr from 14101 Marine Dr to 14093 Marine Dr	P-1213	200	250	17.1	60	0.91	$ 743	$ 12,698
		Marine Dr from 14093 Marine Dr to 14046 Marine Dr	P-1195	200	250	86.2	60	0.77	$ 743	$ 64,047
		Marine Dr from 14046 Marine Dr to 14008 Marine Dr	P-1193	200	250	71.3	60	1.15	$ 743	$ 52,961
		Marine Dr from 14008 Marine Dr to Nichol Rd	P-1054	200	250	39.6	57	1.78	$ 743	$ 29,453
		Marine Dr from Nichol Rd to 13937 Marine Dr	P-1192	200	250	104.0	43	1.25	$ 743	$ 77,235
		Marine Dr from 13937 Marine Dr to 13881 Marine Dr	P-1247	200	250	114.6	43	1.31	$ 743	$ 85,140
		Marine Dr from 13881 Marine Dr to 13857 Marine Dr	P-1267	200	250	65.4	30	0.83	$ 743	$ 48,570
5.2		Annual CCTV Inspection (Approx. 8km of Storm Sewers)							$ 7.5	$ 60,000
									Sub-Total	**$ 507,056**
									Engineering 10%	**$ 50,706**
									Contingency 25%	**$ 126,764**
									Total	**$ 684,525**
Years 2018-2023 - Replacement of Remaining Undersized Sanitary Sewers										
6.1	Sunset Dr from Brearley St to Kerfoot Rd	Sunset Dr from Brearley St to Kerfoot Rd	P-1215	200	250	100.3	23	0.88	$ 743	$ 74,486
6.2	Finlay St and Columbia Ave Intersection	Finlay St from Columbia Ave to 918 Finlay St	P-970	300	375	22.0	92	0.98	$ 886	$ 19,492
		Columbia from Maple St to Finlay St	P-8046	300	375	92.9	112	0.91	$ 886	$ 82,283
6.3	Pacific Ave from Stayte Rd to Stevens St	Pacific Ave from Stayte Rd to Stevens St	P-114	200	250	96.1	28	0.97	$ 743	$ 71,417
6.4	Stayte Rd from 1127 Stayte Rd to Buena Vista Ave	Stayte Rd from 1127 Stayte Rd to Buena Vista Ave	P-776	200	250	146.7	74	1.31	$ 743	$ 108,968
6.5	Finlay St from Goggs Ave to Roper Ave	Finlay St from 1341 Finlay St to 15590 Thrift Ave	P-451	200	250	100.0	58	0.95	$ 743	$ 74,300
		Finlay St 15590 Thrift Ave to Thrift Ave	P-79	200	250	54.0	52	0.93	$ 743	$ 40,122
		Finlay St from Thrift Ave to Goggs Ave	P-329	200	300	94.5	48	0.98	$ 798	$ 75,411
6.6	Roper Ave from Multi 52 to 15265 Roper Ave	Roper Ave from Multi 52 to 15265 Roper Ave	P-52	200	300	94.3	30	0.99	$ 798	$ 75,227
6.7	Property of Centennial Arena, Multi 76	Property of Centennial Arena, Multi 76	P-314	150	200	52.9	16	0.92	$ 700	$ 37,044
6.8		Bergstrom Pump Station Upgrade							$ 900,000	$ 900,000
6.9		Annual CCTV Inspection (Approx. 8km of Storm Sewers per year)							$ 7.5	$ 300,000
									Sub-Total	**$ 1,858,751**
									Engineering 10%	**$ 185,875**
									Contingency 25%	**$ 464,688**
									Total	**$ 2,509,313**

The City of White Rock

Sewer Master Plan Update

Legend

Proposed Upgrade

- 2013
- 2014
- 2015
- 2016
- 2017
- 2018-2023
- Existing Sanitary Sewer

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60238740	July 2012

Capital Projects

Figure 6.1

Appendix A

- **Dry and Wet Weather Model Calibration and Validation**

Figure 1a: Site 1 Dry Weather Flow Calibration

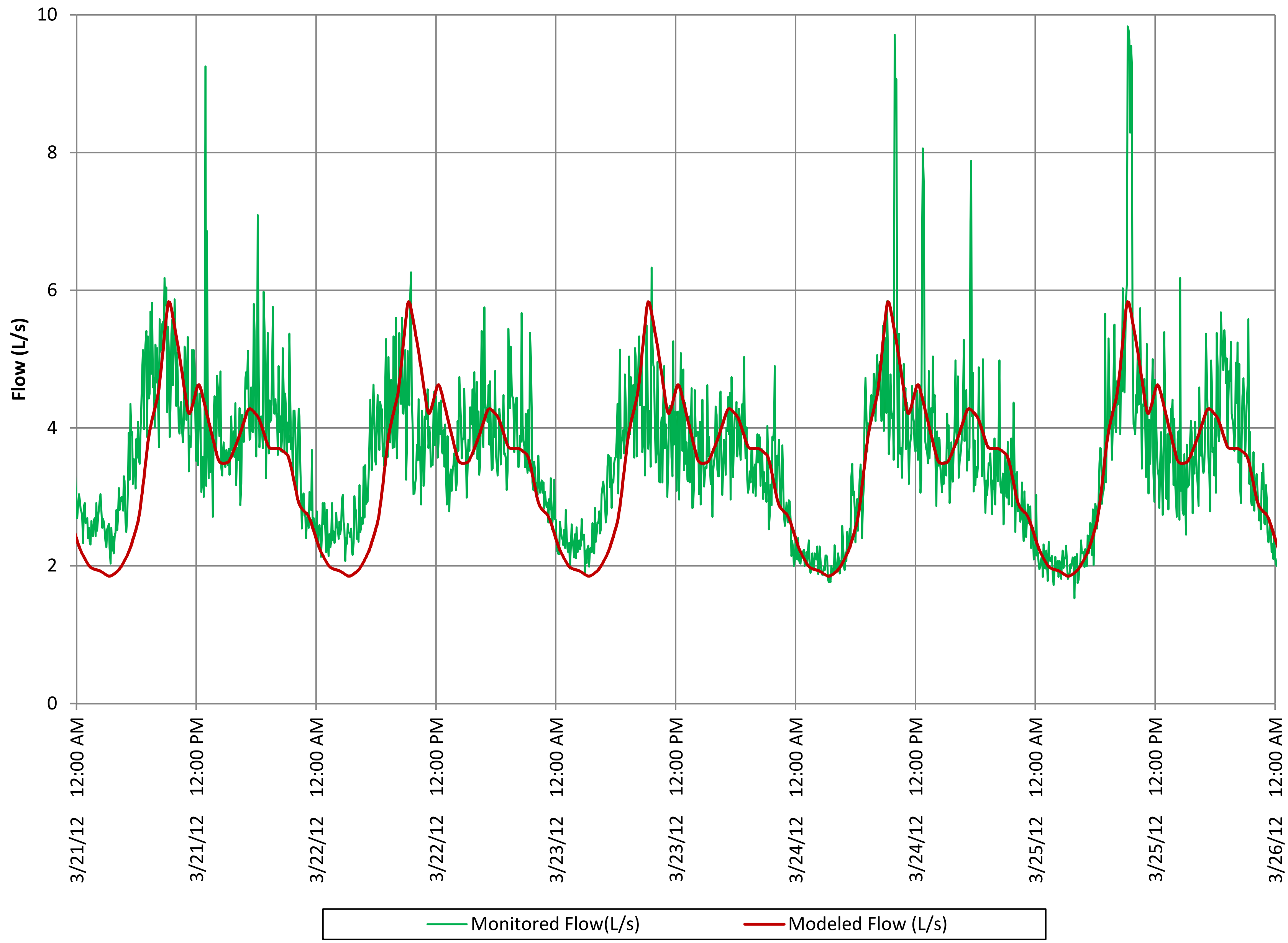


Figure 1b: Site 1 Wet Weather Flow Calibration

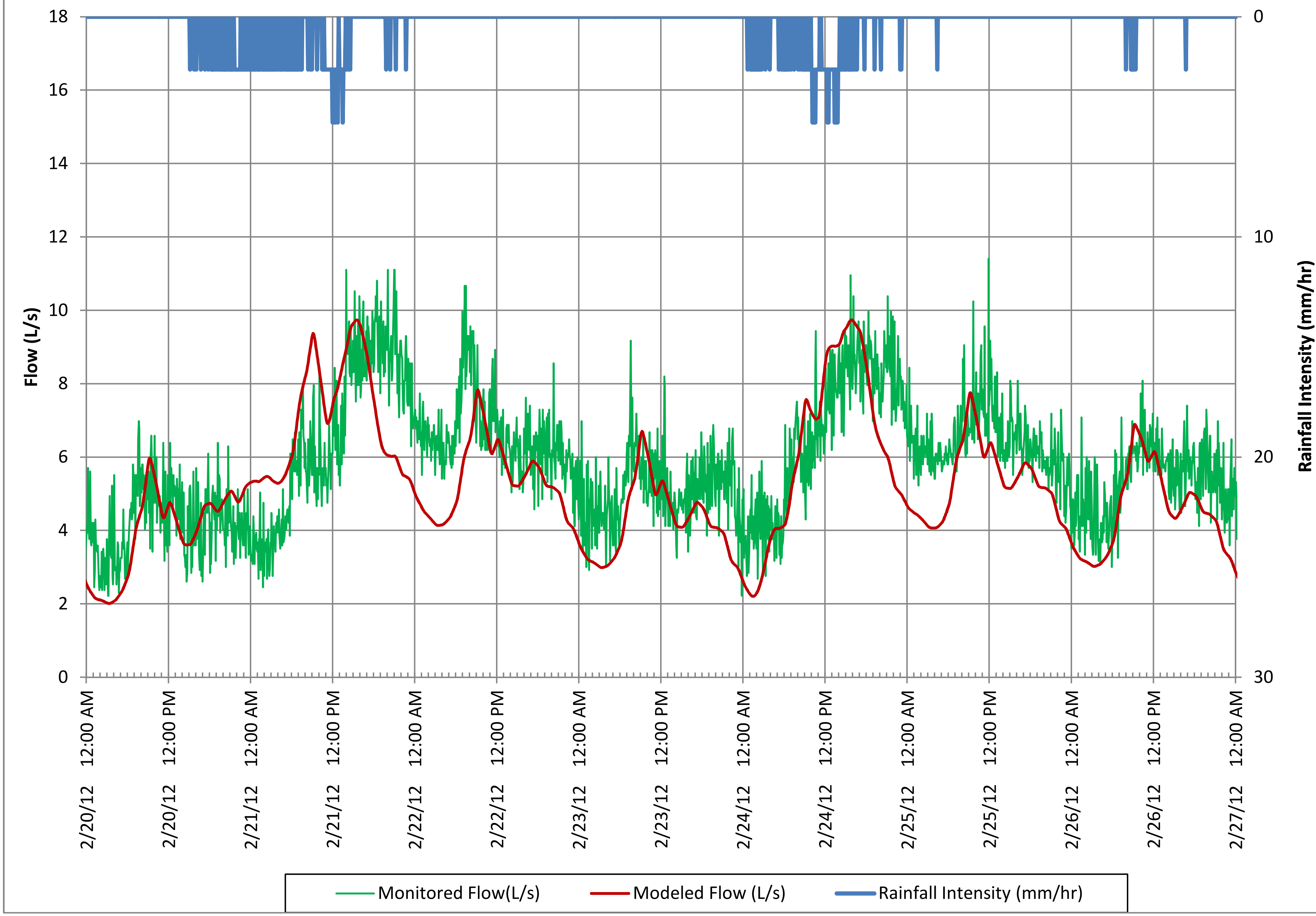


Figure 1c: Site 1 Wet Weather Flow Validation

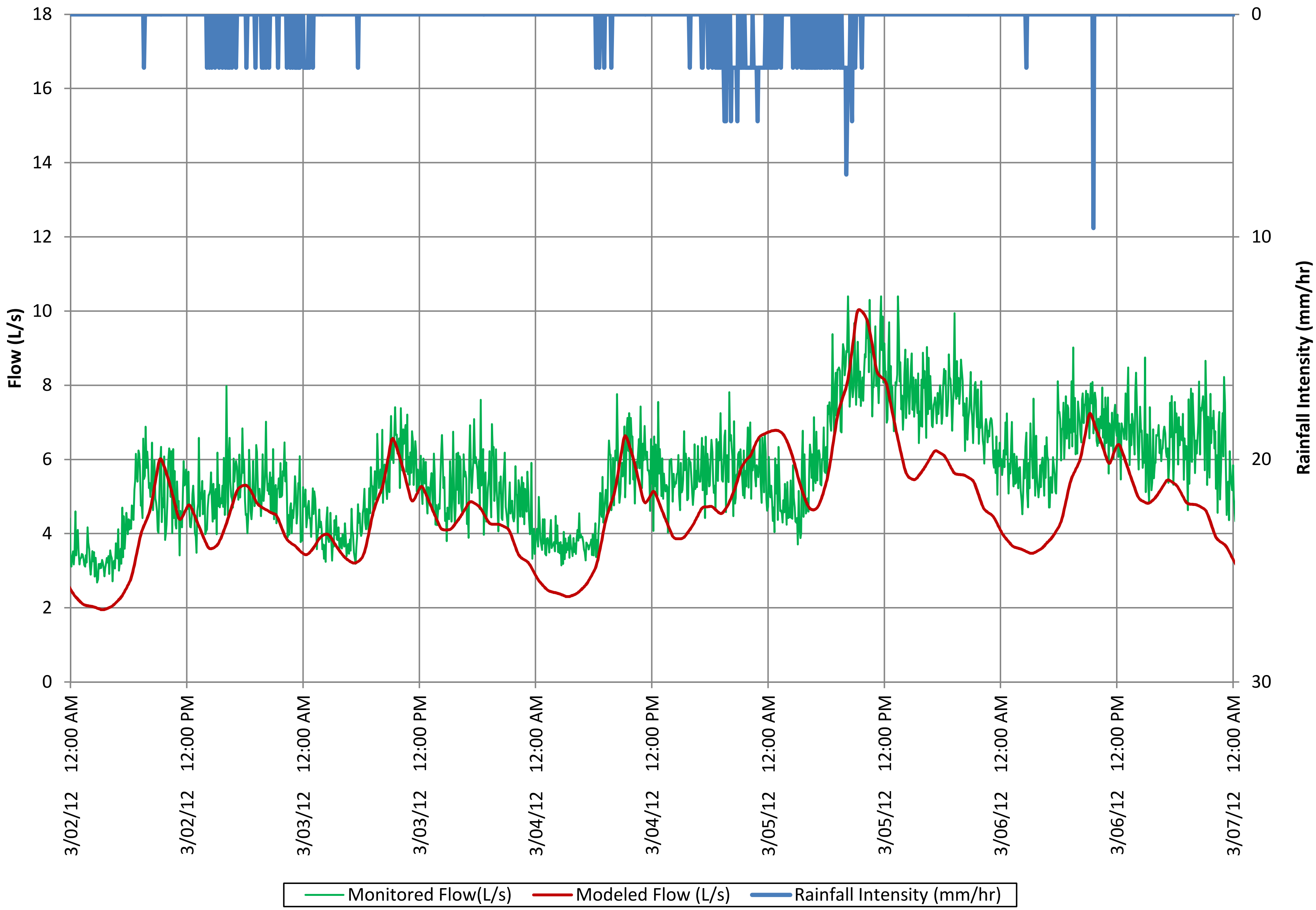


Figure 2a: Site 2 Dry Weather Flow Calibration

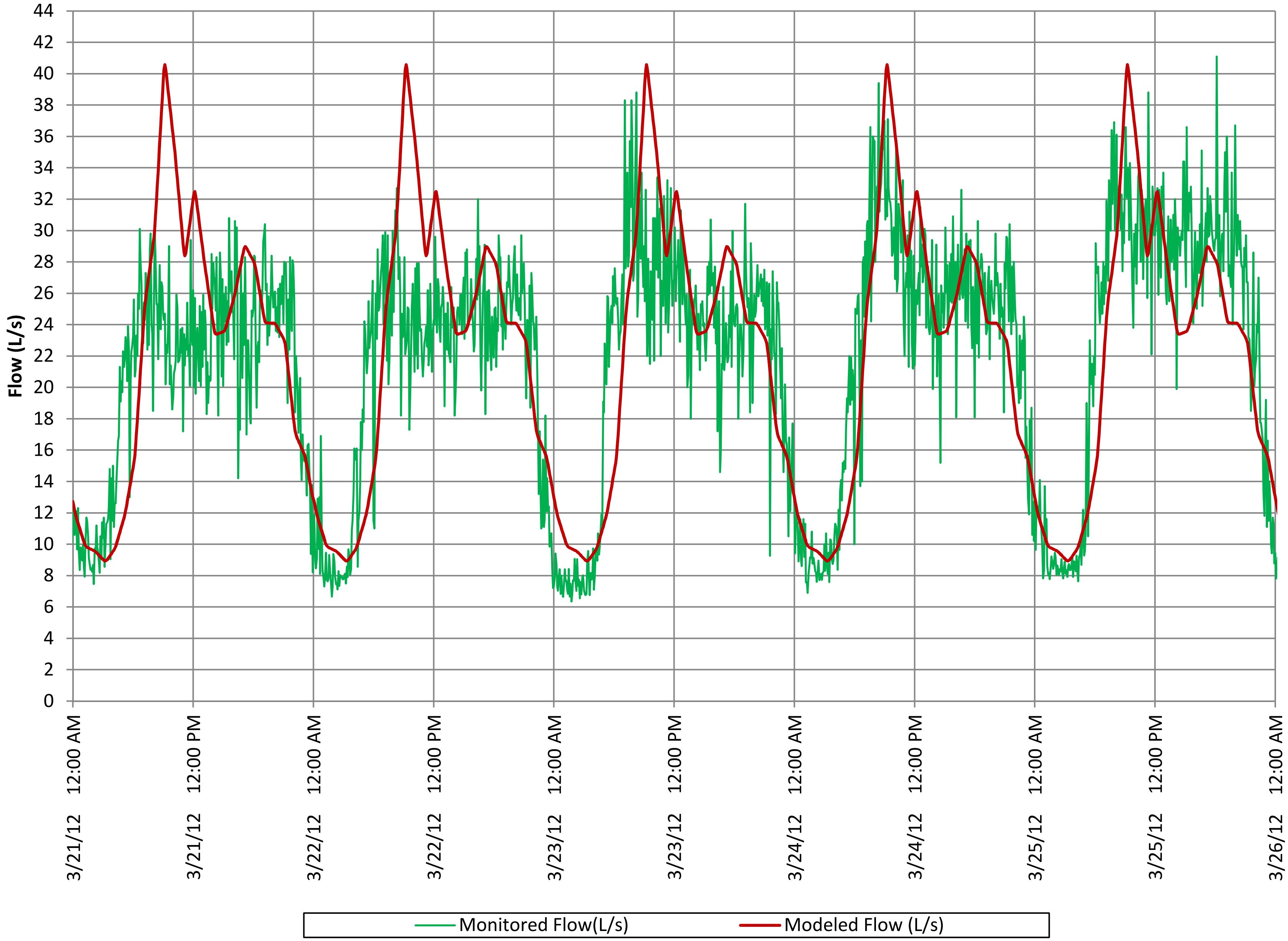


Figure 2b: Site 2 Wet Weather Flow Calibration

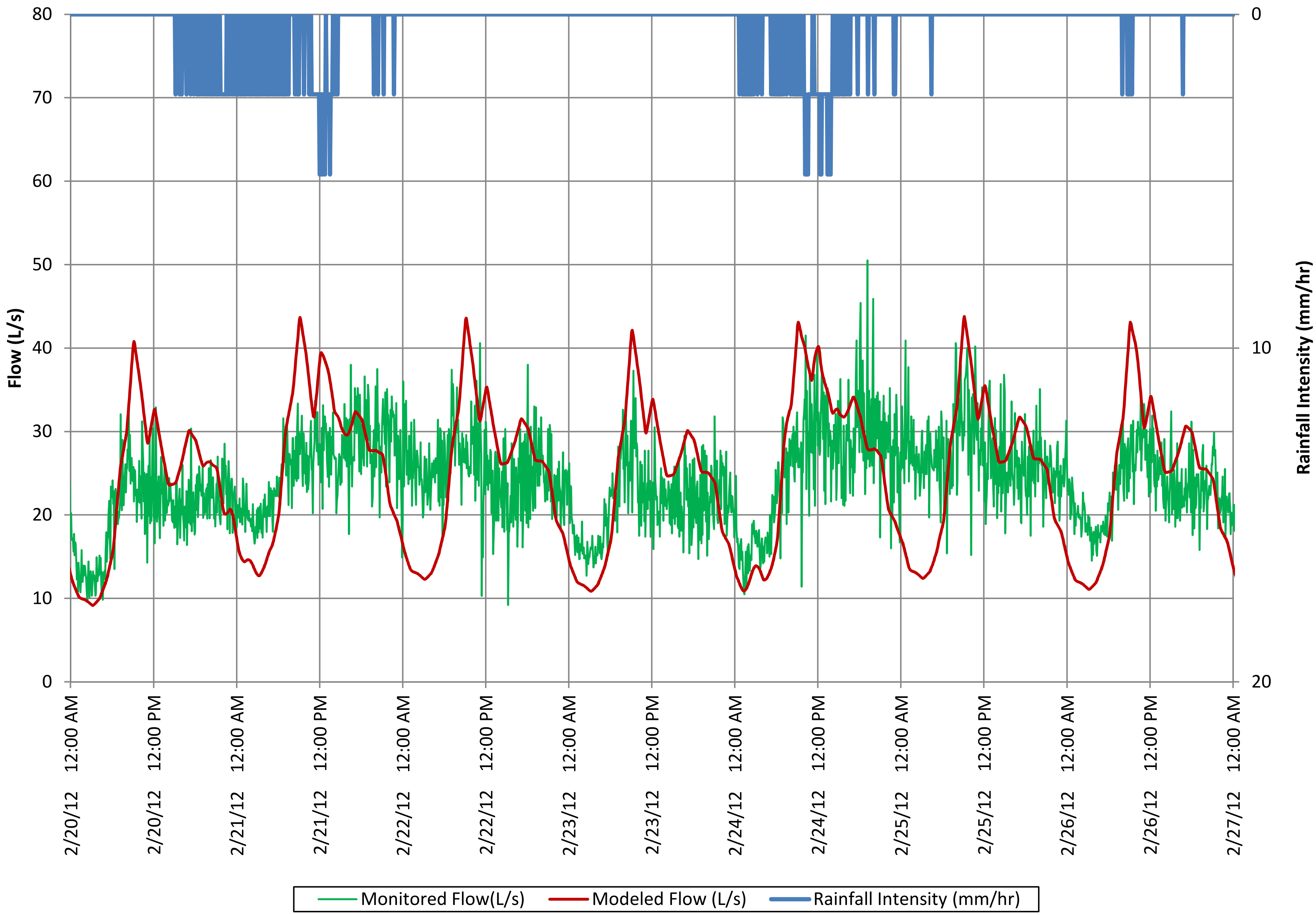


Figure 2c: Site 2 Wet Weather Flow Validation

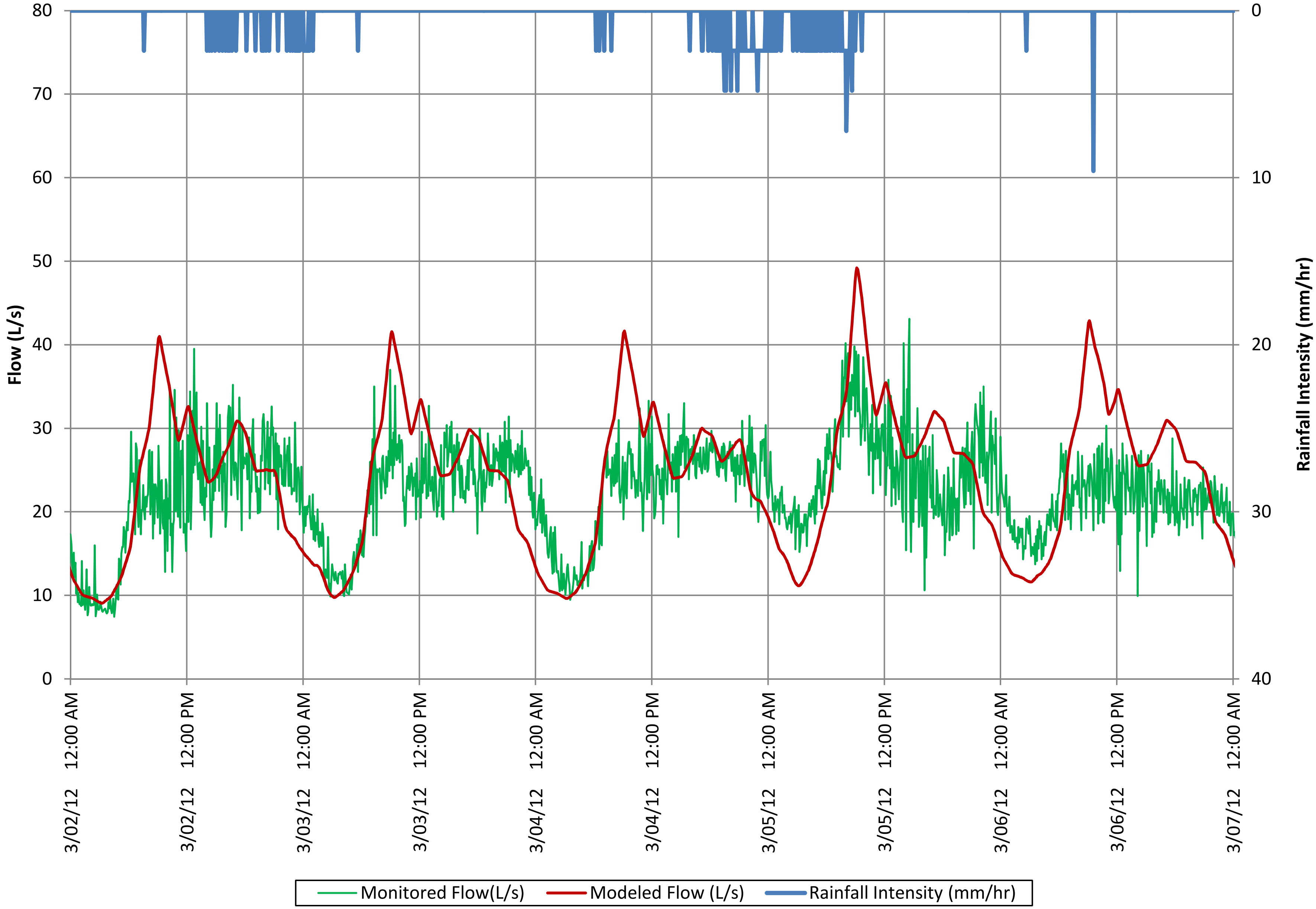


Figure 3a: Site 3 Dry Weather Flow Calibration

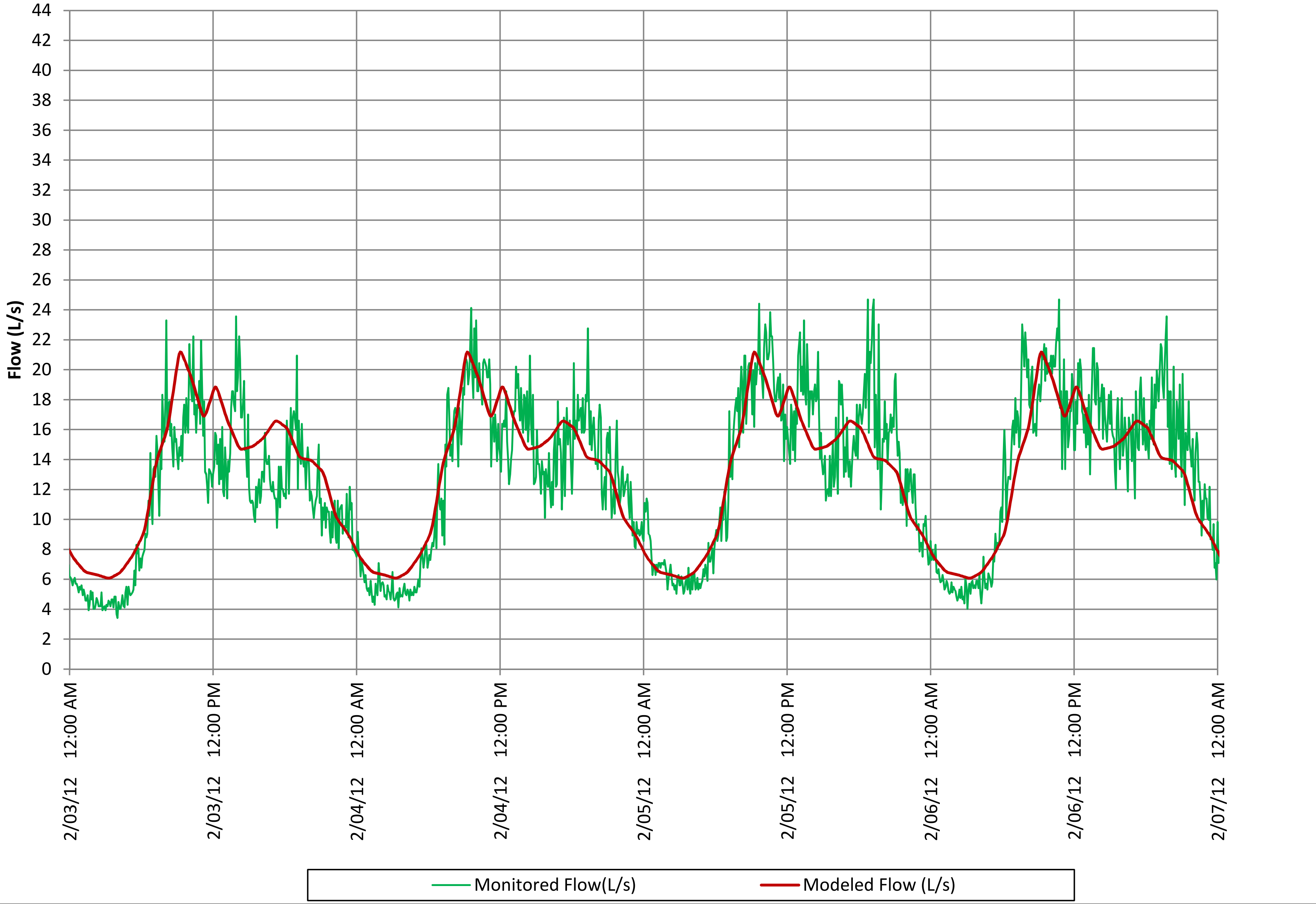
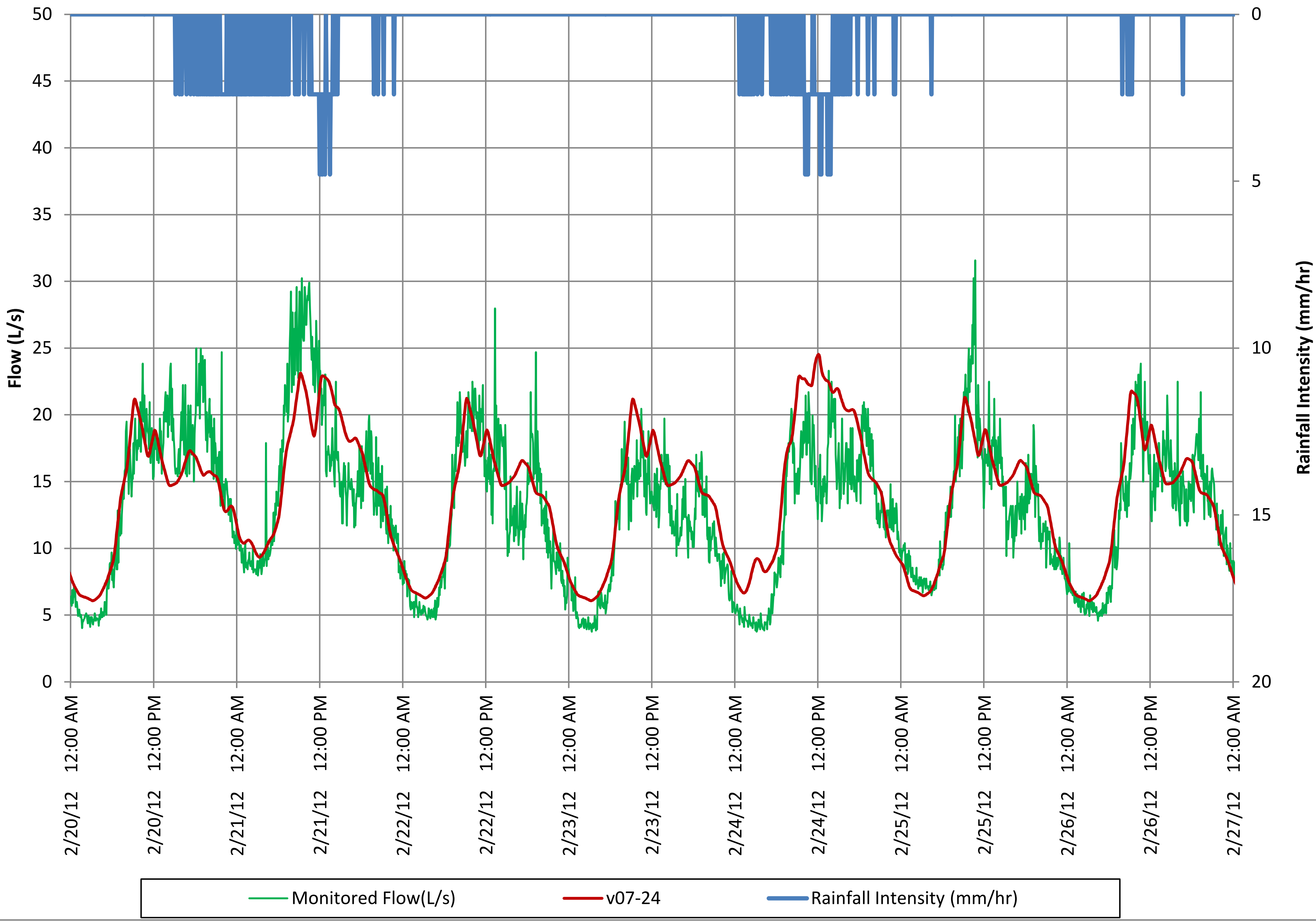


Figure 3b: Site 3 Wet Weather Flow Calibration



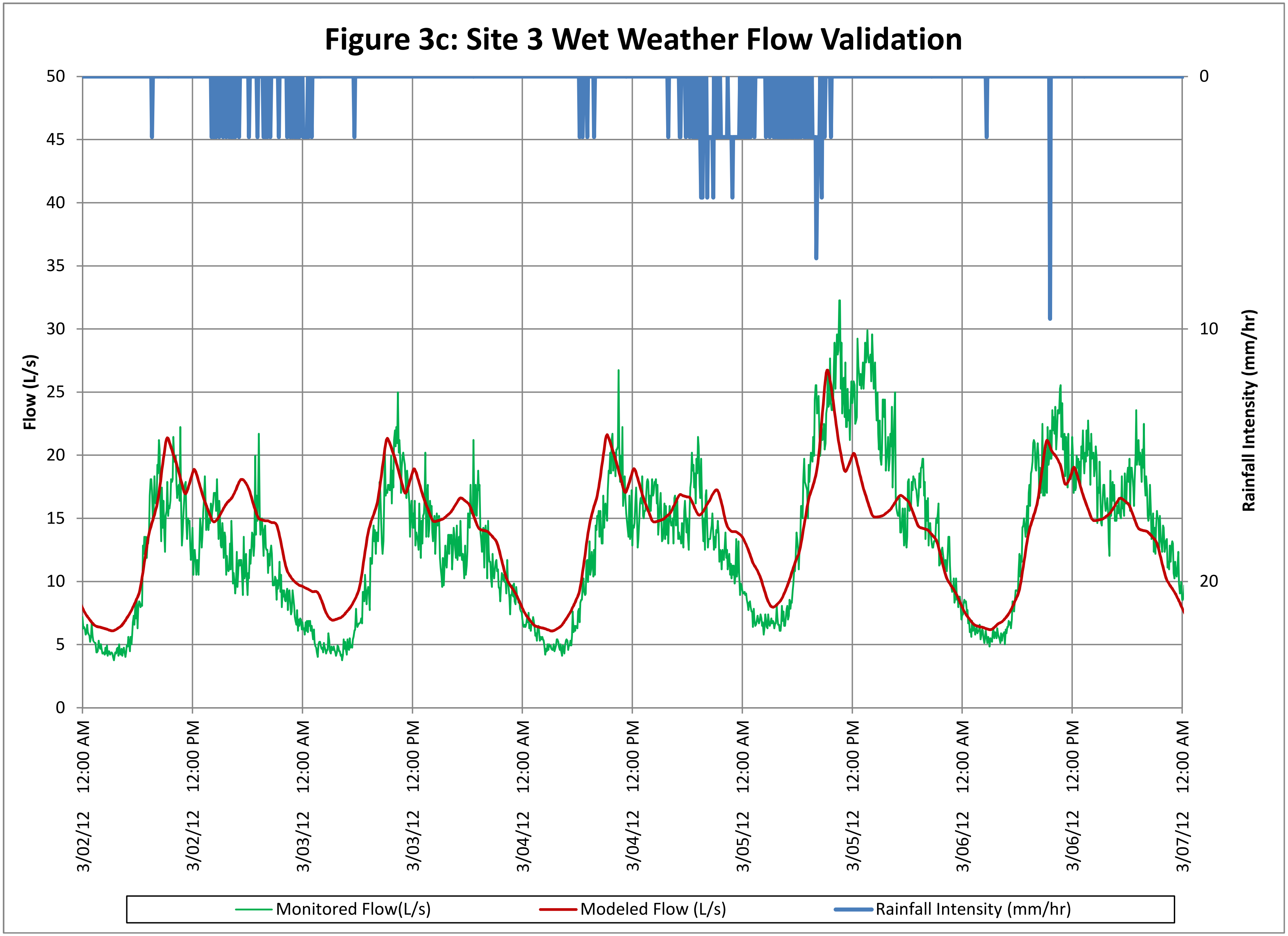


Figure 3c: Site 3 Wet Weather Flow Validation
Flow (L/s)
Rainfall Intensity (mm/hr)
50
45
40
35
30
25
20
15
10
5
0
0
10
20
3/02/12 12:00 AM
3/02/12 12:00 PM
3/03/12 12:00 AM
3/03/12 12:00 PM
3/04/12 12:00 AM
3/04/12 12:00 PM
3/05/12 12:00 AM
3/05/12 12:00 PM
3/06/12 12:00 AM
3/06/12 12:00 PM
3/07/12 12:00 AM
Monitored Flow(L/s)
Modeled Flow (L/s)
Rainfall Intensity (mm/hr)

Figure 4a: Site 4a Dry Weather Flow Calibration

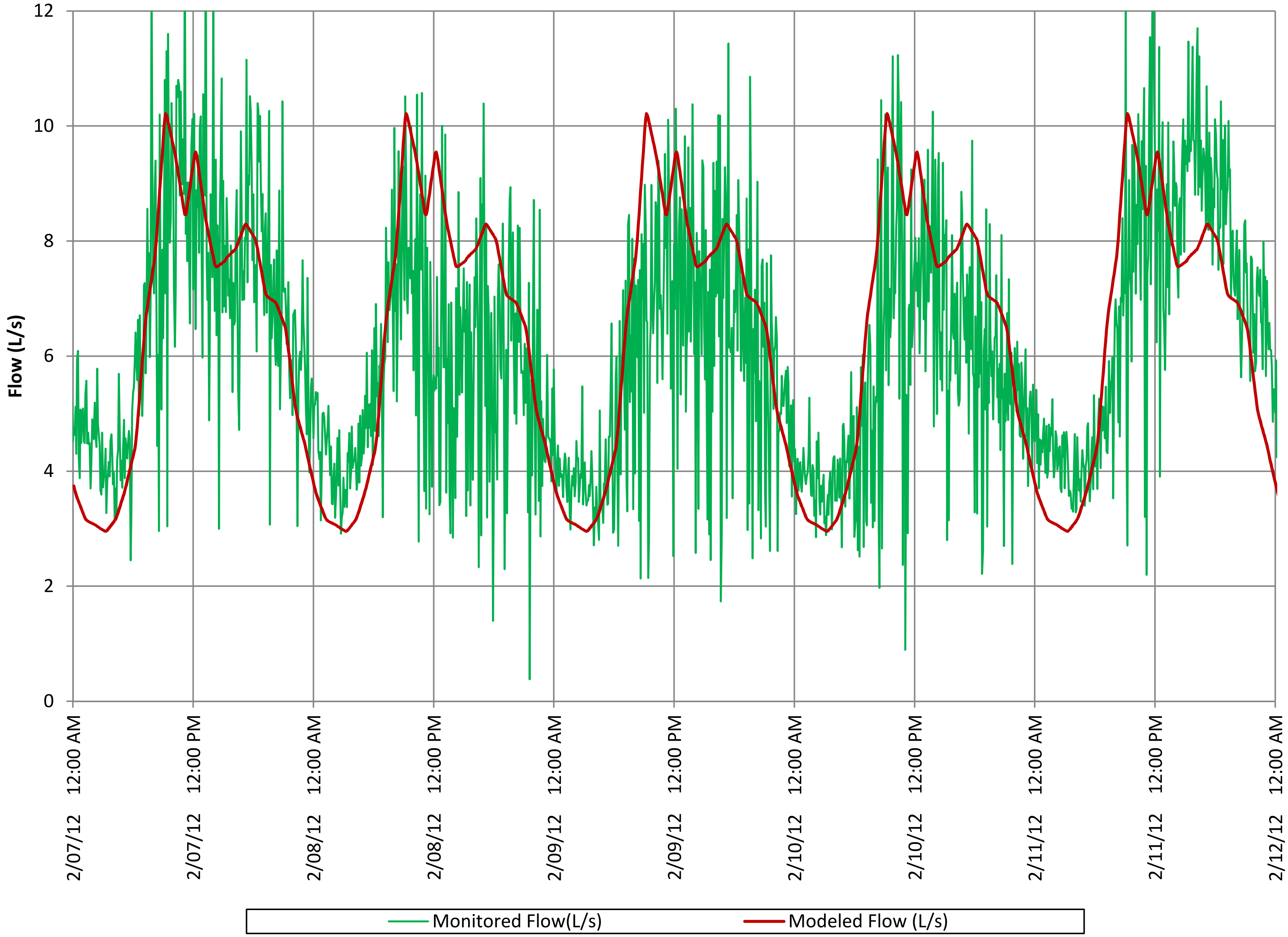


Figure 5a: Site 5 Dry Weather Flow Calibration

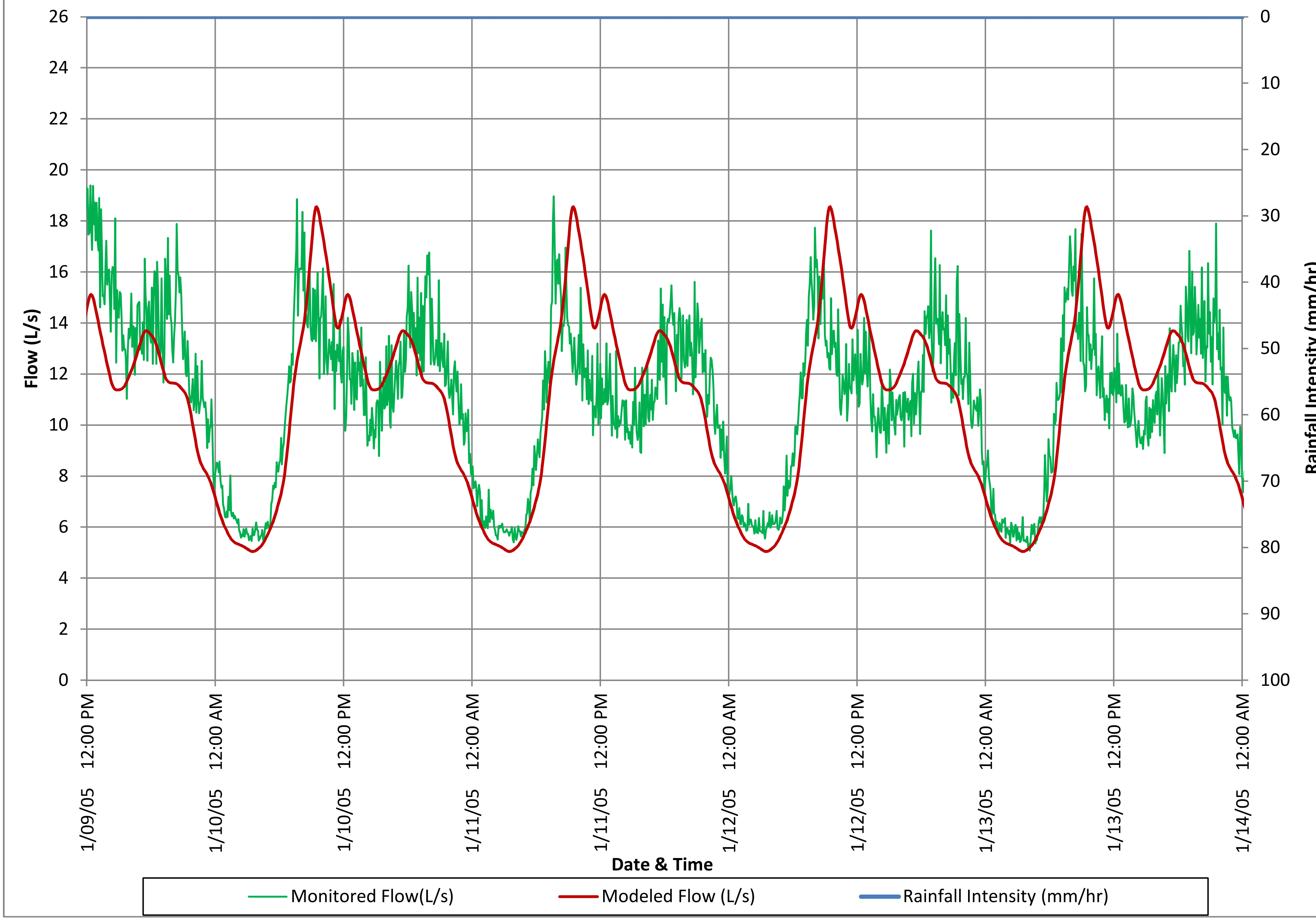
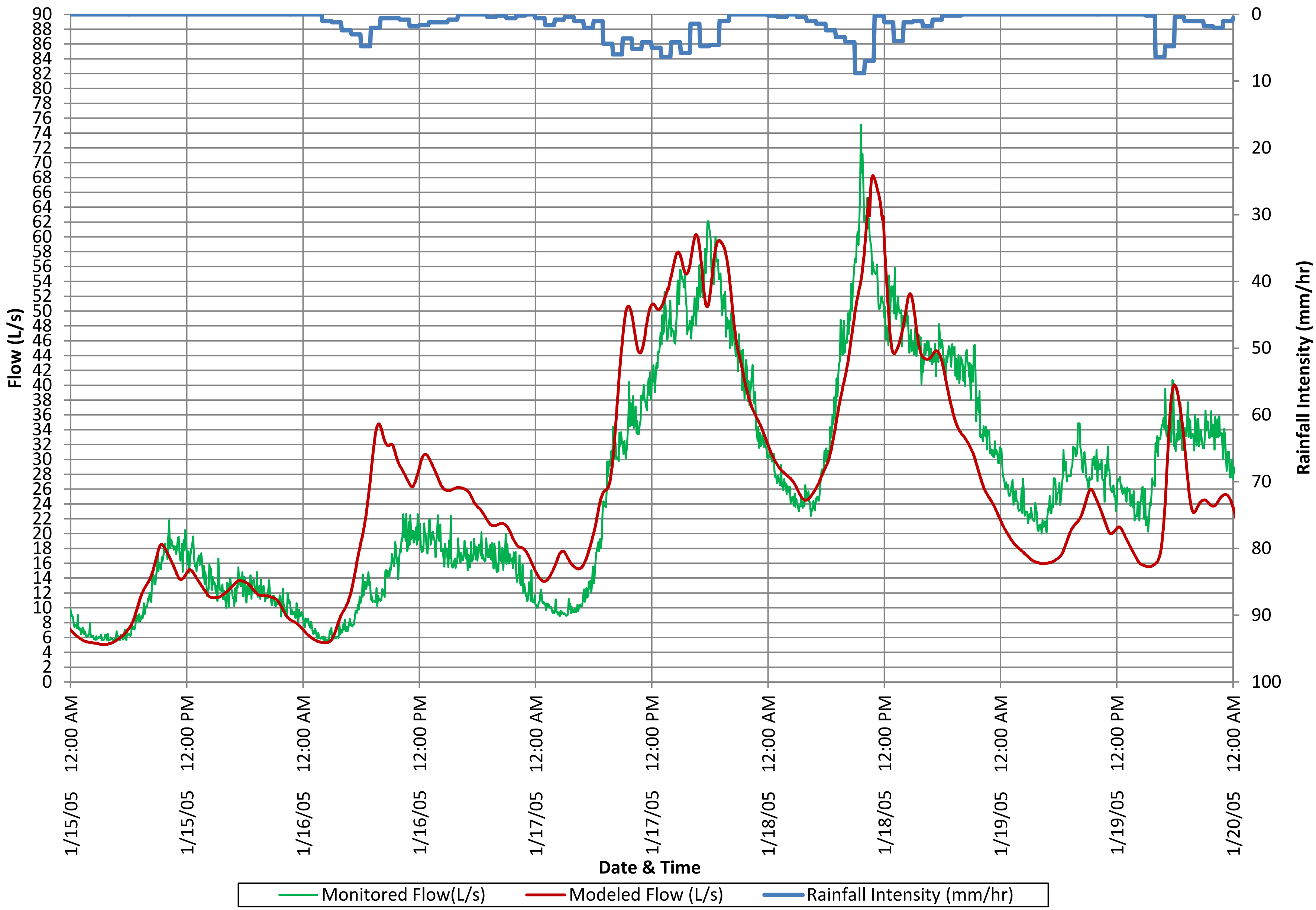


Figure 5b: Site 5 Wet Weather Flow Calibration



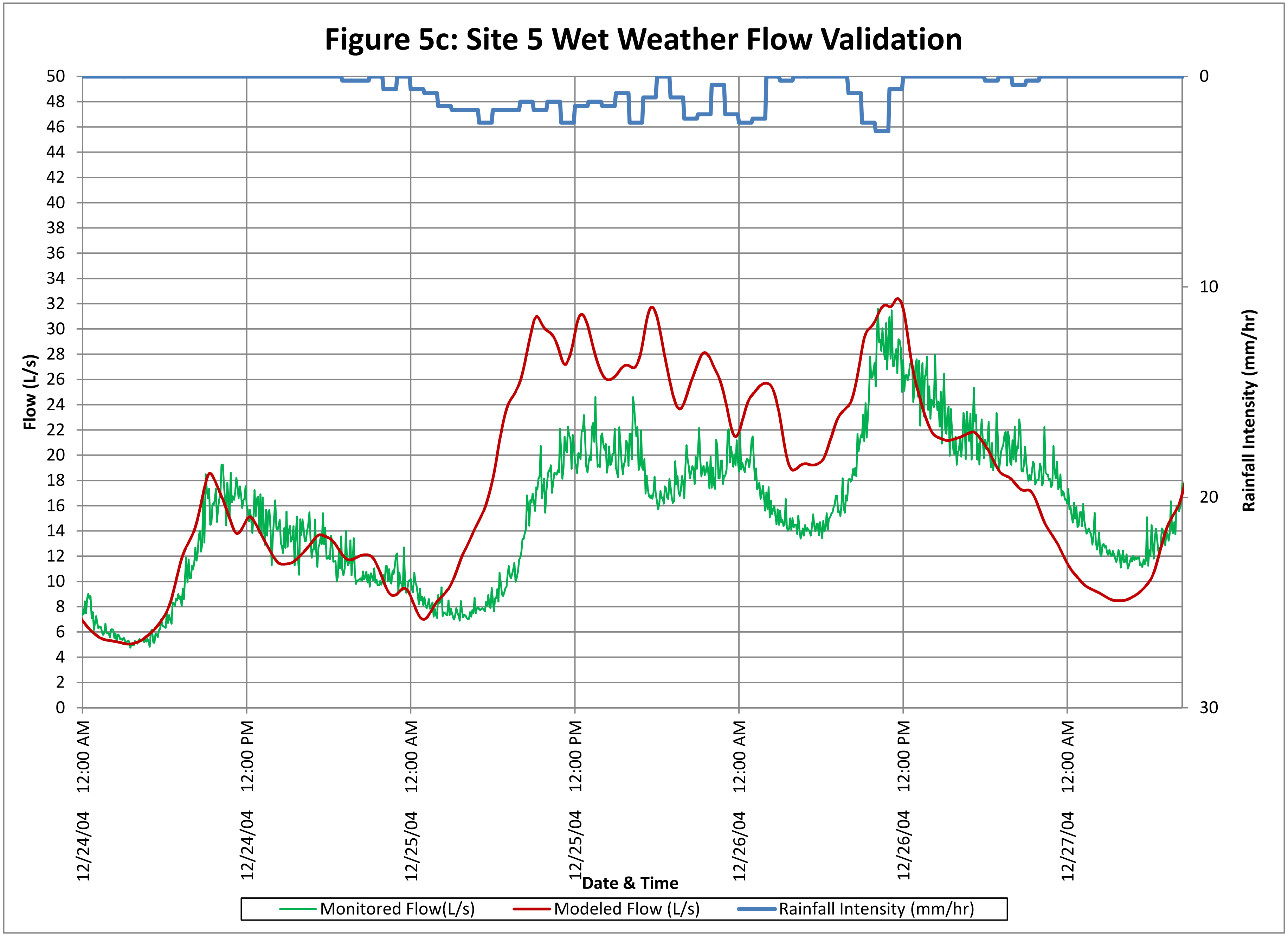


Figure 5c: Site 5 Wet Weather Flow Validation
Flow (L/s)
50
48
46
44
42
40
38
36
34
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
Rainfall Intensity (mm/hr)
0
10
20
30
12/24/04 12:00 AM
12/24/04 12:00 PM
12/25/04 12:00 AM
12/25/04 12:00 PM
12/26/04 12:00 AM
12/26/04 12:00 PM
12/27/04 12:00 AM
Date & Time
Monitored Flow(L/s)
Modeled Flow (L/s)
Rainfall Intensity (mm/hr)

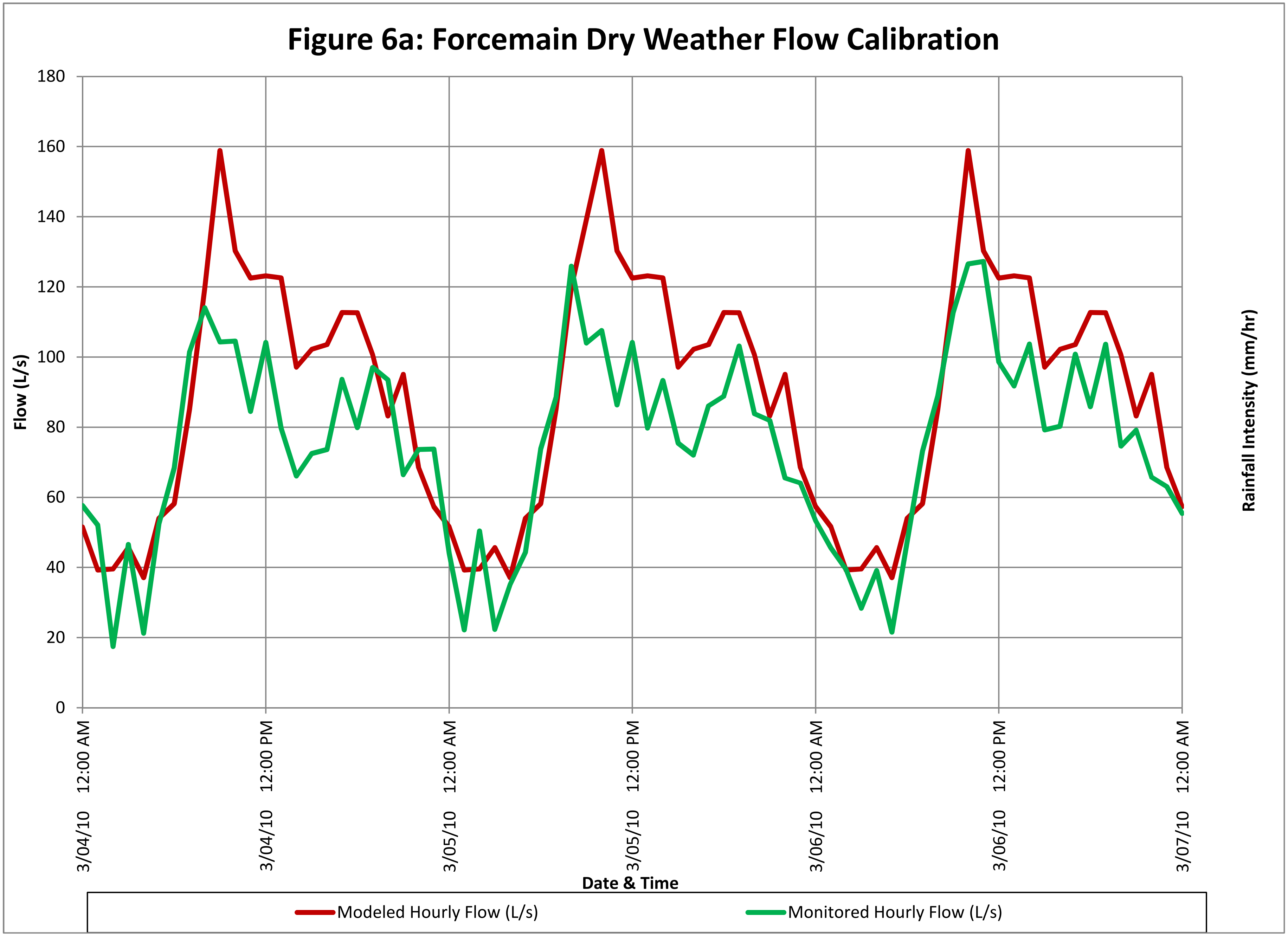


Figure 6a: Forcemain Dry Weather Flow Calibration
Flow (L/s)
180
160
140
120
100
80
60
40
20
0
Rainfall Intensity (mm/hr)
12:00 AM 3/04/10
12:00 PM 3/04/10
12:00 AM 3/05/10
12:00 PM 3/05/10
12:00 AM 3/06/10
12:00 PM 3/06/10
12:00 AM 3/07/10
Date & Time
Modeled Hourly Flow (L/s)
Monitored Hourly Flow (L/s)

Figure 6b: Forcemain Wet Weather Flow Calibration

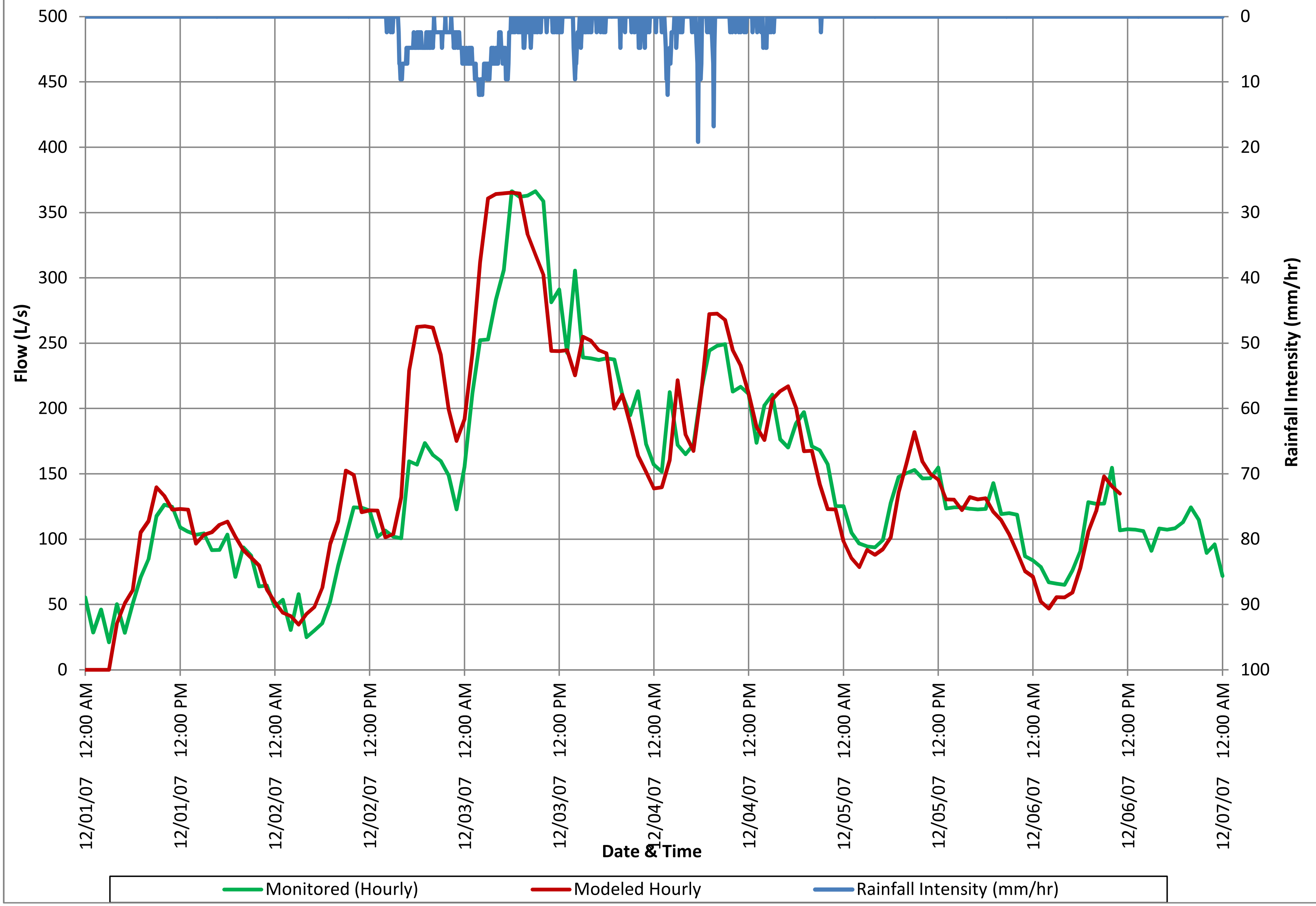
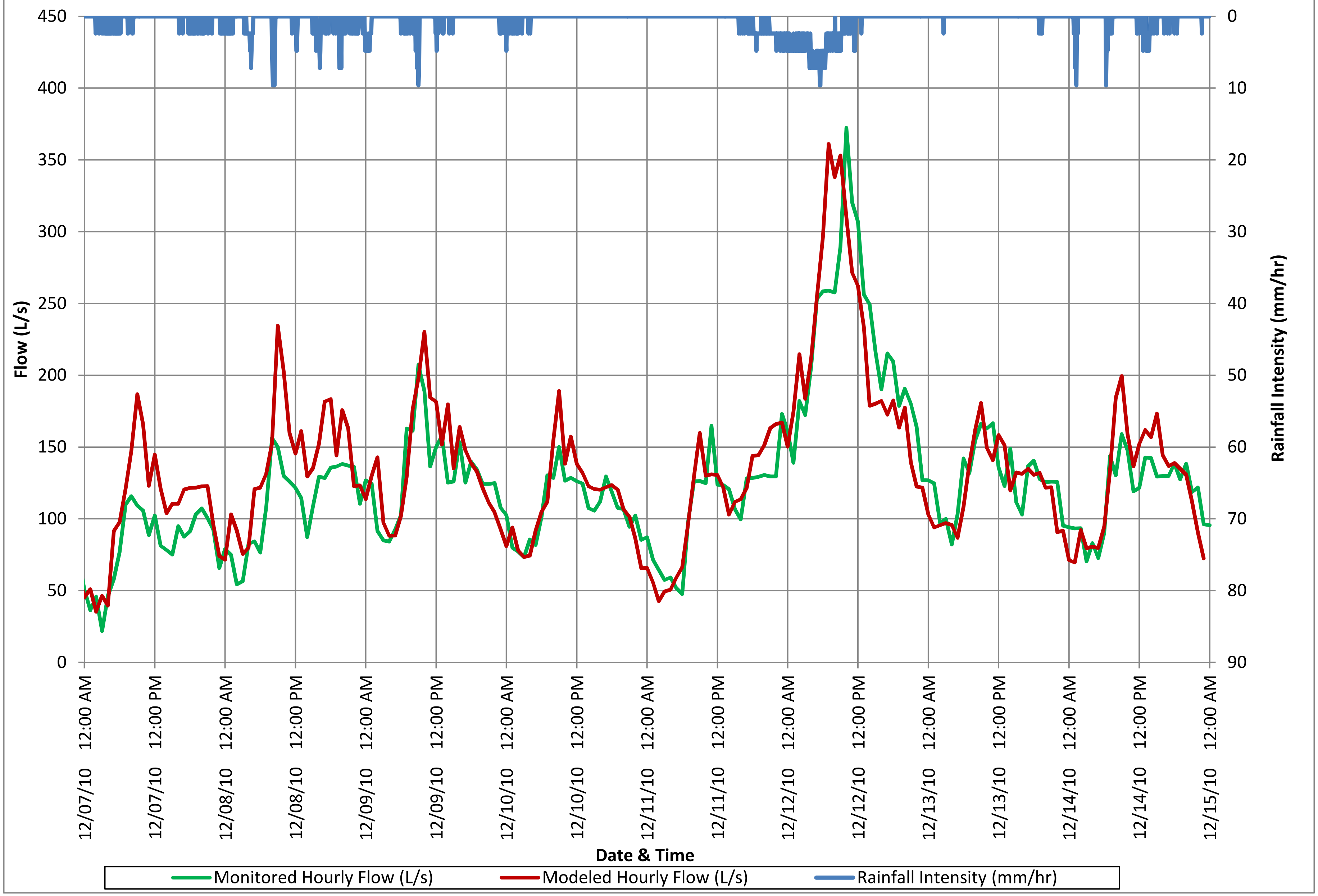
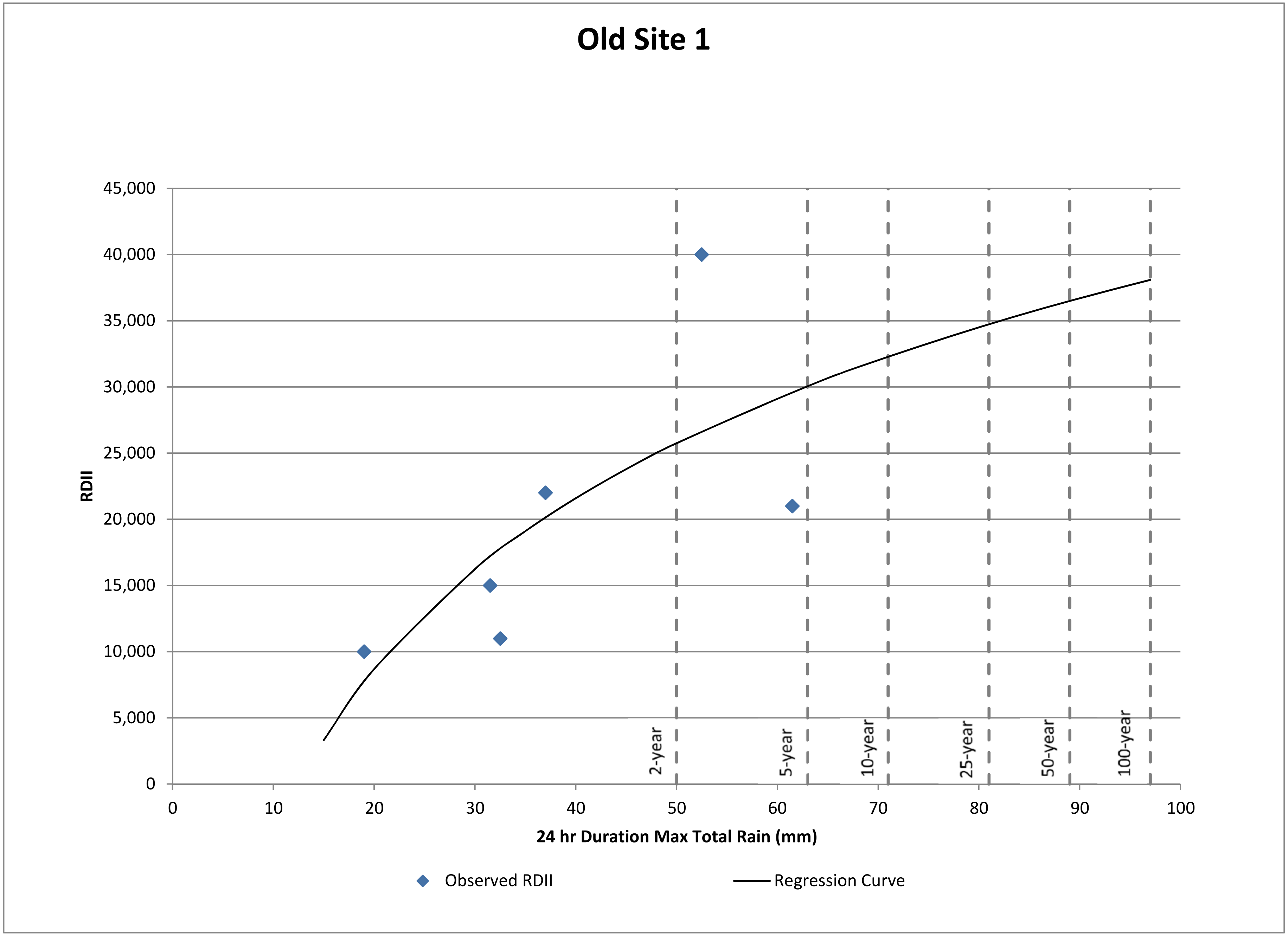


Figure 6c: Forcemain Wet Weather Flow Validation

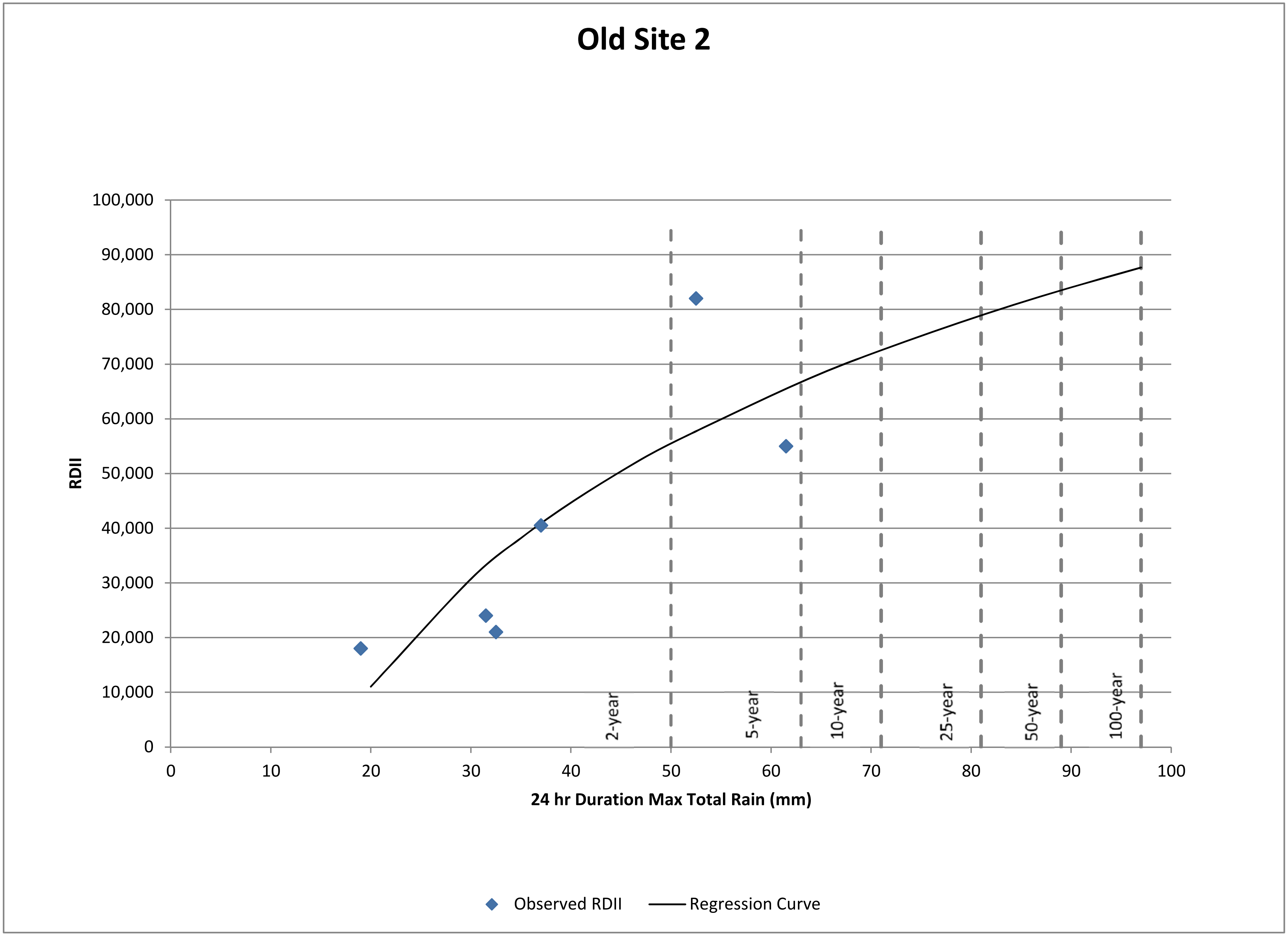


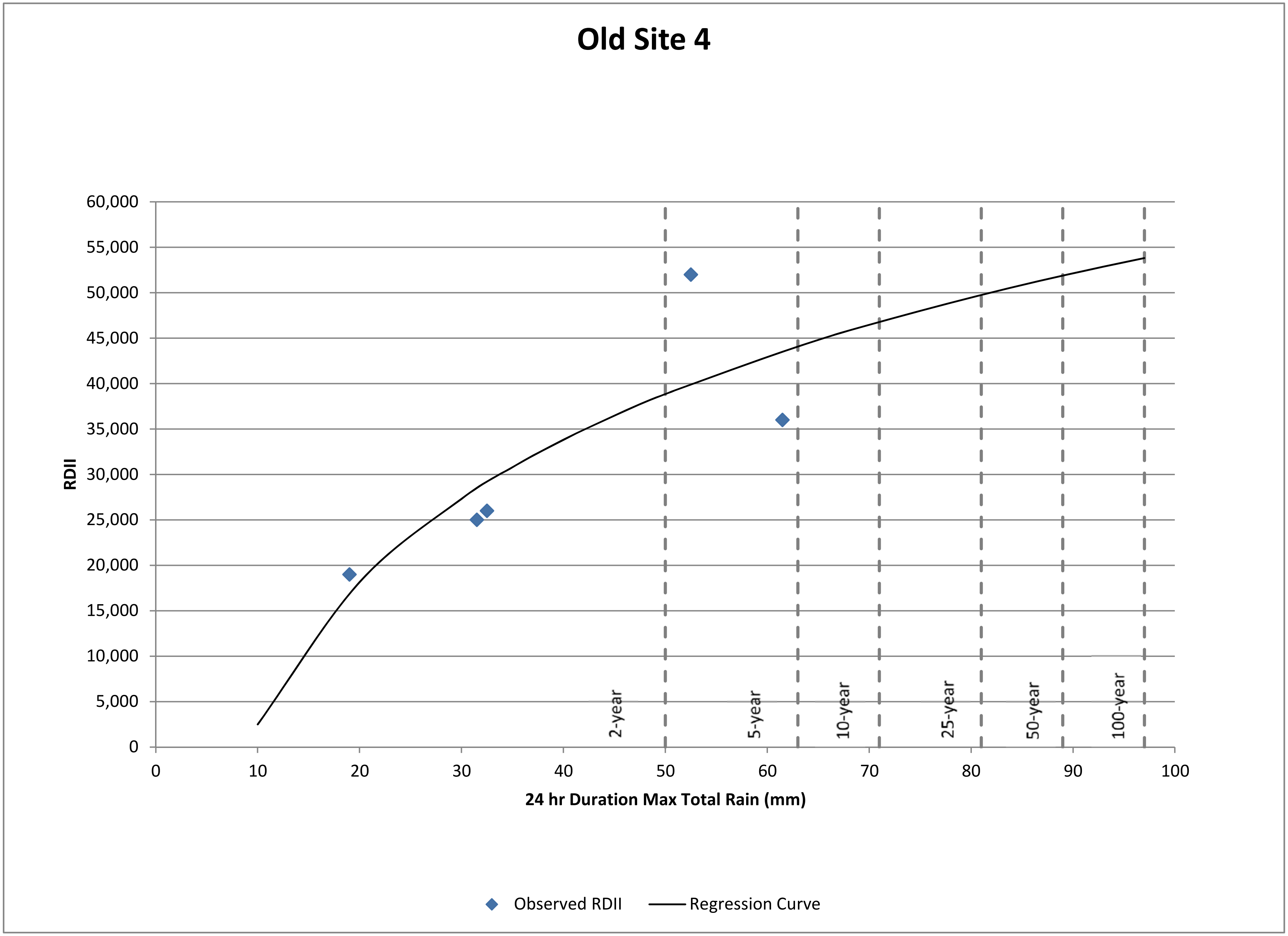
Appendix B

- **RDII Estimation Graphs**

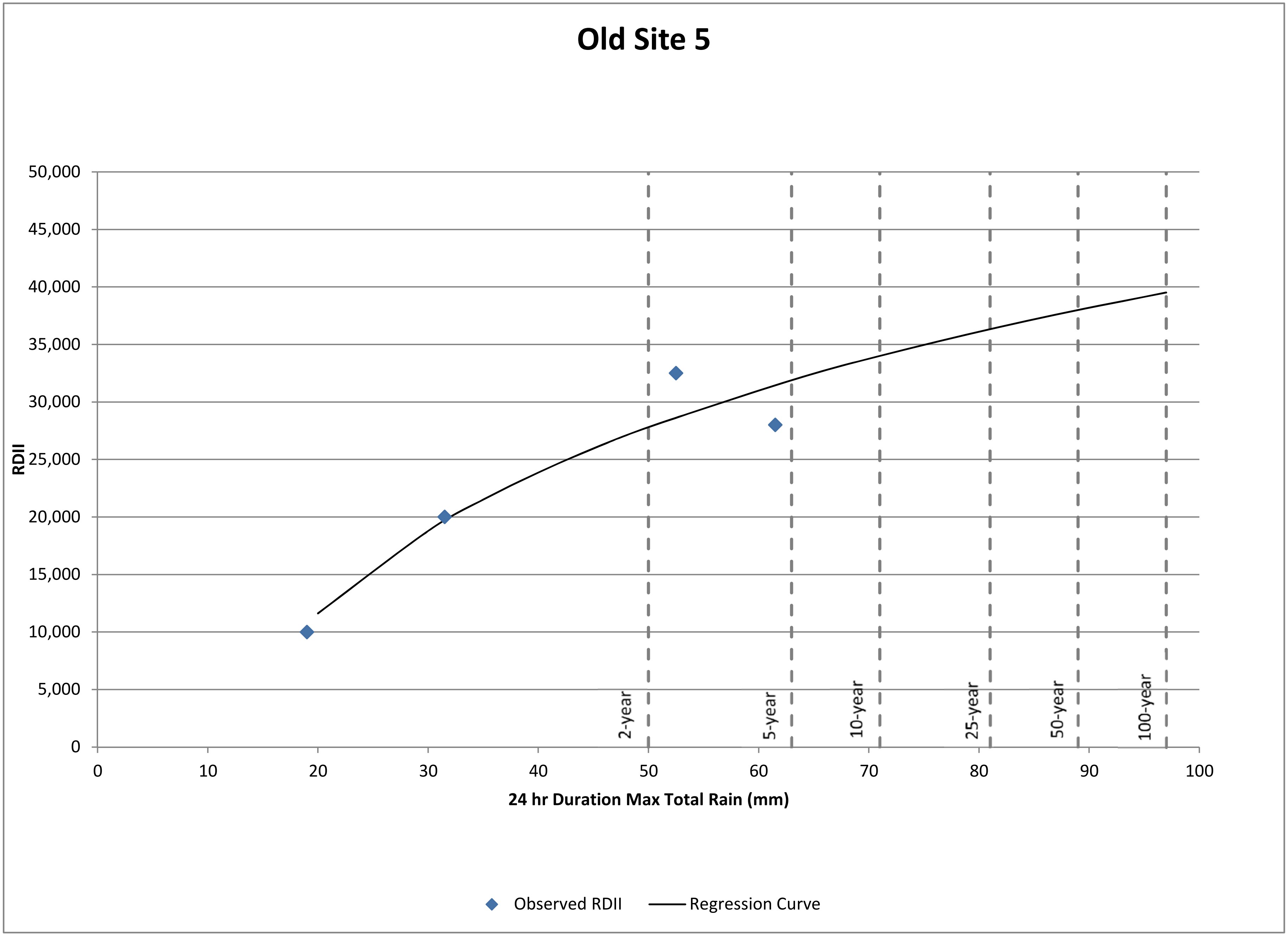


Old Site 1
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
RDII
0
10
20
30
40
50
60
70
80
90
100
24 hr Duration Max Total Rain (mm)
2-year
5-year
10-year
25-year
50-year
100-year
Observed RDII
Regression Curve

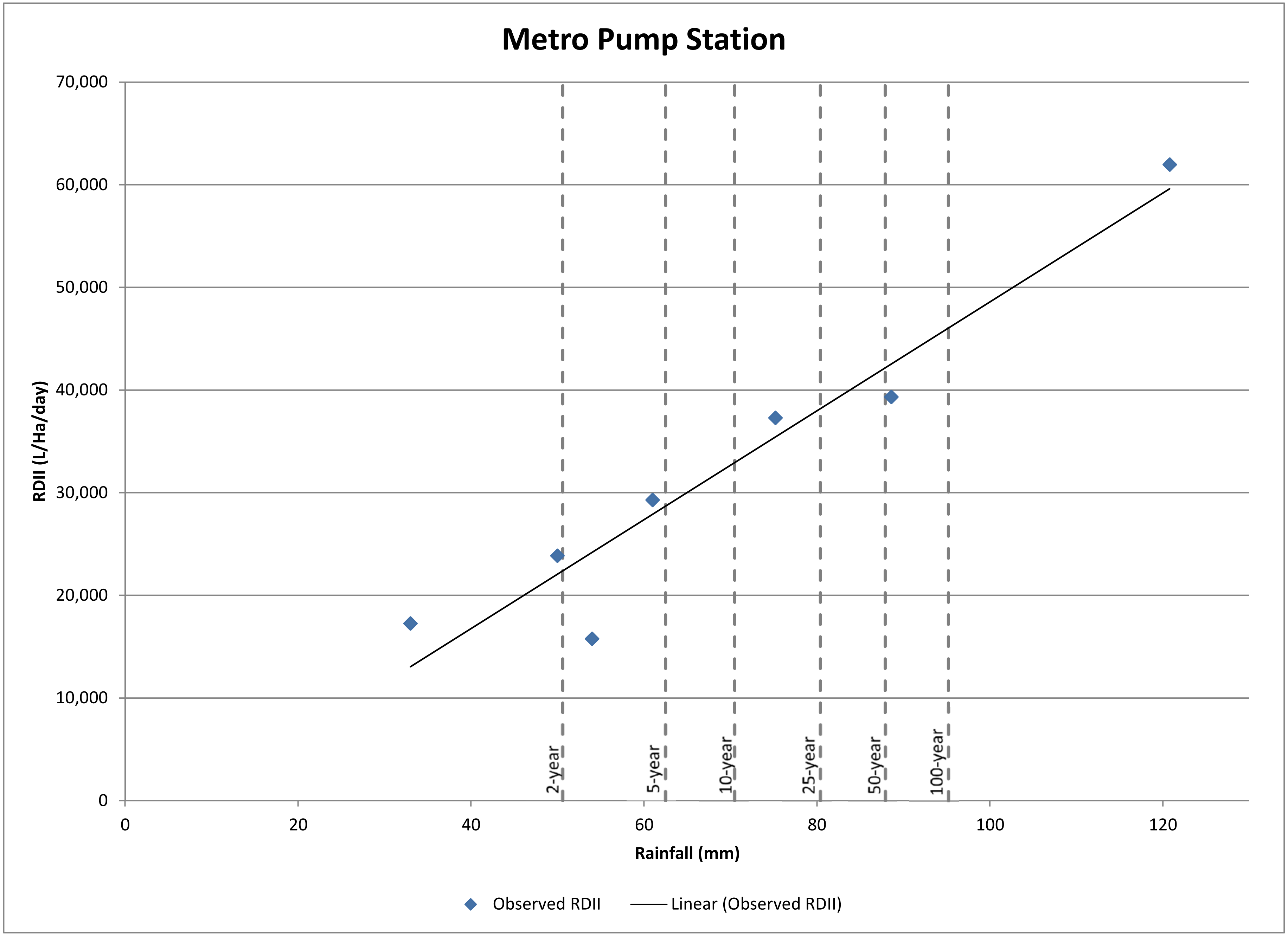

Old Site 2
RDII
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
0
10
20
30
40
50
60
70
80
90
100
24 hr Duration Max Total Rain (mm)
2-year
5-year
10-year
25-year
50-year
100-year
Observed RDII
Regression Curve



Old Site 4
RDII
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
0
10
20
30
40
50
60
70
80
90
100
24 hr Duration Max Total Rain (mm)
2-year
5-year
10-year
25-year
50-year
100-year
Observed RDII
Regression Curve



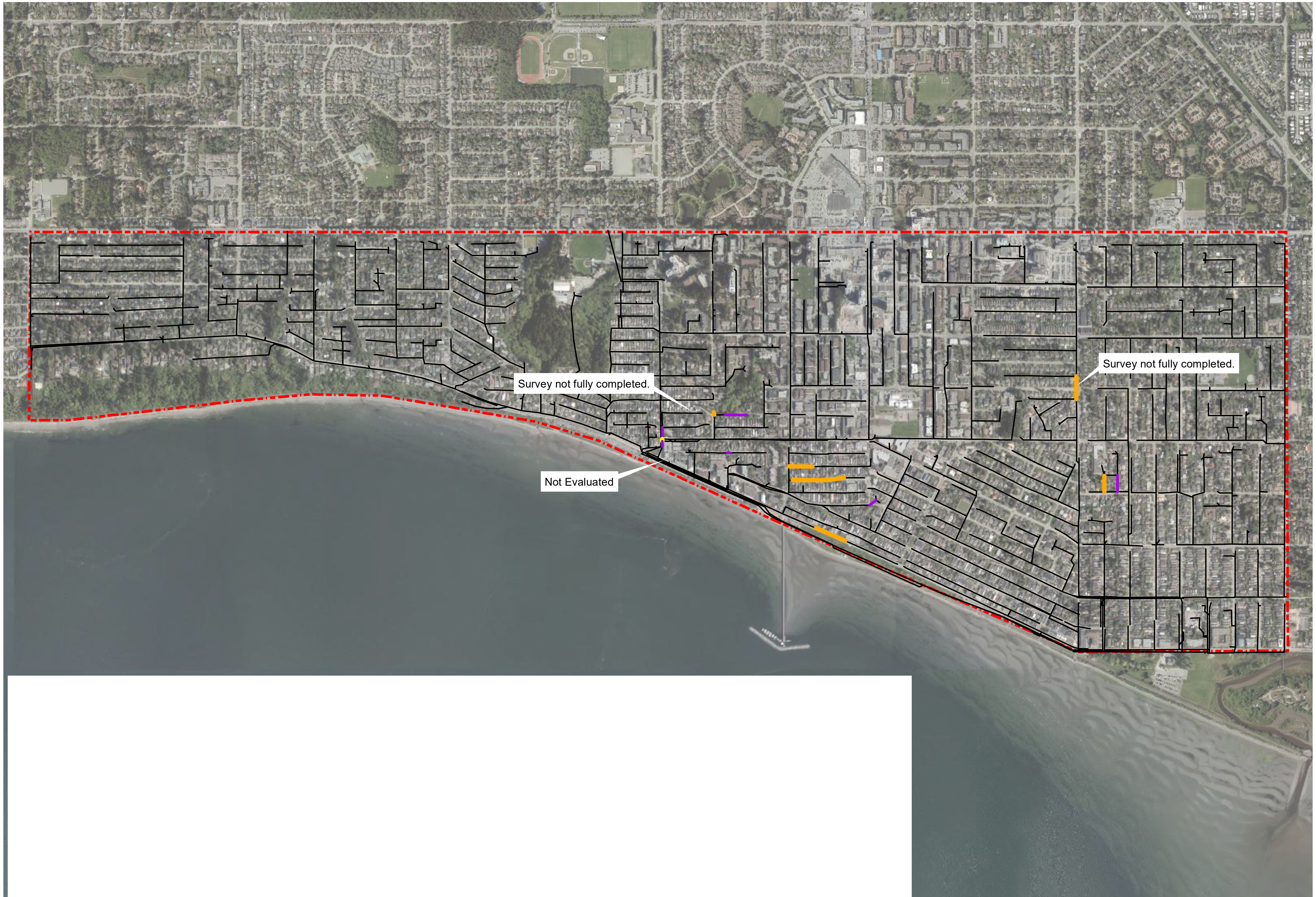
Old Site 5
RDII
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
0
10
20
30
40
50
60
70
80
90
100
2-year
5-year
10-year
25-year
50-year
100-year
24 hr Duration Max Total Rain (mm)
Observed RDII
Regression Curve


Metro Pump Station
RDII (L/Ha/day)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
2-year
5-year
10-year
25-year
50-year
100-year
0
20
40
60
80
100
120
Rainfall (mm)
Observed RDII
Linear (Observed RDII)

Appendix B

2018 CCTV Programs Pipe Condition Summary

FIGURE B1



Legend

Study Area

Condition / Maintenance Rating

- 3
- 4
- Survey Not Completed or Not Evaluated

Note: Only pipes in Fair to Very Poor condition are identified on this map

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

WHITE ROCK City by the Sea!

Sewer Master Plan Update 2019

2018 CCTV Programs Pipe Condition Summary

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