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City of White Rock Drainage 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 Drainage Master Plan and the drainage model. The drainage master plan was originally completed in 1999 and was updated in 2004 by Urban Systems and in 2012 by AECOM. With recent developments and the new Official Community Plan (OCP) published in 2017, the City would like to update the drainage model to reflect new development conditions and future projected growth conditions to develop an updated drainage master plan.

The City has a land area of 513 Ha with a 2016 population of 19,952 people. Existing land use is predominantly low density residential with some medium density residential, commercial, institutional, and open space areas. Future development through densification is expected to concentrate in Town Centre, which is the urban centre of White Rock with a mix of multi-unit residential and commercial uses. Future population is expected to reach to as high as 27,300 by 2045 depending on development conditions.

The City's drainage network receives runoff from a total catchment area of 772 Ha, of which 229 Ha are from the City of Surrey. The majority of storm sewers discharge directly to Semiahmoo Bay or to Little Campbell River through piped outfalls or culvert crossings along Marine Drive, while the rest discharge to Surrey at Stayte Road and 136 Street. There are approximately 100 km of storm sewers in the City and 76.7 km are currently built into the drainage model. Typically, storm connections, local sewers, or upstream segments of storm mains that did not have sufficient data in the GIS database are not included in the model. Pipe sizes in the City's drainage system range from 100 mm to 1325 mm.

There are two stormwater pump stations operated by the City. The Oxford Pump Station discharges directly to Semiahmoo Bay and the Keil (Habgood) Pump Station discharges to Little Campbell River. The City is currently designing a new Habgood Pump Station so that it can be relocated from First Nations land. Relocation of the Keil (Habgood) Pump Station will also include a new forcemain, larger size outfalls near Finlay Street, and reconnection of existing gravity sewers to the new pump station.

Slope stability review on the City's major ravines was completed by Thurber Engineering Ltd. In general, Thurber recommended that dumping of garden waste along the slope crest should be prohibited. Detailed assessment on the Lower Coldicutt Ravine was recommended as a landslide was observed in 2012. Specific recommendations for each ravine can be found in the report attached in Appendix B.

The drainage model was developed using XP-SWMM. To update the existing drainage model, GIS data of the drainage system and record drawings of recent developments were reviewed. The model's hydrologic parameters and existing land use imperviousness were maintained from the previous study since model calibration was not completed as part of this update.

Future development is expected to increase the land imperviousness in the City and thus increasing the runoff volume and peak flows in the drainage system. In the previous model update, catchment widths were increased to simulate higher peak flows in the future model. Catchment imperviousness was only adjusted if a significant change in the land use was expected. In this update, catchment imperviousness was increased to simulate future flow conditions rather than catchment width. As future developments will take form of infill and redevelopment, subcatchment sizes are not expected to change significantly. A better prediction of future peak flows is to increase subcatchment imperviousness to reflect the increase in impervious area from infill and redevelopments activities.

The City's drainage system was evaluated under the 10-Year 1-Hour storm event under both existing and future growth conditions. The assessment criteria in this master plan would require that pipes with peak flow exceeding 125% of their design capacity and the HGL less than 1.2 m below ground to be upgraded. These pipes are at a higher risk of flooding and should have a higher priority when staging upgrades. Pipes with peak flows at 100% - 125% of their capacity and with HGL well below ground surface are at a lower risk of flooding. These pipes were identified as optional upgrades and the upgrades can be completed with future developments as necessary.

Based on the capacity assessment, the number of pipes that surcharge under the 10-Year 1-Hour storm increases significantly from existing to future conditions. A majority of these additional surcharged pipes are in the Mature Neighbourhood area due to the overall increase in impervious coverage expected from future infill and redevelopment activities. Since the future condition is modelled to the year 2045, these pipes have a much lower upgrading priority compared to the pipes that are currently undersized. While identifying upgrades, flow diversions were proposed if incorporating a short section of diversion pipe to the drainage system can reduce the length of pipes required to be upgraded to reduce cost and disturbance area.

An upgrade was identified for the 750 mm storm sewer on Nichol Road in the 2012 study. However, based on simulations with different flow scenarios, it was determined that the pipe has sufficient capacity to convey the 10-Year peak flows under future growth conditions in White Rock and an assumed 10% increase in the overall catchment imperviousness from Surrey. To confirm whether an upgrade is required, it is necessary to confirm the future discharge rate from Surrey to White Rock.

CCTV data on pipes in Areas A to E of the City were reviewed. Pipes with condition rating of ‘Fair’, ‘Poor’ or ‘Very Poor’ are identified and these pipes should have priority for replacement or repair.

The City of White Rock and the City of Surrey are currently discussing drainage agreements between the two municipalities. As part of the master plan update, the City wanted to explore the option of completely diverting flows currently discharging to Surrey on Stayte Road to manage the flows within White Rock. In this study, a diversion alignment and the required trunk sizes were proposed. The upgrades and costs associated with the full diversion is compared with the partial diversion option proposed in the 2012 study and with existing condition (i.e. no diversion).

The Oxford Pump Station is under capacity, as was previously identified in 2012. Currently, a bypass pipe is available to direct overflows from the pump station to the beach. Since the model was simulated with free flow conditions, a major storm during high tide may cause backwater flows in the bypass pipe and ultimately causing surface flooding in the parking area.

The model was developed assuming the design configurations of the new Habgood Pump Station and the associated forcemain, gravity sewer connections, and outfalls in the Preliminary Design Report by Opus. Based on the current design capacity, the pipes upstream of the pump station will surcharge (with HGL above pipe obvert but below ground elevation) under the 10-Year 1-Hour event under improved conditions of the partial and no diversion options. Note that the Preliminary Design Report completed by AECOM states that the capacity for the new pump station was designed to eliminate flooding.

A Capital Plan was developed based on upgrades required with the full diversion option. A summary of the estimated expenditures for the proposed upgrades is provided in Table ES-1 below.

Table ES.1: Summary of Capital Plan

Year	Approximate Length to be Replaced (m)	Cost Estimate (without Engineering & Contingency)	Cost Estimate (with Engineering & Contingency)
2019	1,935	$2,419,316	$3,266,077
2020	1,479	$3,229,914	$4,360,384
2021	1,732	$1,650,557	$2,228,253
2022	1,053	$1,650,557	$2,228,253
2023	1,021	$1,084,929	$1,464,654
2024-2029	2,964	$2,852,583	$3,850,987
Total	10,184	$12,887,856	$17,398,608

1.0 Introduction

The City of White Rock's ("the City's") drainage master plan was last updated in 2012. With recent developments and the new published Official Community Plan (OCP), the City would like to update its drainage master plan to incorporate the new drainage infrastructure and ensure that the strategic and sustainable vision presented in the OCP will be fulfilled. The drainage master plan was originally completed in 1999 and was since updated twice in 2004 by Urban Systems and in 2012 by AECOM.

The master plan update assesses the drainage system under existing and future conditions to determine upgrades that are necessary to address existing shortcomings and support future developments. A 10-Year Capital Plan was also developed to prioritize upgrades and help the City's Administration and Council with infrastructure asset planning.

White Rock is a seaside community located in the southwest corner of the Lower Mainland. The City is located directly north of Semiahmoo Bay and is bound by the City of Surrey to the north, west and east. The City has a land area of 513 Ha and the population is just under 20,000 people (2016). The majority of the land in the City is developed. Future development in the form of densification (infill and redevelopment) will be concentrated in Town Centre, which is the urban centre of White Rock that has a mix of multi-unit residential and commercial uses. The study area is shown in Figure 1.1.

1.1 Key Issues & Objectives

Since the 2012 update, a number of developments were completed. The drainage model needs to be updated with new infrastructure constructed in recent years and the system has to be reassessed under existing and future conditions. The goal of this project is to create a plan suitable to address the 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. In addition to addressing the existing and future needs, the City would also like to explore the option of diverting flows that currently discharge to Surrey through outfalls on Stayte Road and manage them within White Rock.

To achieve the objectives of this master plan update, the following were completed:

- Updated the previous drainage model with drainage infrastructure constructed in recent years.
- Reviewed the capacity of the existing system and recommended upgrades.
- Reviewed the system capacity under future development conditions (by estimating catchment imperviousness of future OCP land use designations) and recommended upgrades.
- Designed flow diversion to capture runoff to Surrey within the White Rock.
- Prioritized proposed upgrades into a 10-Year Capital Plan.

1.2 Previous Studies and Relevant Reports

Drainage Master Plan Update by AECOM, 2012

In this study, the drainage model was updated to include drainage infrastructure constructed since the 2004 update. Four flow monitoring stations were setup by SFE Global between January and March 2012 to collect flow data for model calibration purposes. The report identifies that there are four locations where the City of White Rock discharges flow to City of Surrey, and AECOM recommended the City to maintain base flow to Surrey and manage high flows within the City of White Rock.

Capacity assessment was completed with the 10-Year 1-Hour design event and future land use conditions from the 2008 OCP. Under the 10-Year 1-Hour design event, both pump stations are undersized. The report proposed system upgrades and diversions to address flooding potentials and control peak flows within White Rock. Outfalls and major culverts were reviewed under the 100-Year design storm. Only 17 out of the 25 outfalls had enough information to be modelled, the review found 5 outfalls to be undersized to convey 100-Year flows.

A copy of this report is provided in Appendix A.

City of White Rock Habgood Pump Station Relocation, Preliminary Design Report by Opus International Consultants, 2017

The Habgood Pump Station is currently located on Semiahmoo First Nations land and discharges to Little Campbell River. This report was completed for the relocation of the Habgood Pump Station and other drainage infrastructure connected to the pump station that are currently outside the City's jurisdiction boundary. The report provides preliminary design details on the pump station, forcemain, gravity sewers connected to the pump station, system curve of the pumps, etc. Design information provided in this report was used to develop the updated storm model, although the pump station design is not yet finalized.

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 decisions on planning and land use management. The current OCP is developed to the year 2045 and provides policies in growth management and land use designations. Future growth will follow the existing growth patterns concentrating in Town Centre, Town Centre Transition, and Lower Town Centre areas. The goal in land use designation is to maximize the use of the City's limited land to create a complete community. There are 11 land use designations in the OCP's future land use plan, shown in Figure 1.2. Future developments will focus in Town Centre and its surrounding areas. Mature Neighbourhood will have some redevelopment in the form of residential infill.

1.3 Existing Drainage Infrastructure

The City's drainage network receives runoff from a total catchment area of 772 Ha, of which 299 Ha are from the City of Surrey. The drainage pattern in the City is generally from north to south, with some areas in the east side of the City draining towards Surrey to the east. The majority of the storm sewers discharge directly to Semiahmoo Bay or Little Campbell River through piped outfalls or culvert crossings under the BNSF Railway along Marine Drive, while the rest discharges to Surrey at four locations (one on the west boundary and three on the east). There are 18 major drainage catchment areas within the City, and with the removal of the outfall to Little Campbell River that is currently in progress, catchment 14 and 15 (shown on Figure 2.1 of AECOM's 2012 master plan update) will be combined into one catchment. The major drainage catchment areas are shown on Figure 1.3.

There are two stormwater pump stations operated by the City: Oxford Pump Station and Habgood Pump Station. The Oxford Pump Station is located west of the Oxford Street and Marine Drive intersection. This pump station has a catchment area of 6.5 Ha and discharges directly to Semiahmoo Bay. The Habgood Pump Station is currently located at the foot of Habgood Street. It receives runoff from a catchment area of 14 Ha and discharges to Little Campbell River. The Habgood Pump Station will be relocated one block west of its existing location to Keil Street and the new pump station will discharge to an existing outfall at the corner of Marine Drive and Maple Street. The relocation design is currently undertaken by Opus International Consultants Ltd (Opus), details of the relocation will be discussed in later sections.

Based on the City's GIS database, there are approximately 100 km of storm sewers in the City including major trunk sewers and flow diversions. Among the 100 km of sewers, 76.7 km are in the drainage model.

Typically, pipes that were not included in the model were storm connections, local sewers or upstream segments of storm mains that do not have sufficient data in the GIS. Figure 1.4 shows pipes in the drainage system that were included in the model. Pipe sizes in the City's drainage system range from 100 mm to 1325 mm, Table 1.1 shows a breakdown of the pipes by size.

Table 1.1: Breakdown of the City's Storm Pipes by Size

Size Range	Length of Pipe
100-250	51,410m
300-525	29,597m
600-900	9,833m
1050 and up	1,597m
Unknown	8,337m
Total	**100,774m**

1.4 Pump Station Upgrade

The existing Habgood Pump Station is located on the Semiahmoo First Nations (SFN) land, south of the Marine Drive and Habgood Street intersection. The pump station currently discharges through the SFN land to Little Campbell River, and subsequently into the Semiahmoo Bay and the Strait of Georgia. A Preliminary Design Report was recently completed by Opus to relocate the Habgood Pump Station from SFN land to White Rock city land.

The proposed location of the new Habgood (Keil) Pump Station is near the intersection of Marine Drive and Keil Street, which is one block west of the existing location. The new pump station will discharge into the existing 1050 mm storm main at the corner of Marine Drive and Maple Street and drain to the outfall at the base of Finlay Street. The existing outfall on Finlay Street will be relocated 20 m to the west so that it would be within the White Rock jurisdiction. The relocation of the Habgood Pump Station will also require reconnection of existing gravity sewers to the new pump station.

Since the relocation design has not been finalized, the storm model was built using design details provided in the Preliminary Design Report (Opus, 2017). The new pump station is designed for a 10-Year 1-Hour storm event with a design capacity of 582 L/s to pump flow from a catchment area of approximately 14 Ha. The pump station has a triplex design meaning it will have a primary and a secondary pump to provide design flow, and one standby pump. The proposed forcemain is a 550 mm HDPE pipe with an internal diameter of 489 mm. The proposed pumps are 110 HP, Flygt NP3315LT 3~628.

1.5 Slope Stability Review

Thurber Engineering Ltd. (Thurber) was retained by ISL to complete a geotechnical slope stability review on the major ravines in White Rock. The report completed by Thurber is attached in Appendix B. The report includes detailed results of the reconnaissance, comments on the condition of existing slopes, and recommendations for further work.

As a general comment, Thurber noted that dumping of garden waste near the slope crest can increase the risk of landslides during or after heavy precipitation, and recommended that the dumping activities should be prohibited. Some of the specific recommendations for each ravine are summarized below.

Coldicott Ravine

No evidence of slope instability was observed in the Upper Coldicott Ravine. However, Thurber observed dumping of garden refuse, such as garden waste, along the crest of the ravine at some locations. This can be detrimental to the stability of ravine slopes and is recommended that the City should remind home owners not to dump garden refuse.

A landslide was noted in the Lower Coldicott Ravine during the 2012 assessment. It was observed that polyethylene sheeting has been placed to cover a section of the slope, which could indicate some instability. There was also a corrugated plastic pipe connected to a PVC pipe on the failed slope surface but the inflow and outflow locations could not be located. It was recommended that a detailed assessment be completed to determine the likelihood of further slope movement and risk to adjacent private properties.

Collingwood Ravine

The creek is partially lined with concrete. The ravine slopes above the concrete lined channel had no evidence of slope instability although some dumping of garden waste was present near the ravine crest. Thurber recommended the City to remove the block of wood across the creek to prevent debris build-up. Bank slopes near the outlet could not be assessed as they were covered with vegetation. Trees on the bank near the culvert outlet showed signs of tilting and pistol butting.

Duprez Ravine

Duprez Ravine had several stability issues from 1999 to 2002, and had since had remedial works completed. In the 2018 assessment of the ravine, it was believed that the creek channel slopes are in general relatively stable. Some recommendations provided in the 2018 report are:

- Repair sinkhole observed behind the gabion wall below 14541 Magdalen Avenue.
- Surface erosion was observed on the west bank east of High Street and Blackburn Crescent. Slopes should be stabilized to mitigate further surface erosion. Thurber can provide further recommendation if needed.
- The gabion wall on the upstream end of the ravine appears to be bulging and the 1500 mm culvert is slightly out of round. It was recommended that the gabion wall should be monitored and the culvert should be structurally assessed to determine if remedial work is required.

Anderson Ravine

The slopes at Anderson Ravine are high and relatively steep at some locations although no evidence of major instabilities were observed.

Everall Ravine

In the west branch of Everall Ravine, erosion channel was observed downstream of the 300 mm corrugated steel (CSP) pipe outlet. It was recommended that the CSP pipe should be extended to the bottom of the ravine to reduce further bank erosion. In the east branch, a significant portion of the ravine is located within private property. Thurber recommended that the debris buildup against the south property line fence should be removed to prevent the weight of the debris from causing the fence to topple, which could cause a sudden release of water and debris potentially damaging the creek channel downstream.

1.6 Existing and Future Land Use

The majority of the existing land use in the City is dedicated to low density single family residential use in the Mature Neighbourhood area. Low-rise multi-unit residential use is located in the Town Centre Transition and the Urban Neighbourhood areas. Commercial land use is mainly along Johnston Road and Marine Drive. Town Centre is the urban centre of White Rock and it consists of a combination of low-rise, mid-rise, and high-rise residential and mixed-use buildings. Institutional and open space uses are scattered throughout the City.

In the OCP's future land use plan, there are 11 land use designations in the City. This land use plan is provided in Figure 1.2. The future 2045 population is projected to be 27,300 people. Future growth will concentrate in Town Centre and its surrounding areas (Town Centre Transition and Lower Town Centre). Mature Neighbourhood will expect some redevelopment or infill activities but not any significant residential growth. It was noted by the City that the average redevelopment in Mature Neighbourhood from the past five years is 60 lots per year. Future residential development will take the form of apartments, duplexes and town houses. The OCP projects an annual increase of 145-170 new apartment units and 5-10 new duplex/ townhouse units. The main future commercial developments will be additional retail and service floor space in Town Centre, Lower Town Centre and Waterfront Village. Additional grocery store space is also expected with the growth in population.

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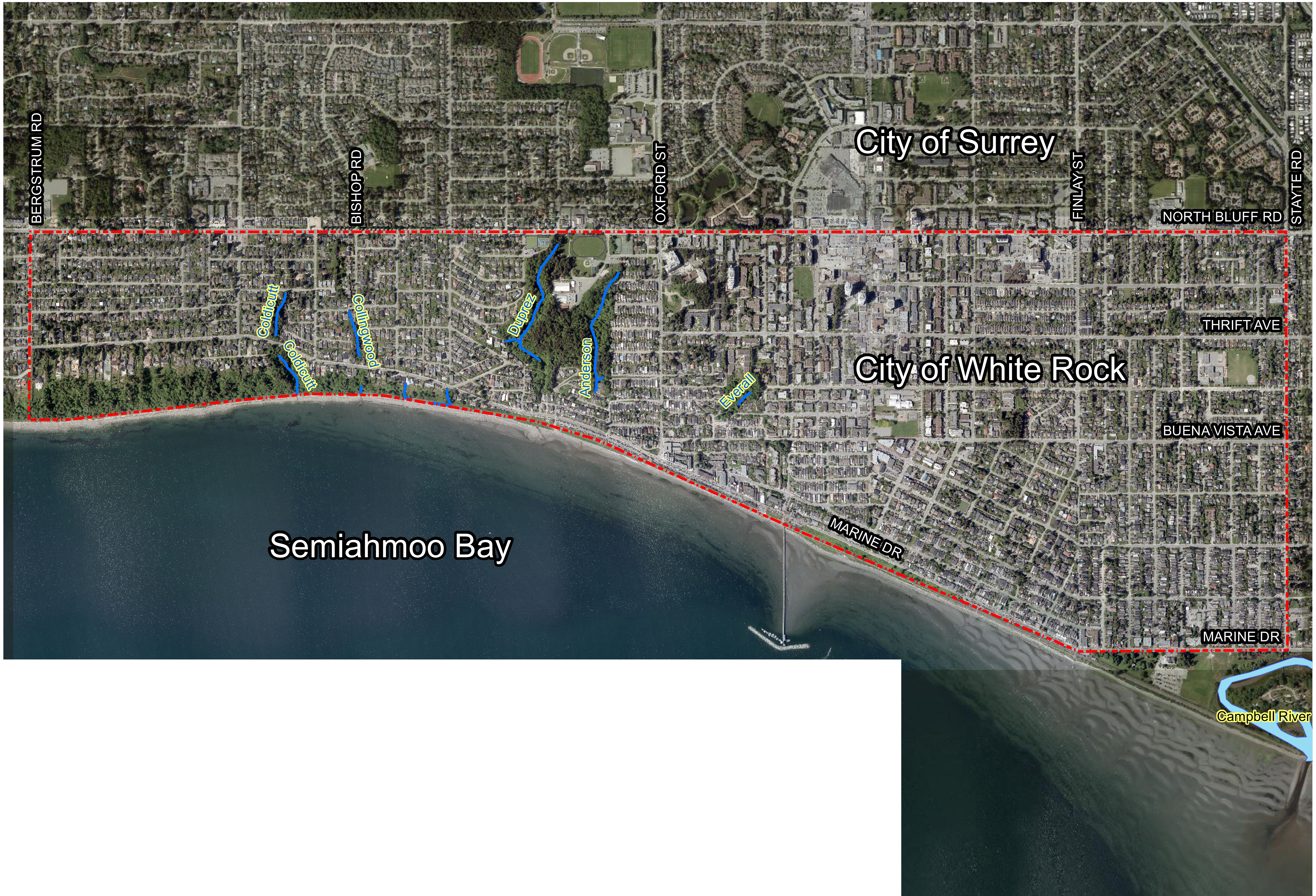
FIGURE 1.1



Legend

- Study Area
- Streams/Ravines
- River

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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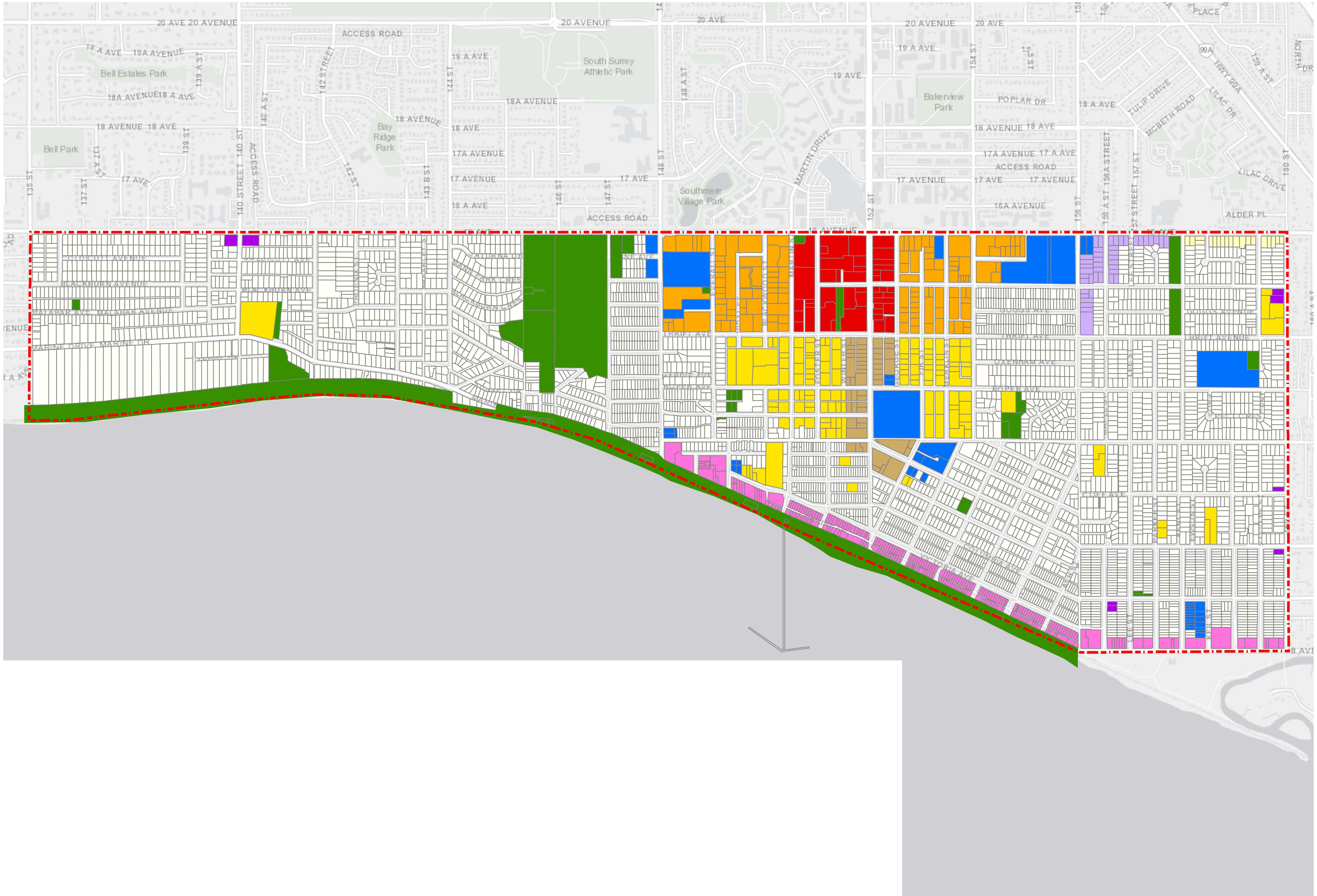
Drainage Master Plan Update 2019

Study Area

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

FIGURE 1.2



Service Layer Credits: Esri, HERE, Garmin, © OpenStreetMap contributors, and the GIS user community

Legend

Study Area

2017 OCP Zoning

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

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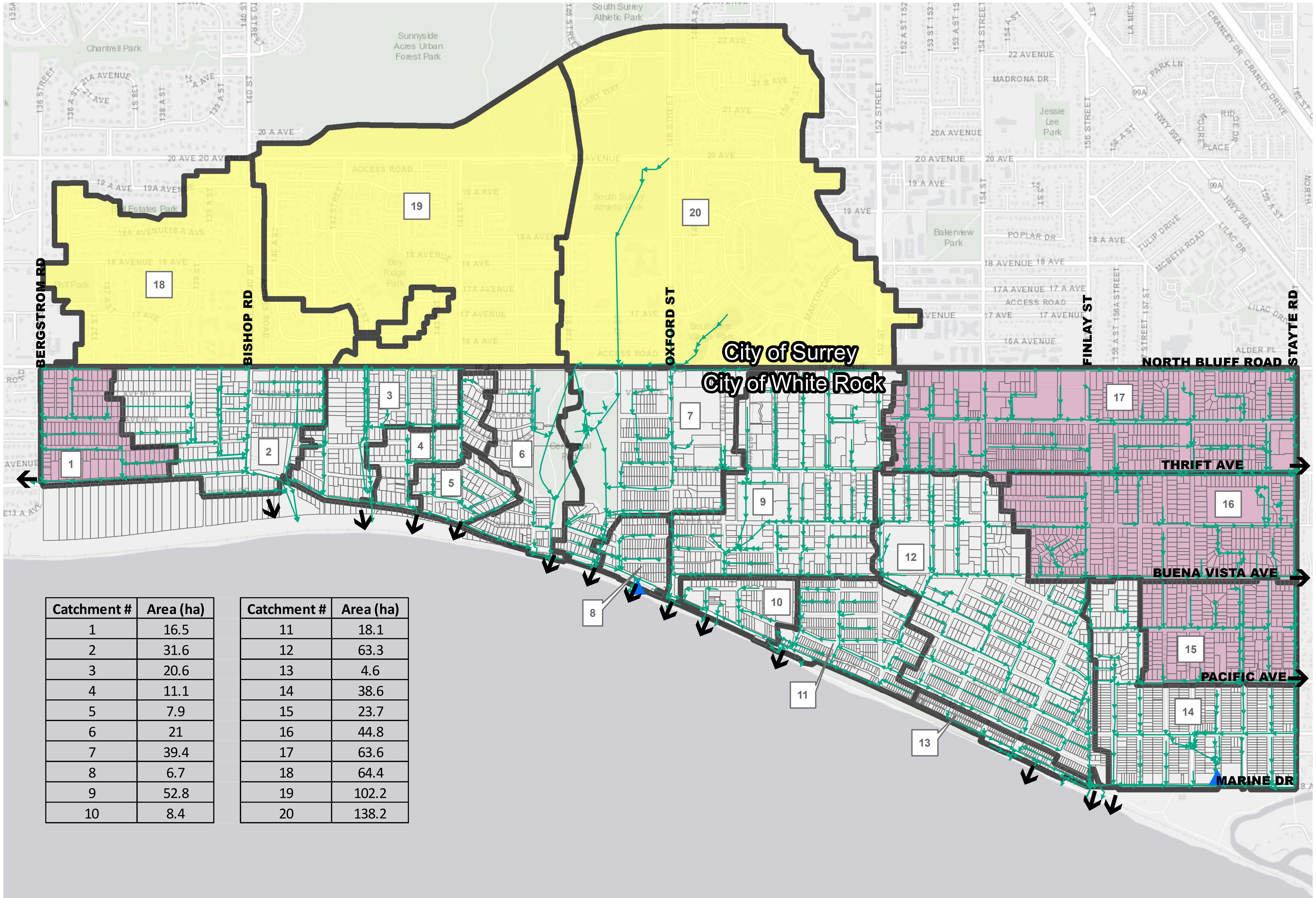
Drainage Master Plan Update 2019

2017 Official Community Plan

ISL Engineering and Land Services

FIGURE 1.3



Catchment #	Area (ha)
1	16.5
2	31.6
3	20.6
4	11.1
5	7.9
6	21
7	39.4
8	6.7
9	52.8
10	8.4

Catchment #	Area (ha)
11	18.1
12	63.3
13	4.6
14	38.6
15	23.7
16	44.8
17	63.6
18	64.4
19	102.2
20	138.2

Legend

- Storm Catchment From Surrey
- Storm Catchment to Semiahmoo Bay
- Storm Catchment to Surrey
- Modelled Storm Sewers
- Pume Station

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

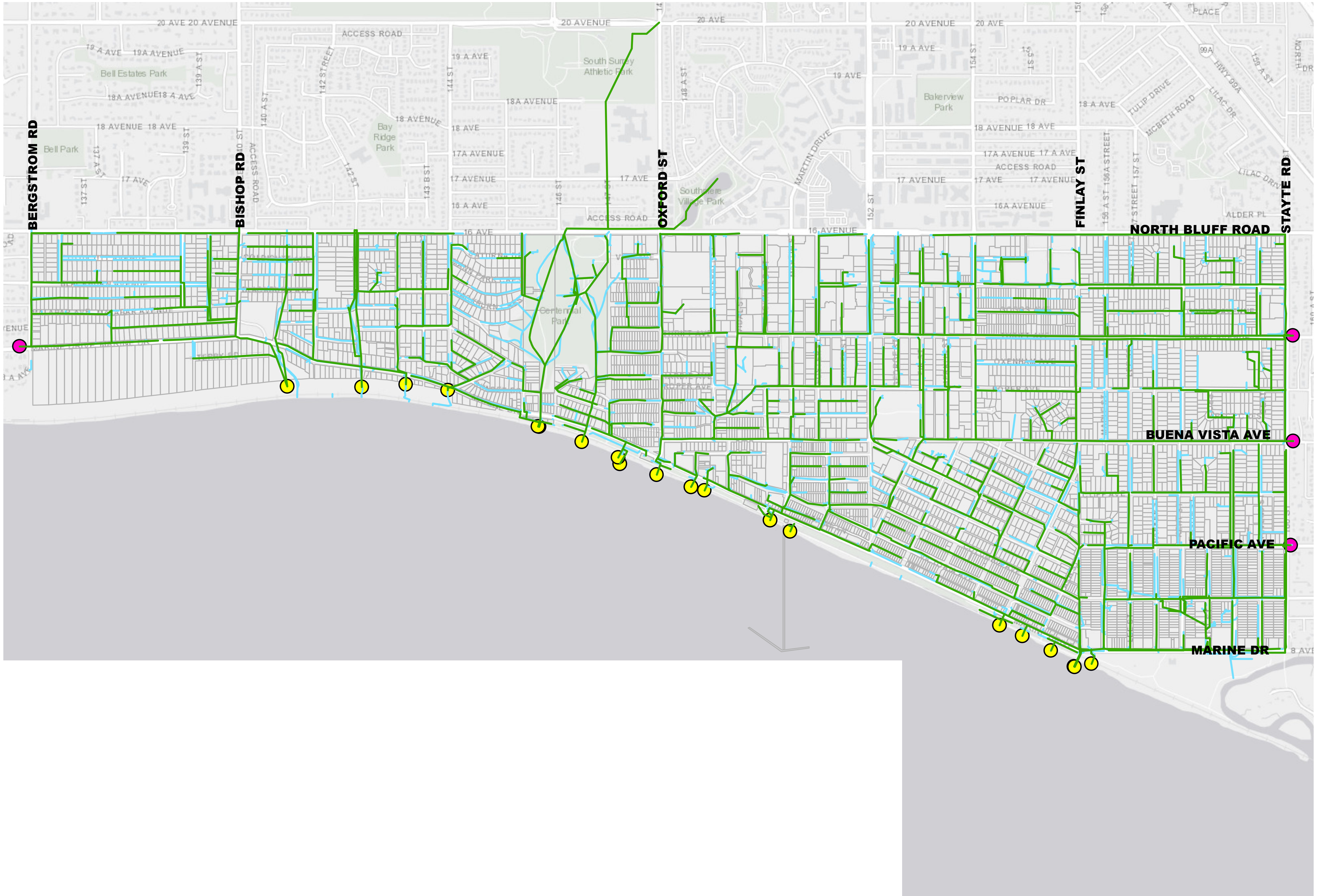
WHITE ROCK *City by the Sea!*

Drainage Master Plan Update 2019

Major Drainage Catchments

ISL Engineering and Land Services

FIGURE 1.4



Legend

- Pipes Not in Model
- Modelled Storm Sewers
- Outfalls to Semiahmoo Bay
- Outfalls to Surrey

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0 0.1 0.2 0.4 0.6 0.8 km

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Drainage Master Plan Update 2019

Modelled Storm Sewer System

ISL Engineering and Land Services

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Date: 2018-12-04

2.0 Model Update

XPSWMM version 2018.1 was selected as the software to complete the model update. The previous drainage model was developed with an older version of XPSWMM by AECOM as part of the 2012 Drainage Master Plan Update.

The drainage model was updated to incorporate new drainage infrastructure constructed in recent years. These new infrastructure were identified by comparing the City's GIS databases with the 2012 model. Additional upgrades that were not yet updated in the databases were identified through record drawings provided by the City. The record drawings were also used to verify the accuracy of the GIS data if it appears to be erroneous. Sub-catchments were maintained from the previous model with impervious coverage data updated to reflect parcel or land use changes as a result of recent developments.

2.1 GIS Data Review

During the process of updating the model's drainage network, both the GIS data and the 2012 model were reviewed to identify and correct "data gaps" (i.e. erroneous or missing data).

In the storm mains database, each pipe segment had an upstream and a downstream diameter. A common data gap in the storm mains database is pipes with missing diameters (either upstream, downstream, or both). Within these pipes, the ones that had to be added to the model contained either an upstream or downstream diameter, and the available diameter was assumed to be the diameter of the pipe segment.

Another data gap in the storm mains database was that some pipe segments were missing inlet and/or outlet invert elevations. A majority of these were small local sewers, upstream ends of pipes, or sewer connections that were not included in the model. For the ones that had to be added to the model, the invert elevations were either obtained from record drawings or were estimated based on inverts of the connected pipes.

In the 2012 model data, there were a few apparent errors in pipe invert elevations, such as two connecting pipes having the same inlet and outlet elevations. These errors were corrected using either record drawings or with estimated inverts based on upstream and downstream conditions. A model connectivity gap was also identified where the downstream end of a storm main was not connected to an outfall. This was corrected with data in the GIS.

There were over 20 inactive links in the 2012 model connecting pipes that were not linked in the GIS. These pipes had a small diameter (50 mm) and were very likely not part of the simulation. All inactive links from the previous drainage model were removed.

2.2 Hydrologic & Hydraulic Parameters

The drainage model has two components: the hydrologic model and the hydraulic model. The hydrologic model simulates runoff based on rainfall data, catchment characteristics, soil characteristics and other hydrologic parameters. It determines the amount of rainfall that becomes surface runoff and the travel time from sub-catchments to local sewers. The runoff gets routed in the drainage network in the hydraulic model. The simulation in the hydraulic model determines the capacity, peak water levels, velocities and flow rates in the drainage system.

Catchment data and characteristics were maintained from the 2012 drainage model with the exception to update catchment imperviousness in areas that were recently redeveloped. The imperviousness for redeveloped areas was determined based on the land use and these were consistent with the 2012 study. Table 2.1 lists the existing land use imperviousness from the 2012 study.

Table 2.1: Existing Land Use Imperviousness

Land Use	Imperviousness
Single Family Residential	30-55%
Single Family Residential (Roof Leaders Disconnected)	18%
Multi-Family Residential	60%
Commercial & Institutional	90%
Open Space and Recreation Areas	10%
Roads	90%

Hydrologic parameters were maintained from the 2012 study because model calibration was not completed for this model update. Table 2.2 lists the hydrologic parameters used in the model.

Table 2.2: Model Hydrologic Parameters

	Impervious	Pervious
Depression Storage, mm	0.5	5.0
Manning's Roughness, n	0.11	0.3
Max Infiltration Rate, mm/hr	-	45
Min Infiltration Rate, mm/hr	-	2.5
Infiltration Decay Constant, 1/hr	-	0.0009

Hydraulic parameters were also maintained from the 2012 study. Most of the storm mains in the model are either concrete or PVC. Table 2.3 lists the hydraulic parameters of the drainage model.

Table 2.3: Model Hydraulic Parameters

Pipe Material	Manning's "n"
PVC, HDPE	0.011
Concrete, Steel	0.013
Open Ditch	0.024

2.3 Future Impervious Coverage

Future development is expected to introduce additional impervious area to the City, as a result, catchment imperviousness was adjusted upwards for the assessment of the drainage system under future conditions. An impervious coverage was assigned to each one of the 11 land use designations in the OCP and they are listed in Table 2.4. If an existing sub-catchment covered parcels of more than one land use designations, the imperviousness for the sub-catchment was determined based on the approximate land area proportion of each land use within the sub-catchment. Figure 2.1 shows the catchment imperviousness under future OCP conditions. As shown in Figure 2.1, the impervious area is the highest in Town Centre, and decreases outwards in Lower Town Centre and Town Centre Transition as described in the OCP. The increase in catchment imperviousness between the existing and future OCP condition is also provided in Figure 2.2.

Table 2.4: Future OCP Land Use Impervious Percentage

Land Use	% Imp
Town Centre	95%
Town Centre Transition	70%
Lower Town Centre	90%
Waterfront Village	90%
Urban Neighbourhood	70%
North Bluff East	60%
Mature Neighbourhood	25%-45%*
East Side Large Lot Infill	60%
Neighbourhood Commercial	95%
Institutional	90%
Open Space & Recreation	10%

***Note:** The range of impervious coverage in Mature Neighbourhood is based on the existing lot type (i.e. lots with disconnected roof leaders or estate lots). See Table 2.5 for details.

The future imperviousness for each OCP land use designation was assigned based on the development descriptions in the OCP, under 22.0 Development Permit Areas. For Mature Neighbourhood, it was assumed (based on market condition from the past 5 years) that an average of 60 lots will be redeveloped per year and the redeveloped lots will have an average impervious coverage of 60%. For estate lots, a maximum impervious coverage of 35% was assumed. Based on the percentage of lots that will be redeveloped over the 30 year (OCP) period, an average impervious coverage for parcels in Mature Neighbourhood was calculated. Table 2.5 provides a summary of the impervious coverage in Mature Neighbourhood at existing, and over a 5- and 10-year period.

Table 2.5: Impervious Coverage in Mature Neighbourhood

Existing % Imp (2012 Master Plan)	Area (Ha)	% Imp of Redeveloped Lots	% Imp Over 5 Year Period	% Imp Over 30 Year Period (OCP)
18	193	60	22	40
18 (estate lots)	5	35	20	25
30	17	60	33	45

It was also assumed that future development will generally result in a higher (or equal) impervious coverage. For areas where the future zoning resulted in a lower impervious percentage in a sub-catchment, it was adjusted back to the existing percentage.

2.4 Rainfall and Flow Monitoring Data

The City does not currently have any permanent flow monitor stations set up. Flow monitoring data were not available to complete model validation for this study.

2.5 Model Validation

Model validation could not be completed for this study as flow monitoring data were not available. However, a recent (2017) and a historical (1999) storm were both simulated to compare the model results with the City's knowledge of flooding extents. The rainfall data were obtained from Environment Canada.

To simulate the model as close to actual conditions as possible, peak tidal elevations during the two simulated events were obtained from Fisheries and Oceans Canada at the Stevenson's Station. These elevations were added to outfalls in the model to simulate potential backwater effects from high tides during the actual storms.

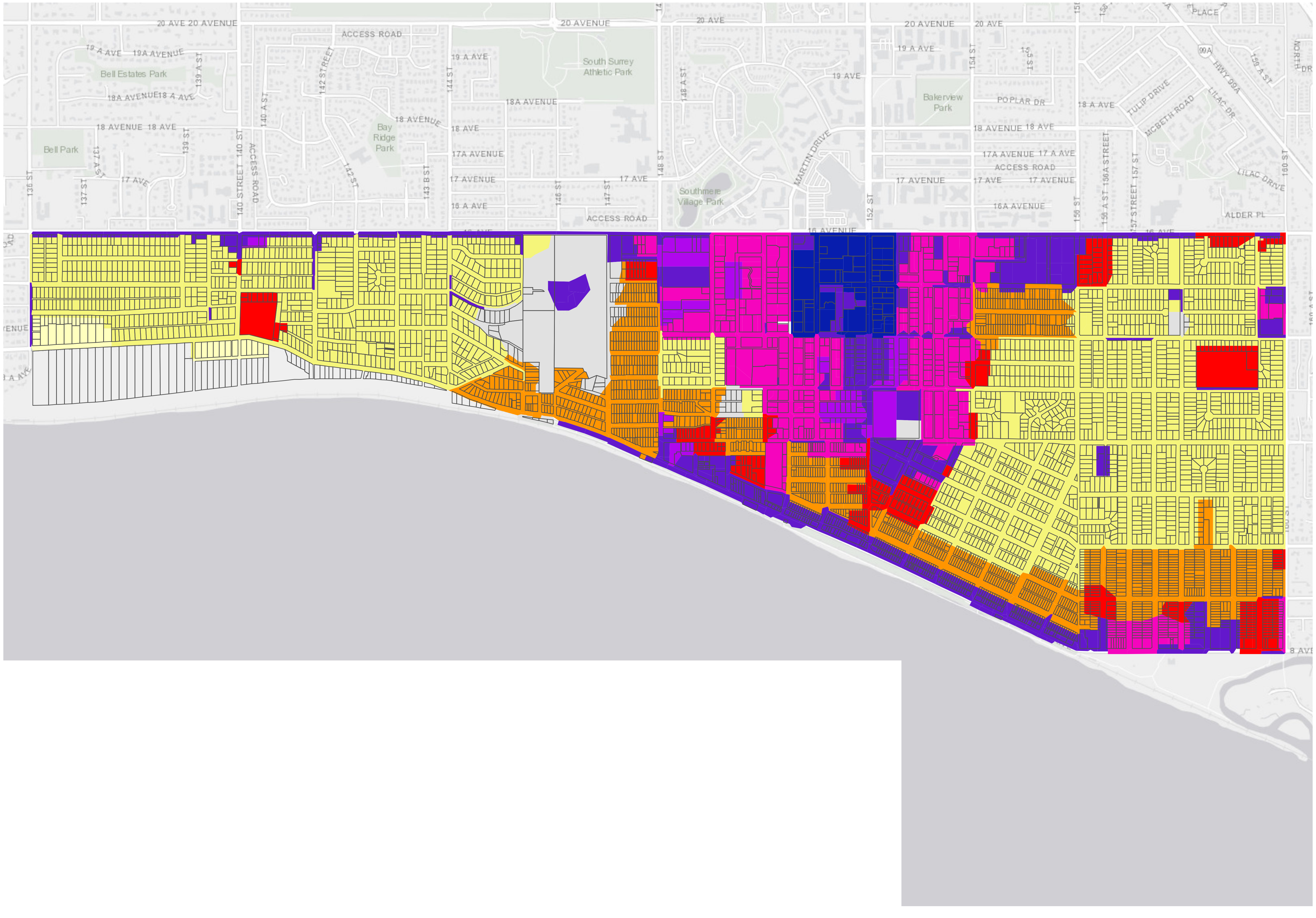
1999 Event (simulation period: December 5th – 15th, 1999)

The City provided photographs showing flooded locations from this event. It was known that the Campbell River was flooded during this event which likely contributed to the flooding within the City. Since river flooding could not be replicated in this model, the simulated results could not verify actual conditions for this event. Figure 2.3 shows results simulated under the 1999 event.

2017 Event (simulation period: October 6th – 9th, 2017)

A recent major storm event occurred in October, 2017. One of the known flooding locations from this event was along Marine Drive. Results simulated in the model also indicates flooding along Marine Drive and this can be visualized in Figure 2.4. Additionally, runoff from surcharged pipes north of Marine Drive likely flowed overland and contributed to the flooding on Marine Drive.

FIGURE 2.1



Legend

- 10%
- 25%
- 40-45%
- 50%
- 60%
- 70%
- 80%
- 90%
- 95%

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Drainage Master Plan Update 2019

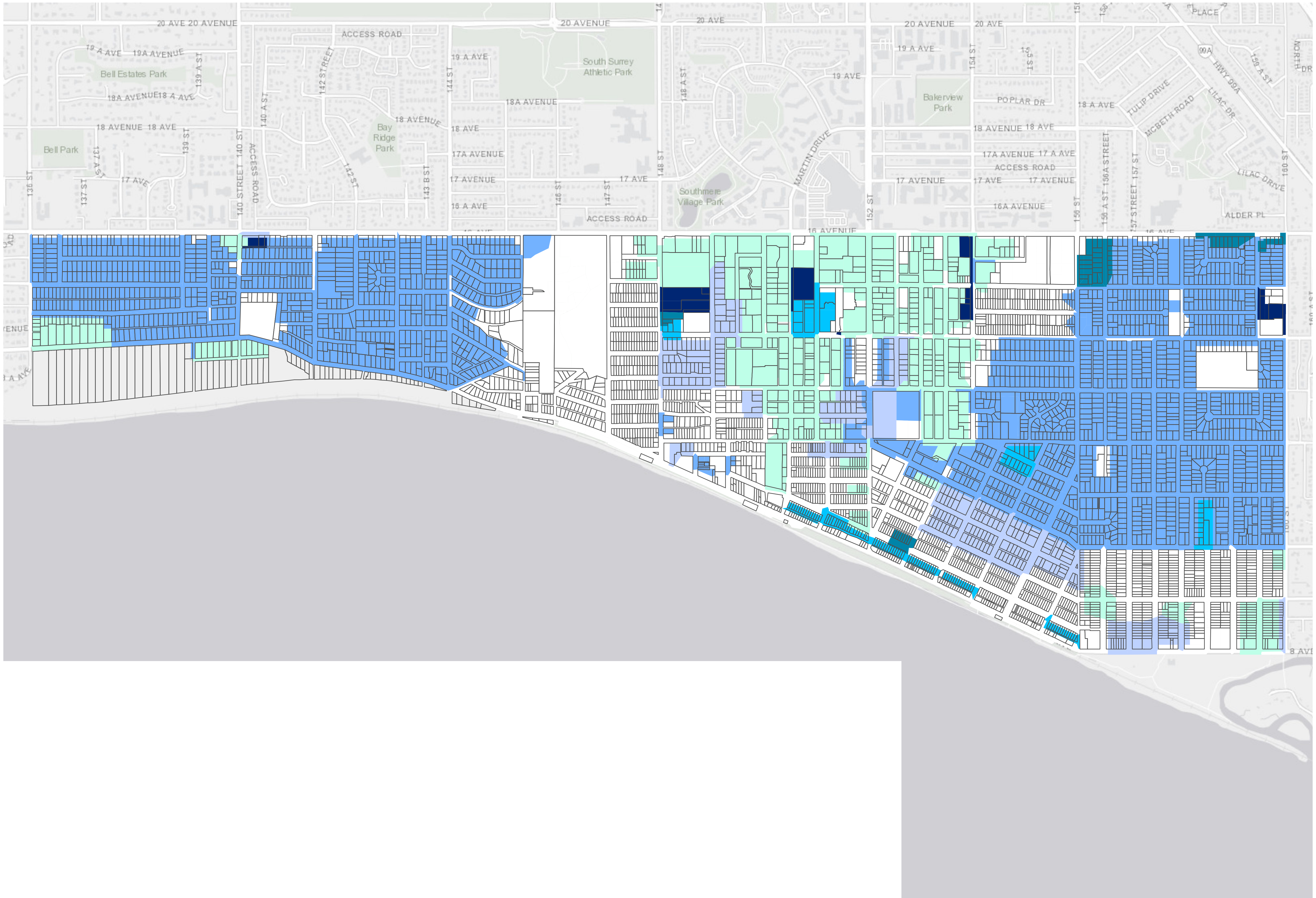
Future Percent Impervious of Model Sub-Catchments

ISL Engineering and Land Services

Document: J:\30000\31900\31939_White_Rock_Drainage-Sewer_MP\02_CADD\05_GIS\251_Figures\Drainage\Future Impervious.mxd

Date: 2018-12-04

FIGURE 2.2



Legend

- No Change
- <10%
- 10% - 20%
- 20% - 30%
- 30% - 40%
- 40% - 50%
- >50%

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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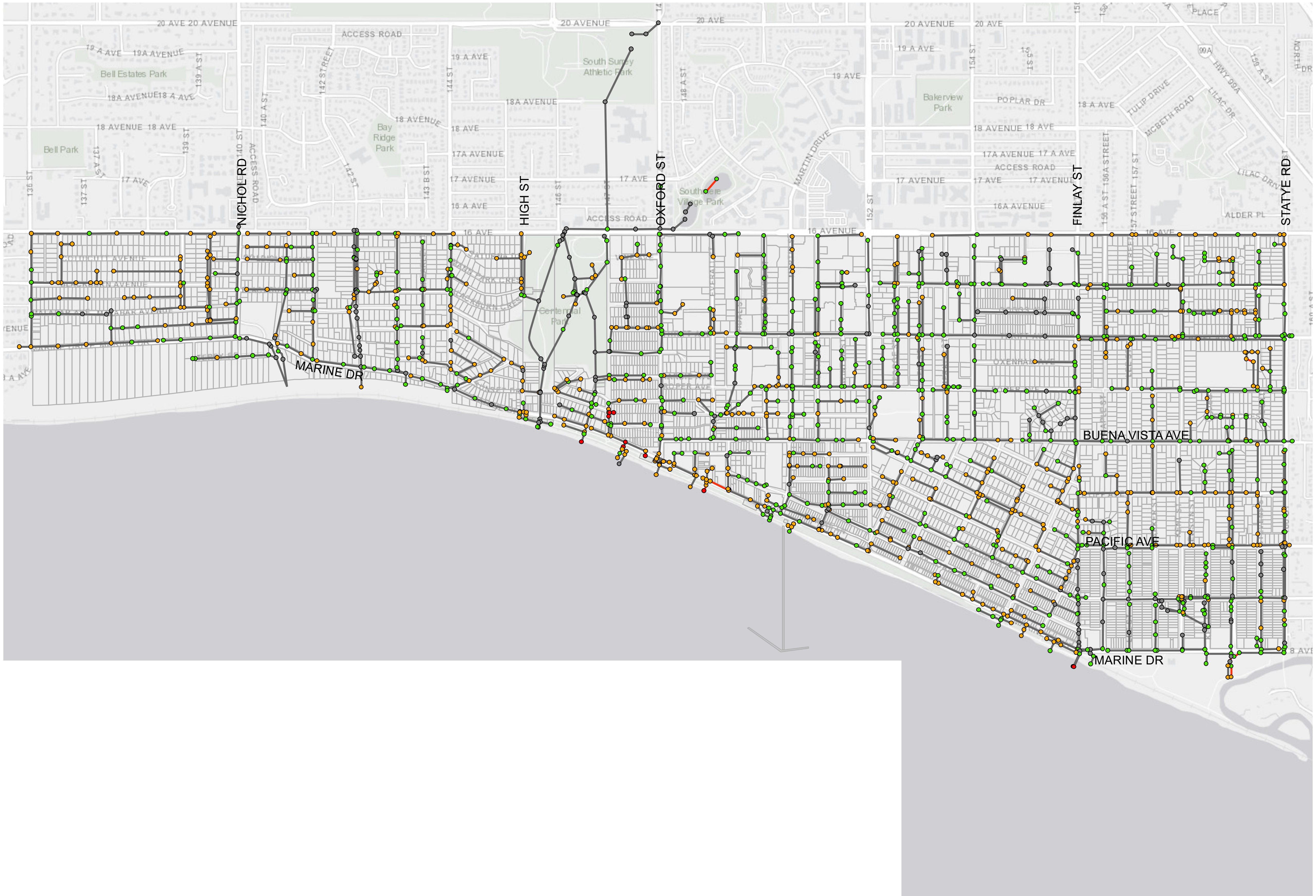
Drainage Master Plan Update 2019

Increase in Percent Impervious from Existing to Future OCP Conditions

ISL Engineering and Land Services

Document: J:\30000\31900\31939_White_Rock_Drainage-Sewer_MP\02_CADD\25_GIS\251_Figures\Drainage\Figure 2.2 - Increase in Percent Impervious from Existing to Future OCP Conditions.mxd
Date: 2018-12-04

FIGURE 2.3



Legend

Freeboard, HGL

- HGL above ground
- HGL between 0 - 1.2m
- HGL between 1.2m - 2.4m
- HGL >2.4m below ground

Max Flow/Design Flow

- Less than 0.85
- 0.85 - 1.0
- Greater than 1.0

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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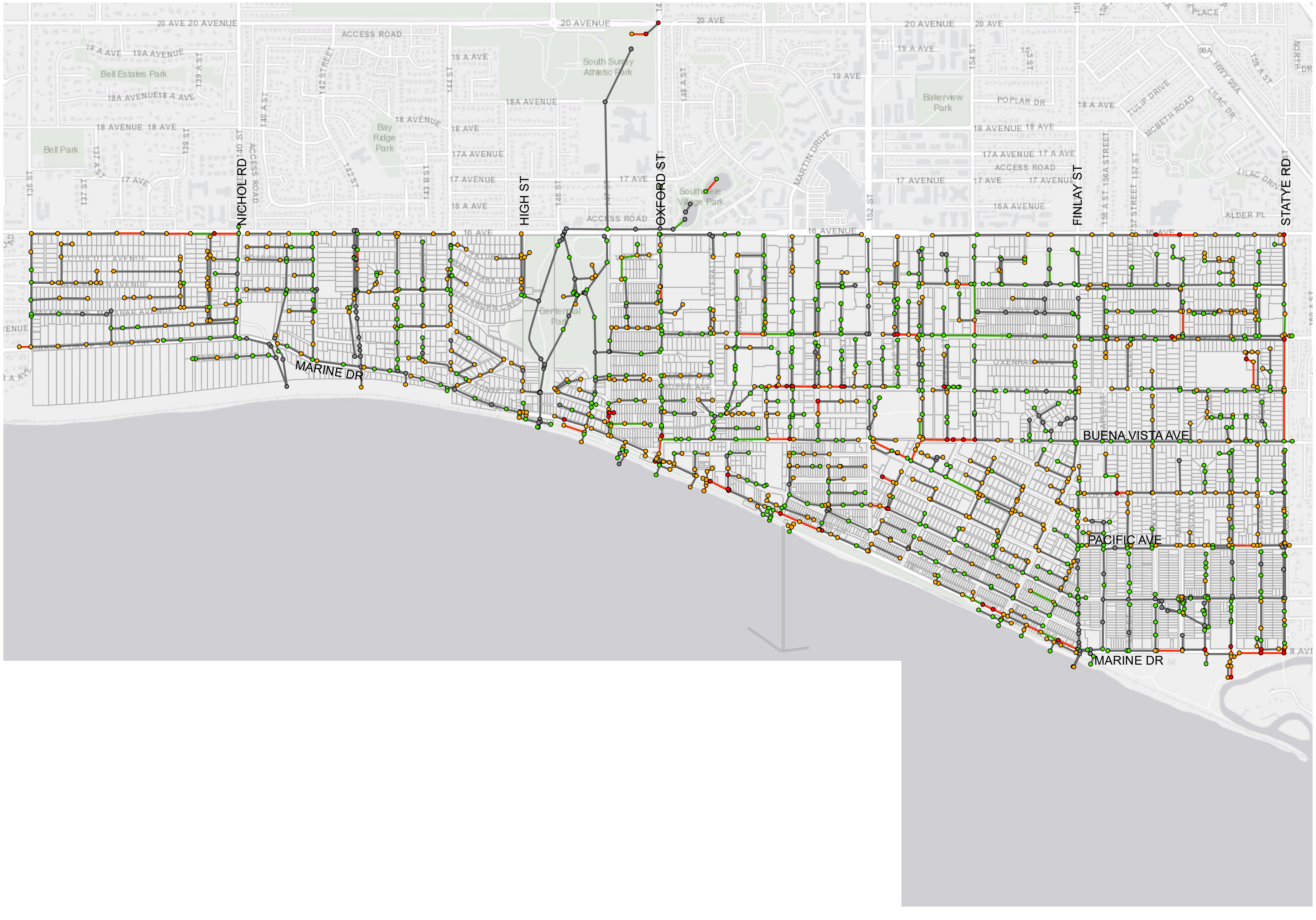
Drainage Master Plan Update 2019

Model Simulation
December 1999 Event

ISL Engineering and Land Services

FIGURE 2.4



Legend

Freeboard, HGL

- HGL above ground
- HGL between 0 - 1.2m
- HGL between 1.2m - 2.4m
- HGL >2.4m below ground

Max Flow/Design Flow

- Less than 0.85
- 0.85 - 1.0
- Greater than 1.0

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Drainage Master Plan Update 2019

Model Simulation
October 2017 Event

ISL Engineering and Land Services

3.0 Design Criteria

The City's current design criteria for new developments is provided in Subdivision Bylaw 777 Schedule B. The Bylaw requires that the minor drainage system shall be designed to convey flows of 5-Year frequency for Storm Water Management design and 10-Year frequency for Conventional design. The major system is to be designed for flows of 100-Year frequency. The Storm Water Management design method limits the post development peak runoff by surface infiltration, and detention facilities, etc. The Bylaw states that ideally, it shall be used for all comprehensive developments and sometimes for conventional developments. The Conventional design is based on the Rational Formula.

The City does not currently have a published design criteria for retrofitting the existing drainage system. Replacement of all surcharged pipes under the existing design criteria (which is for new developments) is not required as it is not cost effective. For consistency with the previous studies, the City's drainage system was assessed under the 10-Year 1-Hour event. Assessment of overland flow under the 100-Year event was not within the scope of this project.

4.0 Evaluation of the Drainage System

4.1 Assessment Criteria

The existing drainage system capacity was assessed based on a combination of the Qpeak/Qcapacity and the hydraulic grade line (HGL) in the system under the 10-Year 1-Hour event. The 1-Hour event was determined to result in the highest flows in the system compared to the 2, 6, 12, and 24 hour durations.

The ratio of peak flow to maximum design flow (Qpeak/Qcapacity) was divided into five groups. Generally, pipes with Qpeak/Qcapacity less than 1.25 were not recommended for upgrades. However, if the HGL in the system is at a level that risks flooding, then an upgrade would be recommended. The five groups are listed as follows:

- Qpeak/Qcapacity < 0.80
- Qpeak/Qcapacity between 0.8 and 1.0
- Qpeak/Qcapacity between 1.0 and 1.25
- Qpeak/Qcapacity between 1.25 and 1.50
- Qpeak/Qcapacity greater than 1.50

The HGL shows the peak water level in the system and provides an indication of the extent of surcharge in the system under the simulated event. In this study, the HGL was measured with reference to the ground level at nodes in the model. A node with HGL above ground level is an indication that the system will surcharge to the ground at that location under the simulated event. To show the water levels in the system, the HGL was divided into four categories listed as follows:

- HGL above ground
- HGL <1.2 m below ground
- HGL between 1.2 m and 2.4 m below ground
- HGL >2.4 m below ground

In general, an upgrade was recommended where the Qpeak/Qcapacity in a pipe is greater than 1.25 and the HGL is less than 1.2 m below ground under the 10-Year 1-Hour event. For a storm system, a slightly surcharged pipe is not at a high risk as long as the HGL is below service connections and ground level. It would be considered over conservative to upgrade all pipes at risk of surcharging given the City's limited capital budget. Pipes with Qpeak/Qcapacity greater than 1.0 but less than 1.25 were identified as optional upgrades for when the City has available budget. The City can also choose to complete these upgrades with future development as deemed necessary.

4.2 System Capacity Assessment (Future Scenario)

The future scenario in this study was modelled differently from the 2012 study. As explained in Section 2.3, the catchment impervious percentage in the "future" model was adjusted upwards to reflect the increase in impervious area as a result of future developments. In the previous study, instead of adjusting the catchment imperviousness, the catchment width was increased to simulate higher peak flows. (The catchment imperviousness was only adjusted where there was going to be a significant change in the OCP land use.) Since the majority of the land area in the City is already developed, future infill and redevelopment activities would not affect the sub-catchment sizes significantly. Whereas in suburban areas, development would result in smaller sub-catchments and thus reduced time of concentration from each sub-catchment to the drainage system. For a developed city like White Rock, adjusting the impervious percentage of sub-catchments based on land use designations and development activities would have a better prediction of future peak flows in the system than increasing all the sub-catchment widths in the model.

4.3 Assessment Results

The drainage system capacity was assessed under the 10-Year 1-Hour storm event. The assessment was completed under both existing and future OCP conditions, shown in Figure 4.1 and 4.2. The results are also summarized in Table 4.1 and 4.2. The models were simulated assuming free outfall conditions. In a major event with high tide conditions, there will be backwater effects where the outfalls are lower than tide levels and the overall capacity of the drainage system will be affected by the height of the tide.

Table 4.1: Sewer Capacity Summary

	Existing Condition		Future OCP Condition	
Qpeak/Qcapacity	**Number of Pipe Segments**	**Length (m)**	**Number of Pipe Segments**	**Length (m)**
<0.8	1,266	66,395	1,152	60,314
0.8 – 1.0	70	3,927	117	6,156
1.0 – 1.25	58	3,030	98	5,717
1.25 – 1.5	31	1,572	42	2,216
>1.5	45	1,179	61	1,700

Table 4.2: HGL at Model Nodes

	Existing Condition	Future OCP Condition
Depth Below Ground	**Number of Nodes**	**Number of Nodes**
>2.4 m	163	159
1.2 – 2.4 m	622	597
<1.2 m	641	635
Above Ground	57	92

The increase in the number of undersized pipes and surcharged nodes under the future scenario is due to increased runoff volume and peak flow as a result of increasing the percent impervious of sub-catchments. These undersized pipes are primarily located in Mature Neighbourhood areas as most of the area was assumed to have an increase in impervious coverage due to future redevelopment or infill activities.

Note that Tables 4.1 and 4.2 may not be a complete representation of undersized pipes and surcharged locations. As a surcharged pipe gets upgraded in the system, the flows are no longer restricted in that location and sometimes the higher flows resulted in the pipe downstream to be under capacity.

Table 4.3 provides the capacities of the Oxford and Habgood pump stations and the modeled inflows to each pump station under the 10-Year 1-Hour event.

Table 4.3: Modelled Peak Inflow to the City's Pump Stations

Pump Station	Capacity	10-Year Peak Inflow Under Existing Condition	10-Year Peak Inflow Under Future Condition
Oxford	40[1] L/s	248 L/s	248[3] L/s
Habgood	582[2] L/s	647 L/s	757 L/s

Notes:
1. Capacity based on one pump running
2. Design capacity provided in the preliminary design report
3. A peak inflow of 330 L/s was modeled in the previous study

As indicated in the 2012 study, the Oxford Pump Station is under capacity. There is a bypass pipe located at this pump station to direct overflows to the beach and prevent the parking area from flooding. Under both existing and future conditions, 231 L/s of flow bypass the pump station through a 600 mm pipe that outfalls to the beach. The model was simulated with free flow conditions at the outfalls. In a major storm during high tide, there may be backwater flow in the bypass causing the parking area to flood. The previous study also modelled a peak inflow of 430 L/s in a 100-Year storm.

It was also noted that the 2012 model had a storage node upstream of the Oxford Pump Station on Marine Drive, and the storage was removed in their final model. This storage was ignored as it was not in the final model and there was no evidence of physical storage.

The Habgood Pump Station is currently being designed for relocation with a design capacity of 582 L/s designed to eliminate flooding, according to AECOM's Preliminary Design Report completed in 2017. Based on the flow rates simulated from the model under the 10-Year 1-Hour event, the pipes upstream of the pump station may surcharge.

4.4 Condition Assessment

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

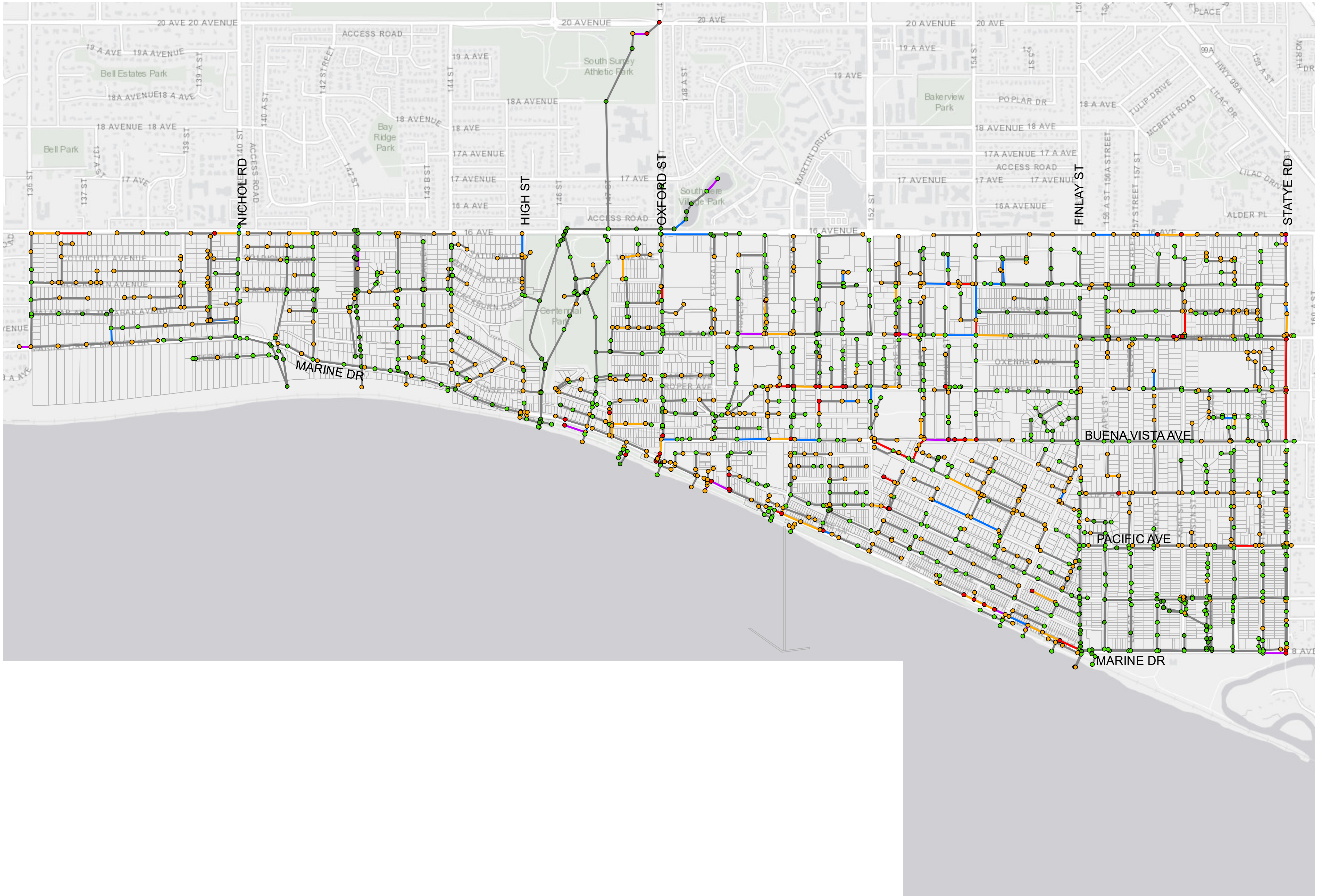
- AECOM Area C Spring Flushing CCTV Memo, 2017
- Binnie Area D & E CCTV Inspection Program Assessment and Evaluation Report, 2017
- AECOM Area B Spring Flushing CCTV Memo, 2016
- AECOM Area A Spring Flushing CCTV Memo, 2015

These reports cover Areas A, B, C, D and E of the City (shown in Figure 4.3). In these reports, AECOM and Binnie reviewed CCTV inspections and provided conditional assessments that ranked storm lines on a scale of 1.0 (Best/Very Good) to 5.0 (Worst/Very Poor). As part of the condition assessments, storm lines were reviewed for surface defects such as longitudinal and circumferential cracking, deformities, offset joints, and broken pipe. In addition, the storm lines were also reviewed for operational and maintenance defects which included debris obstruction and root intrusion.

A summary of the location of storm pipes with structural and service defects as per the CCTV assessments is shown in Figure 4.3. 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 storm pipes with structural and maintenance condition rating of 3 or higher are shown in Appendix C. Condition upgrades required as a result of the additional assessment were not included under Section 5.0 Recommended Capital Work. The City should consider the additional upgrades that may be required during capital planning.

FIGURE 4.1



Legend

Freeboard, HGL

- <0
- 0 - 1.2
- 1.2 - 2.4
- >2.4

Max Flow/Design Flow

- <85%
- 85% - 100%
- 100% - 125%
- 125% - 150%
- >150%

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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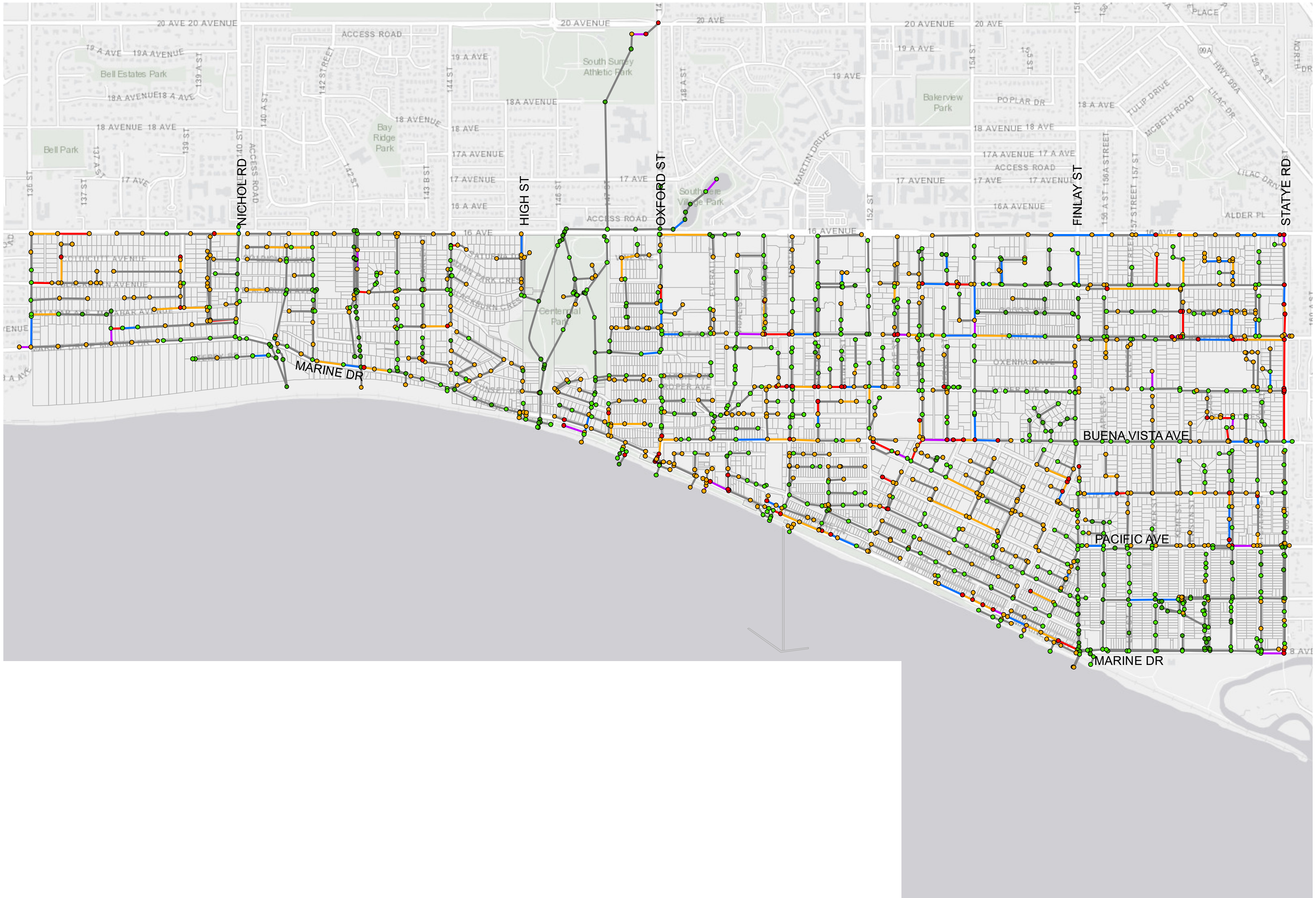
Drainage Master Plan Update 2019

Capacity Assessment of Existing System Under Existing Condition (10-Year 1-Hour Event)

ISL Engineering and Land Services

FIGURE 4.2



Legend

Freeboard, HGL

- <0
- 0 - 1.2
- 1.2 - 2.4
- >2.4

Max Flow/Design Flow

- <85%
- 85% - 100%
- 100% - 125%
- 125% - 150%
- >150%

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

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Drainage Master Plan Update 2019

Capacity Assessment of Existing System Under OCP Conditions (10-Year 1-Hour Event)

ISL Engineering and Land Services

FIGURE 4.3

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

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

WHITE ROCK City by the Sea!

Drainage 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

Date: 2018-12-05 Document: J:\30000\31900\31939_White_Rock_Drainage-Sewer_MP\02_CADD\25_GIS\251_Figures\Drainage\Pipe Condition.mxd

5.0
Recommended Capital Work

5.1 Flow Diversions

In the 2012 study, a diversion trunk on Parker Street was proposed to control peak flows within White Rock and only discharge existing flows and base flows to the City of Surrey and McNally Creek. In this master plan update, the City had asked ISL to determine the possibility of diverting all the 10-Year flows so as to contain them within White Rock. This study compared the following three options:

- Option 1: No diversions (existing condition with three outfall locations on Stayte Road)
- Option 2: Partial diversion (same as the Parker Street Diversion design proposed in 2012)
- Option 3: Full diversion (complete diversion of 10-Year peak flows currently outflows to Surrey on Stayte Road)

A summary of 10-Year peak flow rates at various outfall locations is provided for the three options in Table 5.1. Options 2 and 3 are discussed below.

Table 5.1: Peak Flows at Outfalls Under Different Model Scenarios

Development	Upgrading	Diversion	Peak Flows in L/s				
			Flows to Surrey				Flow to Semiahmoo Bay @Finlay St (7485)
			Thrift Ave (P20)	Buena Vista Ave (P17)	Pacific Ave (P16)	Total	
Existing	No	No	1,059	651	167	**1,877**	1,521
OCP	No	No	1,325	1,021	227	**2,573**	1,774
	Yes	No –Opt 1	1,942	1,993	351	**4,286**	1,862
	Yes	Partial – Opt 2	2,043	847	300	**3,190**	2,761
	Yes	Full –Opt 3	0	0	0	**0**	6,258

Option 2: Partial Diversion (Proposed as the Parker Street Diversion in the 2012 study)

This diversion was proposed in the 2012 study to control peak flows within White Rock. The strategy of this diversion was to maintain flows up to the 10-year event to Surrey while managing high flows (100-Year) and any additional flows within White Rock.

The existing outfalls on Buena Vista Avenue and Pacific Avenue both drain into McNally Creek and to Little Campbell River. Due to erosion issues, no increase in flows to these locations were recommended. There is an existing connection on Stayte Road at Thrift Avenue that allows some flow to bypass the outfall on Thrift Avenue to the outfall on Buena Vista Avenue. This connection would be removed as part of this diversion design to reduce flows that enter McNally Creek.

The proposed diversion alignment is shown in Figure 5.1. The proposed diversion trunk is on Parker Street between Thrift Avenue and Pacific Avenue. Flows on Thrift Avenue and Buena Vista Avenue, west of Parker Street are partially diverted south through this diversion trunk, while flows on Roper Avenue and Cliff Avenue are completely captured by this trunk. The diversion trunk connects to the existing pipe on Pacific Avenue and Parker Street where the diverted flows will continue west from Parker Street to Maple Street, then south along Maple Street, and finally to the outfall near Finlay Street.

For this option, proposed upgrades to pipes that currently discharge to the Stayte Road outfalls (located in the east side of the City) are shown in Figure 5.2. Figure 5.2 also shows a comparison of upgrades needed in the same area if Option 1 (No Diversion) was to be selected. The pipe on Maple Street that was recently constructed as part of the East Beach project would be undersized with Option 2.

Option 3: Full Diversion

After discussions on inter-municipal flows between the City of White Rock and the City of Surrey, White Rock is now exploring the option of completely eliminating 10-year flows to Surrey and managing the flows within White Rock.

To achieve complete diversion, the current best option is to locate the diversion trunk on Stayte Road in order to capture all the flows to the three existing outfalls. The proposed alignment of the diversion is shown in Figure 5.1. The proposed diversion trunk starts along Stayte Road, from Thrift Avenue to Columbia Avenue, then flows are diverted east along Columbia Avenue from Stayte Road to Maple Street. On Maple Street, the proposed diversion trunk is connected to the existing pipe where flows will be diverted south and eventually to the outfall near Finlay Street. The total length of this diversion trunk from the Thrift Avenue to the outfall is just over 2000 m and the proposed size ranges from 825 mm to 1500 mm.

This option is assumed to be the preferred option as it completely captures the 10-Year peak flows within White Rock. The proposed upgrades with this option are shown in Figure 5.3. The proposed diversion would require a major length of the existing pipe on Stayte Road to be upgraded although some sections would require an upgrade regardless of the diversion. The proposed diversion would also replace the existing pipe along Columbia Avenue with a bigger diversion trunk.

Location of existing underground utilities and surveyed ground elevations need to be obtained during design to confirm the feasibility of the proposed alignment.

Upgrades and Cost Associated with the Three Options

The total cost of each option is provided in Table 5.2. The total cost in Table 5.2 includes the cost of constructing the diversion trunks (for options 2 and 3) and upgrades proposed for the City. The costs were determined based on the unit costs listed in Table 5.4.

Table 5.2: Cost Comparison of the Three Diversion Options

Option	Total Upgrading Cost including Diversion (in $1000)	Incremental Cost to Option 1 (in $1000)	Incremental Cost to Option 2 (in $1000)
1-No Diversion	$9,118	-	-
2-Partial Diversion	$10,482	$1,364	-
3-Full Diversion	$12,888	$3,770	$2,406

The majority of the incremental cost between the three options are the cost of constructing the diversion trunk or upgrading existing pipes along the diversion alignment. Other upgrades are generally similar but can be different in size between the three options based on the flows diverted.

5.2 Other Diversions Proposed for the Drainage System

In addition to the proposed diversion to manage flows within White Rock, there are opportunities for other diversions in the drainage network that could reduce of cost of pipe upgrades.

Roper Avenue Diversion

This is a 55 m diversion proposed on Foster Street, from Roper Avenue to the existing pipe to the south (shown in Figure 5.4). This 375 mm diversion pipe intercepts most of the flows at Foster Street and Roper Avenue that are currently conveyed through a 300 mm pipe towards Everall Creek. The 300 mm pipe on Roper Avenue west of Foster Street is undersized to convey the 10-Year peak flows under existing conditions. The diverted flows will continue south on Foster Street towards Buena Vista, then west along Buena Vista Avenue, and eventually to the outfall near Oxford Street.

This diversion reduces the length of pipe upgrades and prevents additional flows from discharging to the Everall Creek. Pipes on Foster Street and Buena Vista Avenue would require an upgrade regardless of the diversion.

Thrift Avenue Diversion

There are two parallel pipes on Thrift Avenue between Finlay Street and Stayte Road. The north pipe between Stevens Street and Stayte Road is surcharged under the existing scenario simulation. Under the future scenario, both pipes would be under capacity to convey the 10-Year peak flow. This diversion (shown in Figure 5.5) would intercept a portion of the flow from the north pipe to the south such that only the south pipe would need to be upgraded. Doing so will reduce cost and minimize road disruption. This diversion may be more ideal if Option 3 Full Diversion is selected. As otherwise this diversion introduces additional flow to the outfall at Buena Vista which will eventually discharge into Little Campbell River.

As a further note, this diversion is not needed until future years when the south pipe becomes undersized to convey the additional flow from increased impervious areas. Diverting the flows under existing condition will cause the south pipe to surcharge. Based on the simulation, the north pipe will not surcharge to ground under existing conditions during a 10-Year storm.

Pacific Avenue Diversion

This diversion (shown in Figure 5.6) is only recommended if Option 3 Full Diversion is selected as it would otherwise require a larger and longer section of pipe to be upgraded downstream of the diversion.

The 525 mm pipe on Pacific Avenue between Habgood Street and Stevens Street is under capacity to convey the 10-Year peak flow. This diversion intercepts a portion of the flow on Habgood Street and diverts it south towards Columbia Avenue. The diverted flows would cause a downstream section of pipe on Columbia Avenue to surcharge. However, with the full diversion option, the pipe on Columbia Avenue would be replaced by a larger diversion trunk and thus a lower cost compared to upgrading the pipe on Pacific Avenue.

5.3 Capacity Upgrades

Proposed capacity upgrades are shown in Figure 5.3. Upgrades will vary with the three diversion options for pipes that currently discharge to the Stayte Road outfalls. These pipes are located in the east side of the City. Figure 5.3 is shown with upgrades associated with Option 3 (Full Diversion). Upgrades associated with Option 1 (No Diversion) and Option 2 (Partial Diversion) are shown in Figure 5.2. Optional capacity upgrades for pipes with Qpeak/Qcapacity greater than 1.0 but less than 1.25 are also highlighted in Figure 5.3.

Note that for Options 1 and 2, the outfalls on Stayte Road were not included in the proposed upgrades as downstream conditions (in Surrey) were not modelled. As a result, the pipe upstream of the Thrift Avenue outfall may be surcharged under the 10-Year event, causing backflow to the upstream pipe on Statye Road and surface flooding near the Thrift Avenue and Statye Road intersection.

It was also noted that some pipes upstream of the outfalls on Marine Drive are reversely graded. An example is shown in Figure 5.7. These pipes may have been constructed to prevent seawater from entering the upstream system during high tides to prevent corrosion of the drainage pipes from salt. However, some of these reverse graded pipes can cause the upstream system to surcharge. It is recommended that instead of upgrading the surcharged pipes to a size larger than what is needed to convey the peak flows, flap gates can be installed to the outfall to prevent seawater from entering the drainage system.

Since the future scenario was modelled to the 30-Year OCP buildout, some of the proposed upgrades are at a much lower priority and will not be required until later in the years. If future development conditions did not meet those projected in the OCP (i.e. if actual developments in future years were much less than predicted), then some of the proposed upgrades would not be required.

To reduce the increase in peak flows from future developments, the City can consider incorporating stormwater control or reduction measures such as low impact development (LID) to the design of future developments. LID can significantly reduce peak flows and runoff volume in smaller storms, and can often improve the runoff quality with proper design and maintenance. Examples of LID measure that may be applicable to the City include rain gardens, green roofs, bio-swales, infiltrators, etc.

Storm Sewer on Nichol Road

The existing 750 mm storm sewer on Nichol Road conveys flow from a catchment area of 64 Ha from Surrey in addition to flows within White Rock. (The 64 Ha was lumped into a single catchment node in the model.) Under existing conditions, the pipe is capable of conveying the flows from both White Rock and Surrey. However, since future re-development in Surrey is unknown, the increase in peak flow from Surrey cannot be predicted. Based on a simulation of existing flows from Surrey and future condition flows from White Rock, this pipe would have enough capacity to convey the peak 10-Year flows. Additionally, another simulation was completed assuming a 10% increase in the overall catchment impervious area in Surrey (from the existing 55% to 65%). Results showed that the pipe would have capacity to convey the 10-Year peak flows. If higher peak flows will be expected from Surrey, then this pipe may need to be upgraded in the future. The cost of this upgrade, if necessary, should be paid for by the City of Surrey.

In the previous model, AECOM adjusted the model catchment width and resulted in an increase from 665 L/s to 1,648 L/s in the peak flow from Surrey to the 750 mm storm sewer on Nichol Road. This higher peak flow caused the 750 mm storm sewer to surcharge under the 10-Year event and an upgrade was proposed in the previous study. ISL disagrees with the previous study’s method in increasing modeled peak flow. Based on increasing the overall catchment impervious percentage by 10%, the modelled peak flow increased from 670 L/s to 691 L/s. Ultimately, to determine the capacity of the 750 mm storm sewer, the future discharge rate from Surrey to White Rock needs to be known.

Thrift Avenue and Kent Street

The previous model shows that there is only one pipe (250 mm) on Kent Street between Goggs Avenue and Thrift Avenue but GIS data shows two parallel pipes. The pipes near Kent Street and Thrift Avenue should be surveyed to confirm the accuracy of the GIS data and determine if the 250 mm should be upgraded.

5.4 Pump Station Upgrades

Oxford Pump Station

The Oxford Pump Station is under capacity to convey the 10-Year peak flows. Flows that exceed the pump station capacity would bypass to the beach through a 600 mm pipe. In the event of a major storm during high tide, the bypass pipe may have reduced capacity and result in surface flooding in the parking area. It is recommended to upgrade the pump station to reduce flooding risks.

As mentioned in the 2012 study, the pump station is very old. Prior to upgrades, the pump station should be structurally assessed and the electrical components should be inspected.

Habgood Pump Station

The design capacity of the new Habgood (Keil) Pump Station was selected to eliminate flooding during a 10-Year 1-Hour storm. Table 5.3 provides the modeled peak inflows to the pump station under different diversion options. Under improved conditions of Option 1 and Option 2, the pipes upstream of the pump station will surcharge under a 10-Year 1-Hour storm. If the City decides to increase the design criteria prior to finalizing the pump station relocation design (i.e. from eliminate flooding to eliminate surcharging) then the current design capacity of 582 L/s will need to be increased.

Table 5.3: Modelled 10-Year Peak Inflows to Habgood Pump Station

	Option 1 No Diversion	Option 2 Partial Diversion	Option 3 Complete Diversion
Peak Flow to PS	757 L/s	755 L/s	634 L/s

5.5 Capital Plan

The proposed drainage improvements were prioritized into a 10-Year Capital Plan based on pipe condition and surcharging risk. The Capital Plan is shown in Figure 5.8 with upgrade details provided in Appendix D. The Capital Plan was developed with consideration of the City's available capital budget although the annual budget in the next 5 years may need to be adjusted. The cost estimates were prepared based on the unit costs listed in Table 5.4. An additional 10% engineering fee and 25% contingency allowance were added to the total of each year's upgrade costs.

Table 5.4: Unit Costs for Upgrades

Size (mm)	Unit	Unit Cost
200	m	$ 770
250	m	$ 817
300	m	$ 878
375	m	$ 975
450	m	$1,059
525	m	$1,234
600	m	$1,319
675	m	$1,400

Size (mm)	Unit	Unit Cost
750	m	$1,585
825	m	$1,650
900	m	$1,972
1050	m	$2,275
1200	m	$2,541
1350	m	$2,844
1500	m	$3,146
Point Repair	each	$ 500

This Capital Plan was developed assuming that Option 3 (Full Diversion) is the preferred option. The construction of the diversion trunk was divided into 3 years. Low priority upgrades or the ones that are only required under future conditions were put together into year 2024-2029. Some of the future upgrades may not be needed depending on actual development conditions. The model should be updated in future years to better prioritize future upgrades. For condition upgrades, the City can refer to the condition assessment reports for specific repair recommendations for each pipe. A summary of the total estimated expenditure on capital improvements per year is provided in Table 5.5.

Table 5.5: Summary of Capital Plan

Year	Approximate Length to be Replaced (m)	Cost Estimate (without Engineering & Contingency)	Cost Estimate (with Engineering & Contingency)
2019	1,935	$2,419,316	$3,266,077
2020	1,479	$3,229,914	$4,360,384
2021	1,732	$1,650,557	$2,228,253
2022	1,053	$1,650,557	$2,228,253
2023	1,021	$1,084,929	$1,464,654
2024-2029	2,964	$2,852,583	$3,850,987
Total	**10,184**	**$12,887,856**	**$17,398,608**

At the time of development of the final report, some upgrades that were planned to be completed in 2017/2018, including the relocation of Habgood Pump Station, were deferred to 2019. These deferred upgrades are identified in Figure 5.8, although the drainage model was developed assuming upgraded conditions.

5.6 Development Contribution Requirements

Under the Local Government Act, developers are required by councils 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 may include a portion of the drainage system that will serve land other than the land being developed. Typically, this can be upgrading of drainage pipes downstream of the development as the downstream system is close to or exceeding capacity. Additional runoff as a result of increased land imperviousness 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 pipe 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.

FIGURE 5.1

Legend

- Option 2: Partial Diversion
- Option 3: Full Diversion

1:8,000

0 0.05 0.1 0.2 0.3 0.4 km

Service Layer Credits: Esri, HERE, Garmin, © OpenStreetMap contributors, and the GIS user community

WHITE ROCK City by the Sea!

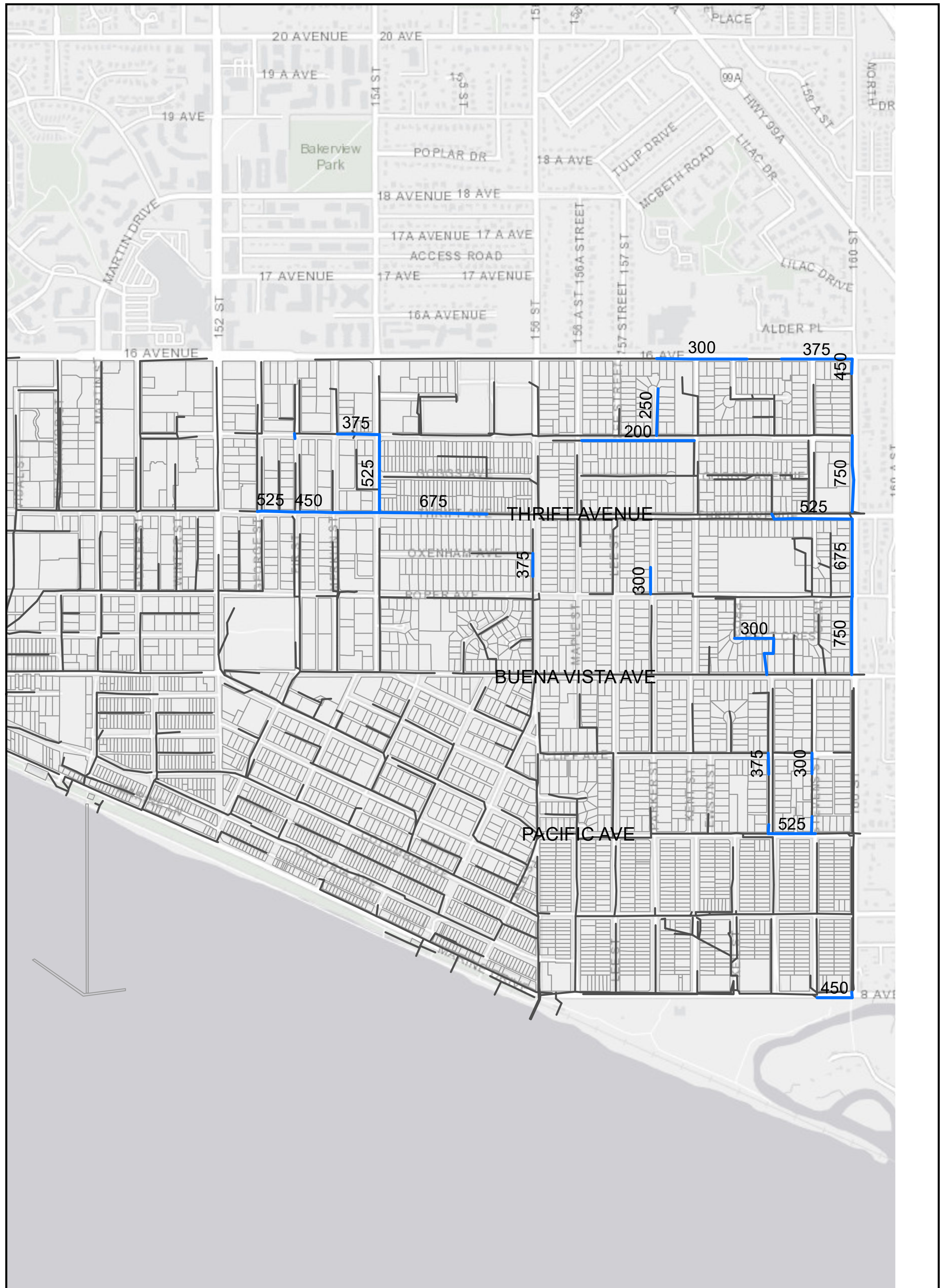
Drainage Master Plan Update 2019

Proposed Alignment for Partial & Full Diversion Options

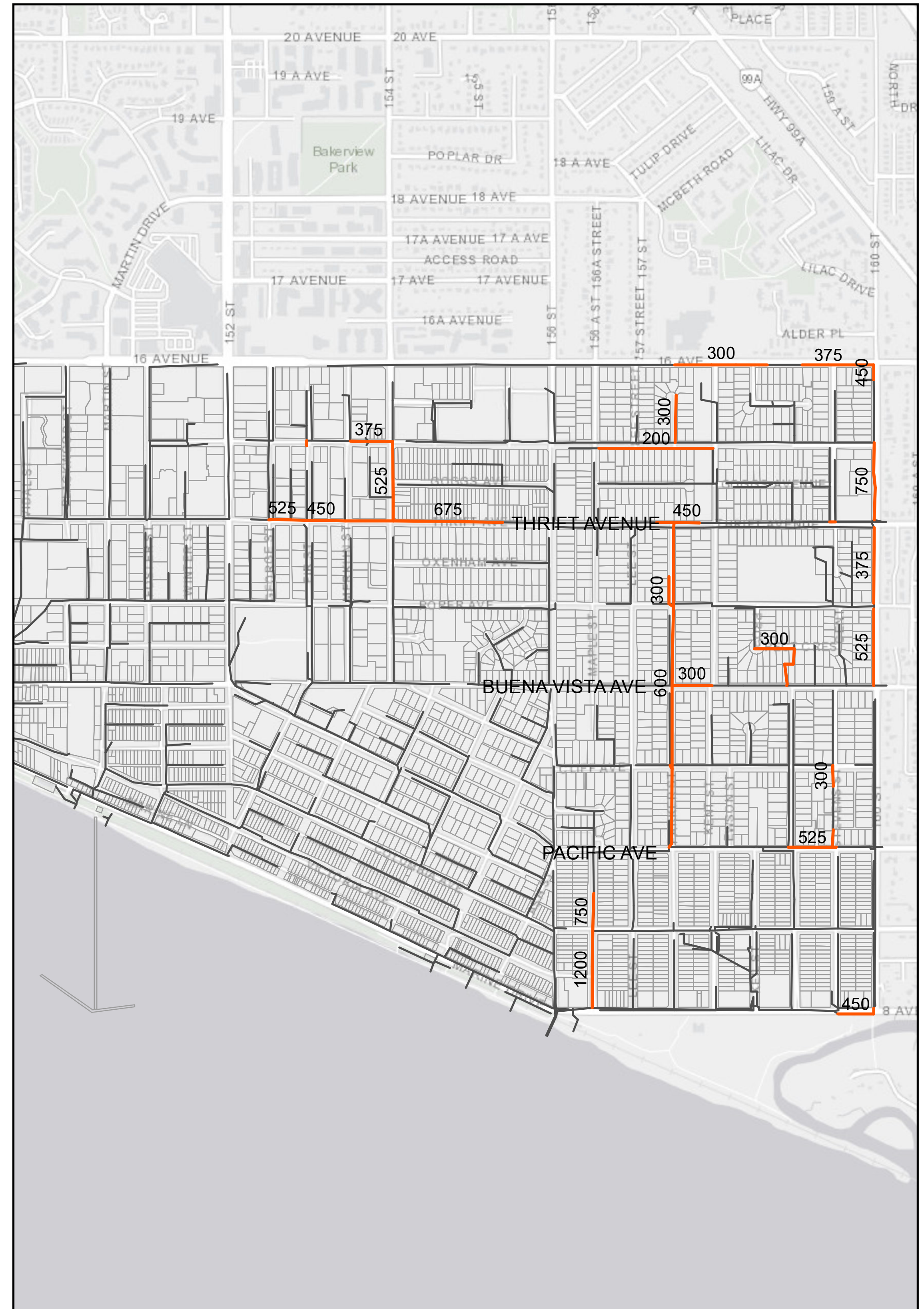
ISL Engineering and Land Services

Date: 2018-12-05

FIGURE 5.2



Option 1: Upgrades with No Diversions



Option 2: Upgrades with Partial Diversion

Service Layer Credits: Esri, HERE, Garmin, © OpenStreetMap contributors, and the GIS user community

Legend

- Option 1: Upgrades with No Diversions
- Option 2: Upgrades with Partial Diversion

1:15,032

0 0.1 0.2 0.4 0.6 0.8 km

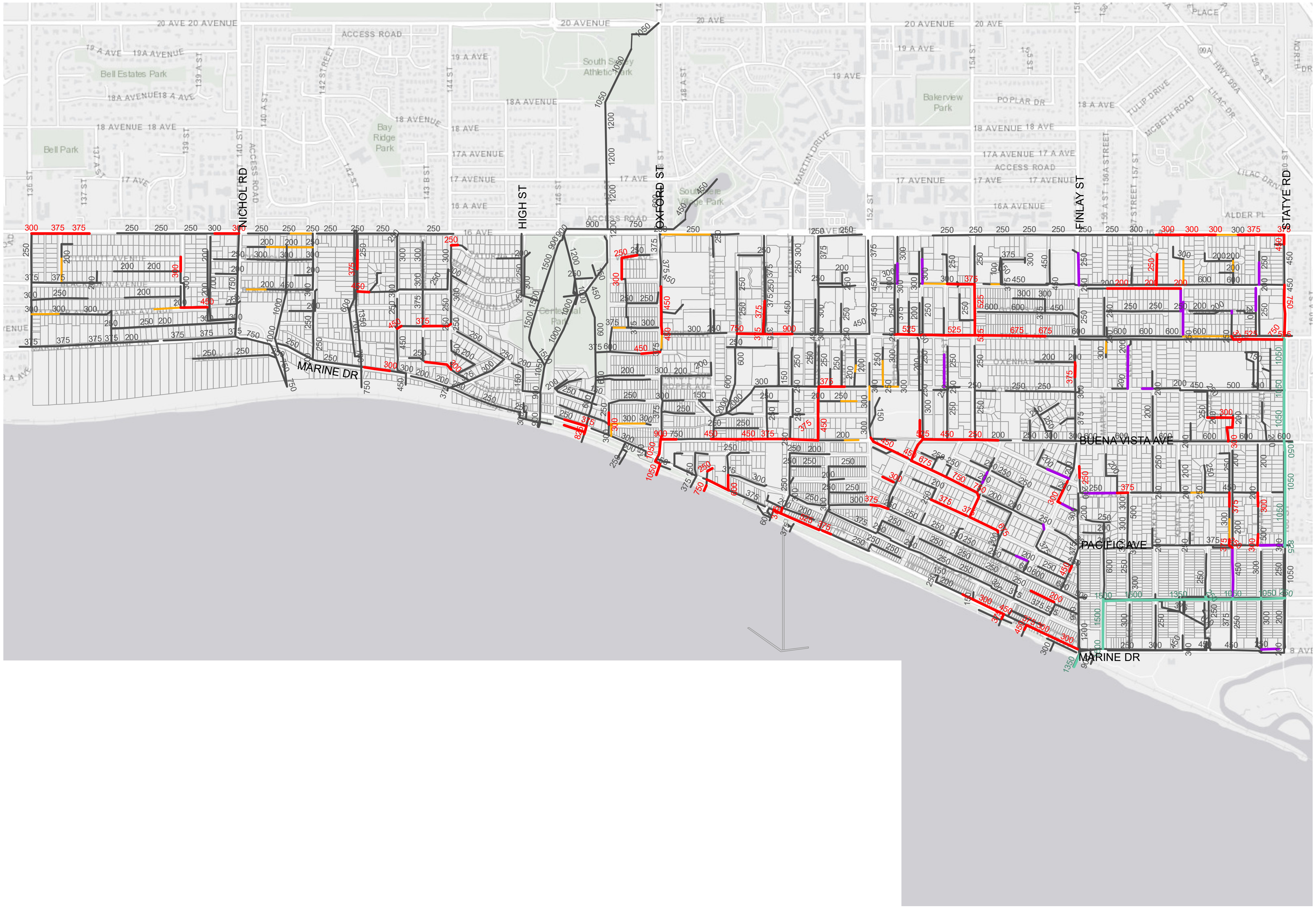
WHITE ROCK City by the Sea!

Drainage Master Plan Update 2019

Proposed Upgrades for Options 1 & 2 (East Side of City Only)

ISL Engineering and Land Services

FIGURE 5.3



Legend

- Capacity Upgrades
- Upgrades Along Diversion Alignment
- Condition Upgrades
- Optional Capacity Upgrades
- Existing Sewers

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

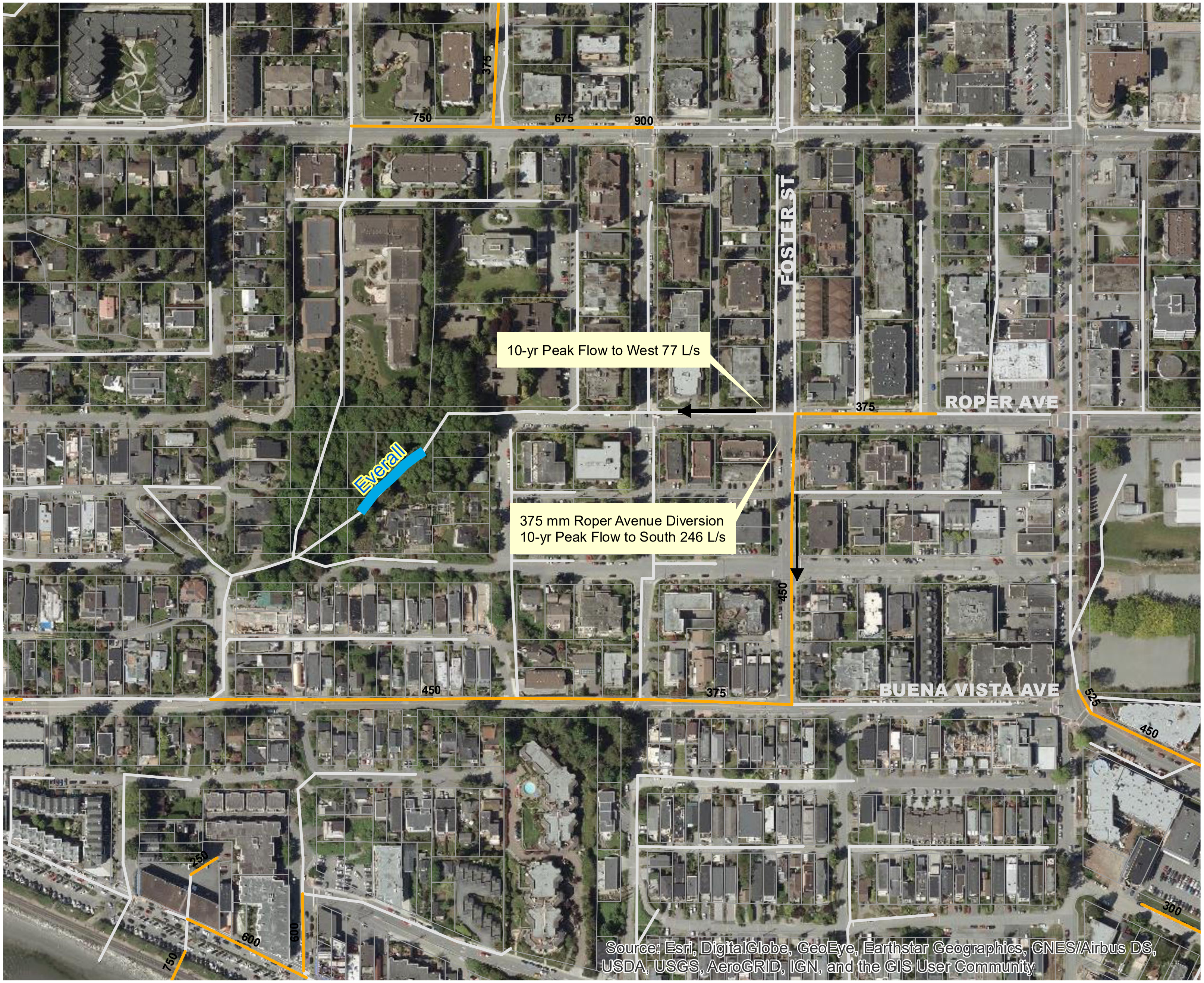
WHITE ROCK
City by the Sea!

Drainage Master Plan Update 2019

Proposed Upgrades (Option 3)

ISL Engineering and Land Services

FIGURE 5.4



Legend

- Capacity Upgrade
- Proposed Diversion
- Streams/Ravines
- Existing Conduits in Model

1:4,000

0 0.015 0.03 0.06 0.09 0.12 km

WHITE ROCK City by the Sea!

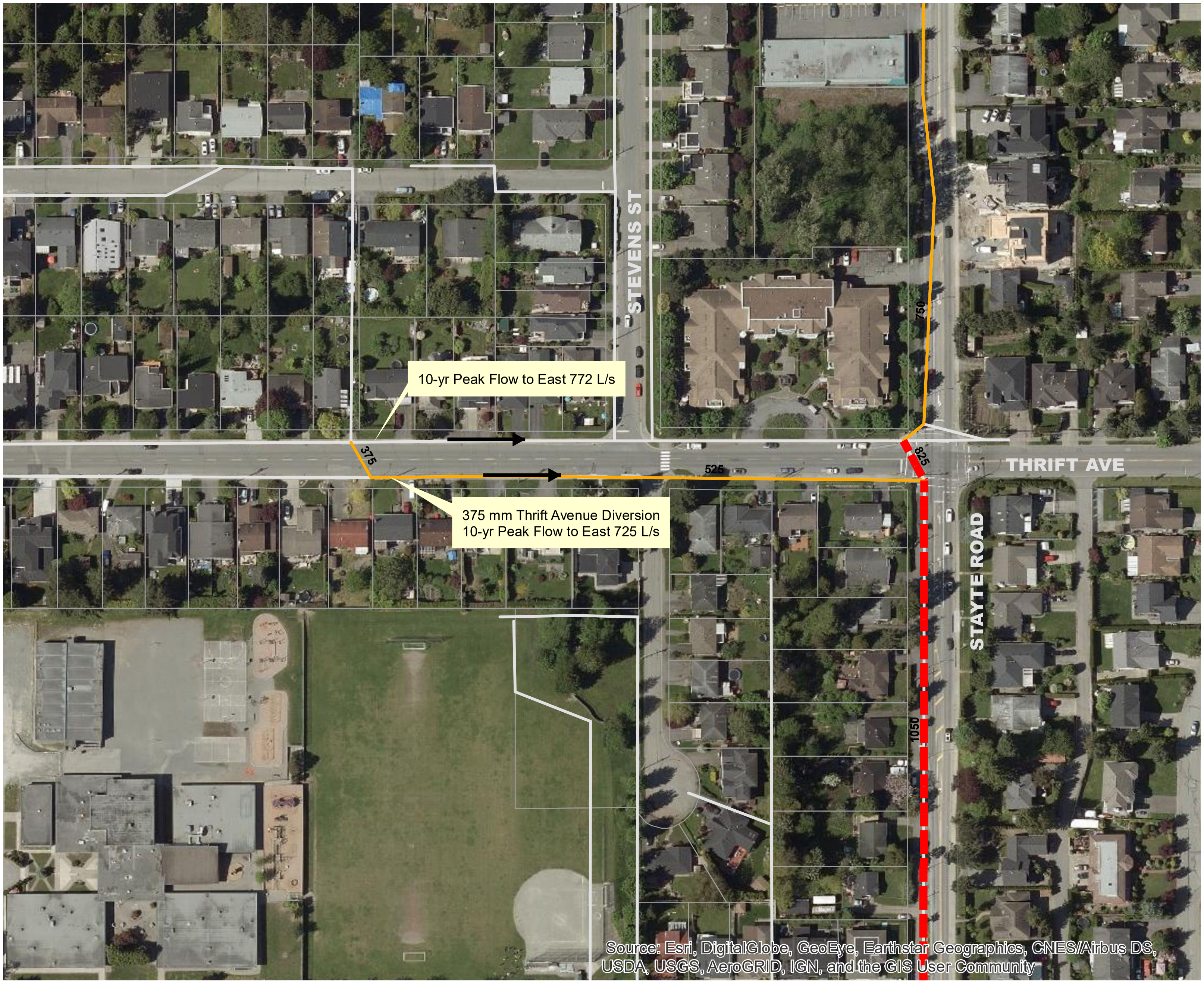
Drainage Master Plan Update 2019

Proposed Roper Avenue Diversion

ISL Engineering and Land Services

Date: 2018-12-05 Document: J:\30000\31900\31939 White Rock Drainage-Sewer MP\02 CADD\25 GIS\251 Figures\Drainage\Roper Diversion (8.5x11).mxd

FIGURE 5.5



WHITE ROCK
City by the Sea!

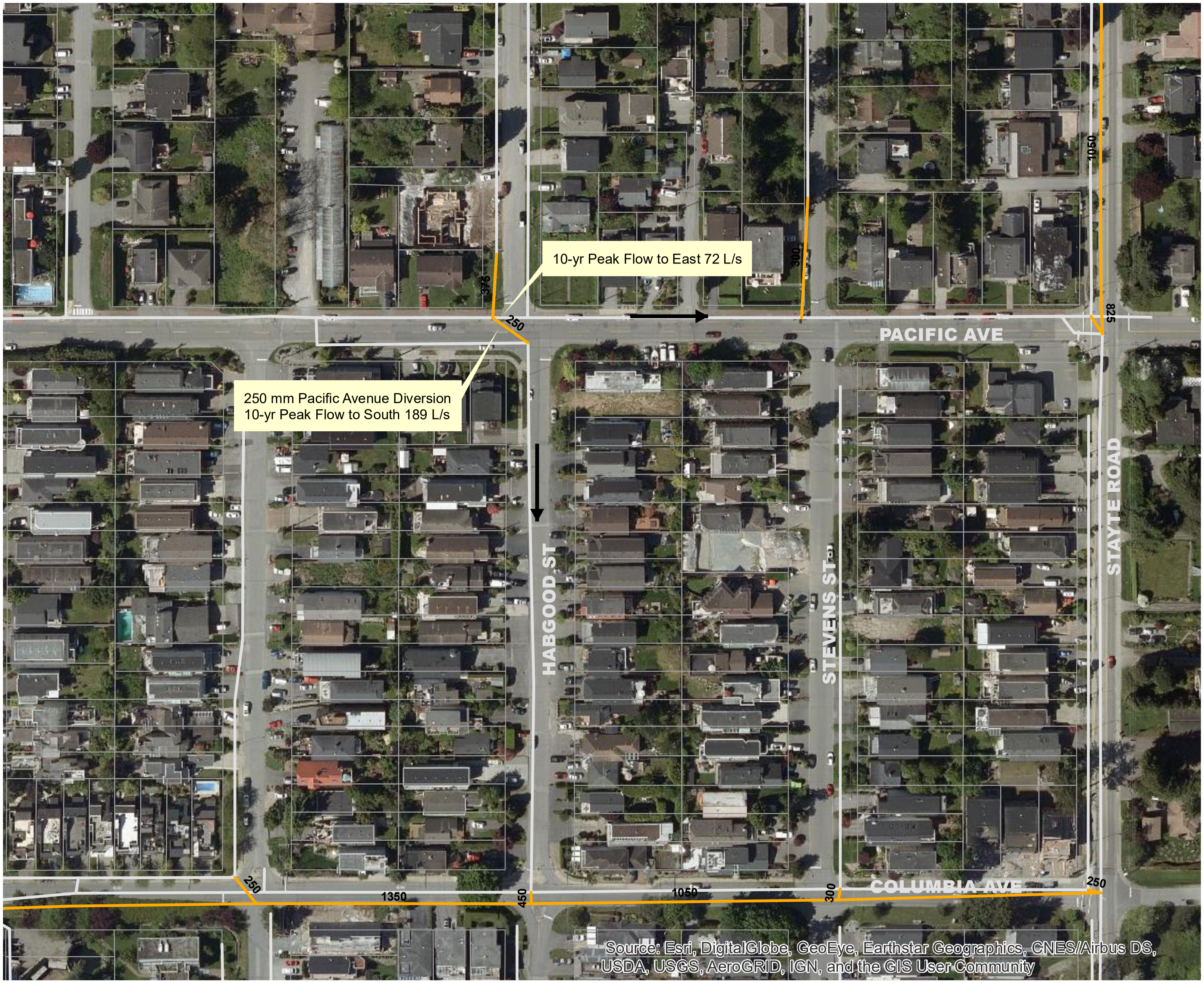
Drainage Master Plan Update 2019

Proposed Thrift Avenue Diversion

ISL Engineering and Land Services

Date: 2018-12-05 Document: J:\30000\31900\31939_White_Rock_Drainage-Sewer_MP\02_CADD\25_GIS\251_Figures\Drainage\Thrift Diversion (8.5x11).mxd

FIGURE 5.6



Legend

- Proposed Diversion
- Capacity Upgrade/Diversion Trunk
- Existing Conduits in Model

1:2,000

0 0.0075 0.015 0.03 0.045 0.06 km

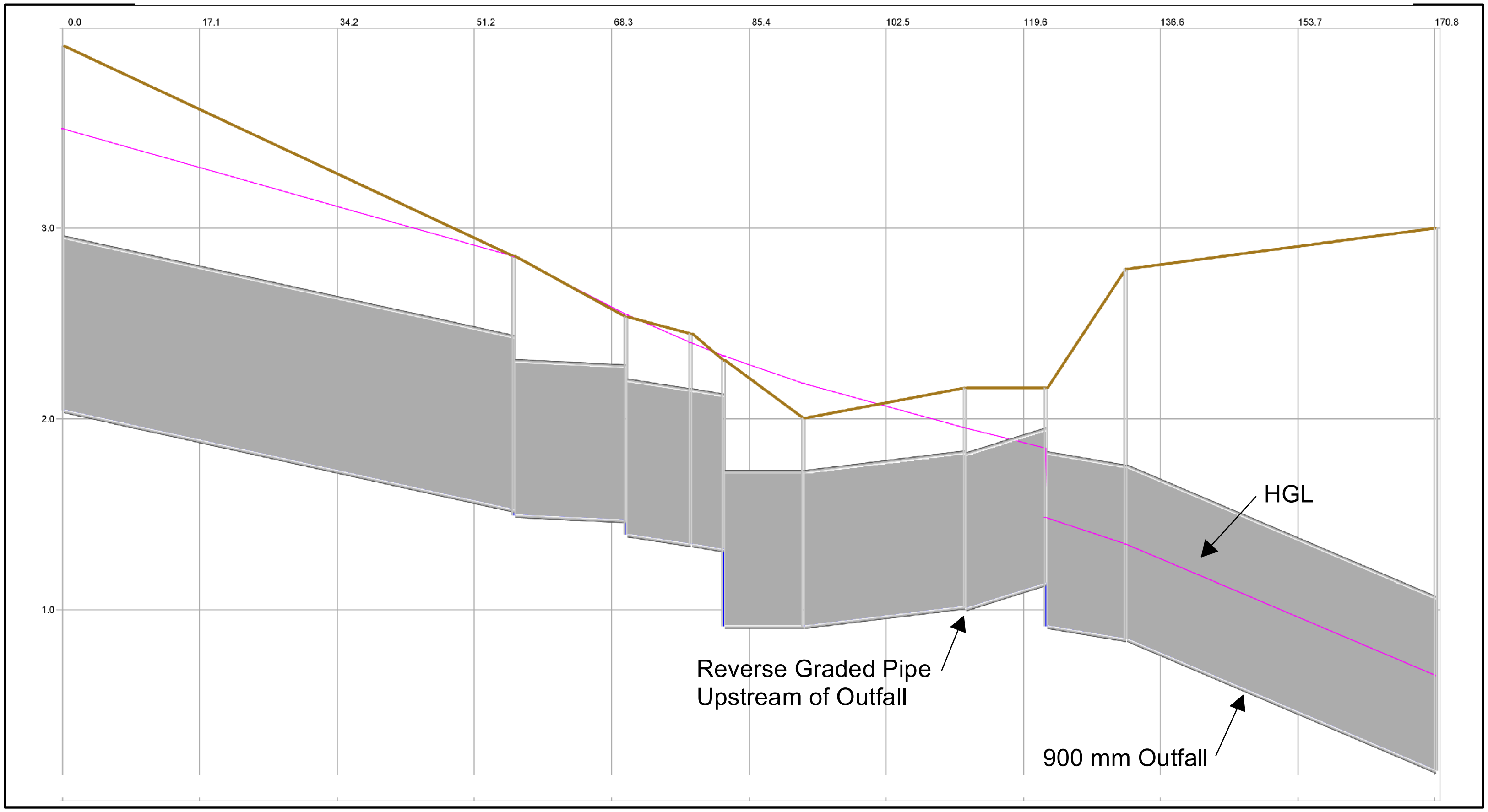
WHITE ROCK City by the Sea!

Drainage Master Plan Update 2019

Proposed Pacific Avenue Diversion

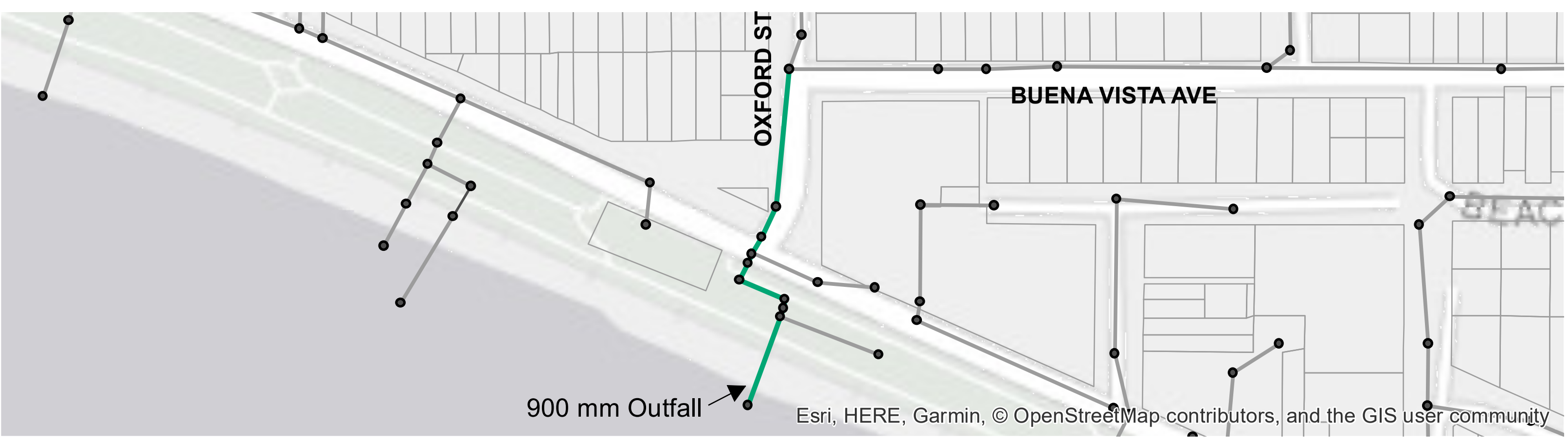
ISL Engineering and Land Services

FIGURE 5.7



Legend

Pipes shown in profile



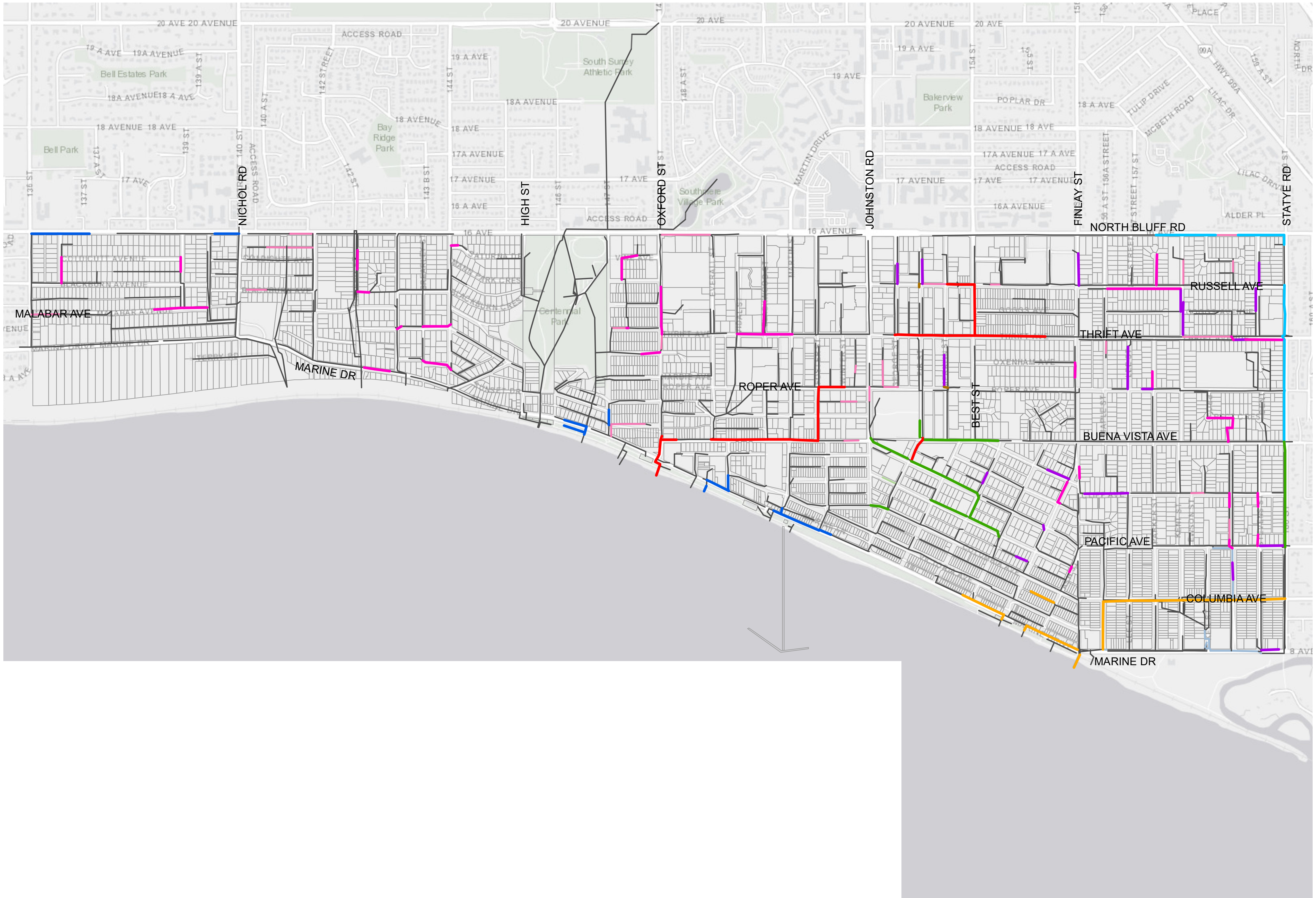
WHITE ROCK
City by the Sea!

Drainage Master Plan Update 2019

Reverse Graded Pipe

ISL Engineering and Land Services

FIGURE 5.8



Legend

- 2018 Condition Upgrade
- 2018 Capacity Upgrade
- 2019
- 2020
- 2021
- 2022
- 2023
- Future (2024-2029)
- Existing Sewers
- Proposed Capacity Upgrade - Completed
- 2017/2018 Upgrades Deferred to 2019
- Optional Capacity Upgrades

1:15,000

0 0.1 0.2 0.4 0.6 0.8 km

WHITE ROCK City by the Sea!

Drainage Master Plan Update 2019

Capital Plan

ISL Engineering and Land Services

Service Layer Credits: Esri, HERE, Garmin, © OpenStreetMap contributors, and the GIS user community

Document: J:\30000\31900\31939_White_Rock_Drainage-Sewer_MP\02_CADD\25_GIS\251_Figures\Drainage\Storm Capital Plan_edit for final.mxd

Date: 2018-12-05

6.0 Conclusion

In this master plan update, the existing drainage system was assessed under both the existing and future OCP conditions to determine upgrades needed to address current capacity issues and support future development needs. In addition to addressing capacity upgrades, the City had indicated that they wanted to divert existing flows that discharge to the City of Surrey and manage all flows within White Rock. A diversion alignment was proposed and was sized to convey the 10-Year peak flows. The cost of full diversion (no flows to Surrey) and associated upgrades were compared with the existing system condition (no diversion) and the partial diversion design proposed in the 2012 study.

Based on the simulation of the 10-Year 1-Hour event, the pipes upstream of the new Habgood Pump Station will surcharge under improved conditions of Option 1 (No Diversion) and Option 2 (Partial Diversion). Under the design event, surcharge levels will be below ground surface. The modeled peak inflow to the pump station under different diversion options was provided in Table 5.3.

The capacity assessment results showed a noticeable increase in surcharged pipes from existing to future condition. This was a result of the predicted increase in impervious areas due to future developments. It was recommended that the City can implement stormwater control measures in future developments to reduce runoff volume and peak flow during smaller storms events. Although these may not decrease the 10-Year peak flows significantly.

Some reverse graded pipes were seen in the model upstream of outfalls on Marine Drive. It may be that the purpose of the reverse graded pipes were to prevent seawater from entering the drainage system which could cause corrosion in the pipes. An alternative to reverse graded pipes for future design is to install flap gates at the outfalls. Removing the reverse grades could help reduce surcharging risks in pipes upstream, especially in low tide condition.

Overall, the proposed capacity upgrades typically increase more than one pipe size from the existing. In some cases, only one pipe size upgrades were required based on the assessment criteria. Upgrading one size may not be the most cost effective solution and there is potential to minimize cost during detailed design.

Appendix A

2012 Drainage Master Plan Update (AECOM)

City of White Rock

Drainage Master Plan Update

Final Report

Prepared by:

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

Project Number:

60239267

Date:

December 06, 2012

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

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AECOM: 2012-01-06

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

December 06, 2012

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: 60239267

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

Please find attached three copies (3) of our Final Report for the 2012 Drainage Master Plan Update including an 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. This report includes documentation and review of the following key aspects:

1. Project introduction and objectives;
2. Modelling approach and development;
3. Existing drainage system assessment;
4. Improvements and recommendations; and
5. 10-Year capital plan.

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 there are any questions or concerns please don't hesitate to contact me 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 19, 2012	Draft Report
1	Stephen Bridger	July 04, 2012	Final Report
2	Suman Shergill	December 06, 2012	Revised Final Report

AECOM Signatures

Report Prepared By:

Sumandeep Shergill, P. Eng.
Project Engineer

Report Reviewed By:

Stephen Bridger, P. Eng.
Project Manager

Executive Summary

The City of White Rock retained AECOM to complete a Drainage Master Plan (DMP) 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 study, the City had completed a drainage study assessment and a Capital Plan in 1999 that was later revised in 2004. Since then, development has continued in the City and the construction of new drainage infrastructure has not been reflected in the 2004 drainage model. The key differences between this DMP and the previous study are listed in **Section 6.1** of the report. In summary, the main objectives of the 2012 Drainage Master Plan were as follows:

- Conduct a flow monitoring program to obtain current flow data;
- Update the 2004 drainage model and validate the model to reflect existing conditions;
- Develop a strategy that maintains existing base flows draining into the City of Surrey while managing high flows and any additional flows within the City of White Rock;
- Assess the capacity of the existing drainage system and review options to divert flows away from Little Campbell River and the previously proposed Stayte Road diversion;
- Assess the condition of the existing drainage infrastructure and provide a plan for a continued CCTV condition assessment program;
- Develop a 10- Year Capital Plan for the City using a phased approach for short, medium, and long term projects that are considerate of the City's annual budget for drainage works; and
- Conduct slope stability review of the 5 major ravines in the City of White Rock.

There are approximately 99.3 km of storm sewers in the City and the majority drain directly to Semiahmoo Bay or Little Campbell River via the 25 outfalls. There are two drainage pump stations (Habgood and Oxford) that drain low areas along Marine Dr. In addition there are four locations where storm sewers drain from White Rock to Surrey. Three are located along Stayte Road and the fourth is at Bergstrum Road. No increase in flows at these locations is recommended, and the proposed strategy is to maintain base flows to Surrey while managing the high flows within the City of White Rock. There are also three locations where flows from Surrey enter into White Rock. These three are all located along North Bluff Road, and one takes in discharge from Surrey's Southmere Detention Ponds. With exception of Nichol Road, the catchments from Surrey enter into dedicated diversion sewers in White Rock, that discharge to Semiahmoo Bay.

The previous drainage model was developed using XP-SWMM software and, as such, we elected to keep the model in XP-SWMM format and update it accordingly. A review of the GIS drainage system data was completed and GIS data gaps were resolved. A total number of 2731 links and 3401 nodes were identified in the updated model, of which 48 new links and 93 new nodes were added to represent new or missing drainage infrastructure. The model was then further updated with information provided by the City and any recent as-built drawings for new sewers. To validate the updated model, SFE Global was retained to install four flow monitoring stations at strategic locations in the City from January to March of 2012. Corresponding rainfall data was obtained from the White Rock STP rain gauge.

The capacity assessment of existing storm system was completed for the 10-year 1hr design storm event under future OCP land use conditions. Results show that a total of 278 out of 1,466 modelled existing storm pipes were found to surcharge under the 10-year storm event. This means that 19% of the modelled pipes within the system are unable to convey the 10-year storm event without surcharging. Model results also show that both pump stations are undersized to convey 10-year peak flows. **Figure 4.1** shows the system capacity assessment under the 10-year 1hr design storm event for OCP conditions.

A list of all the proposed upgrades was prepared based on the capacity assessment criteria mentioned discussed in report. Due to steep terrain, limited undeveloped land and high land cost, neither detention ponds or underground storage tanks were considered feasible. So diversions and sewer upgrades were considered. The proposed upgrades and diversions are discussed in **Section 6.2** of the report.

A review of the outfalls and major culverts under the 100-Year design storm event was completed. Of the 25 outfalls and major culverts that are shown in the GIS, only 17 were included in the hydraulic model. The remainder are downstream of open channel sections, within park areas or under the BNSF railway and limited information is available for them. A recommendation is made to complete a condition and capacity assessment of all outfalls to Semiahmoo Bay.

A review of the available pipe age information and CCTV data is also provided. Upon review of the City's GIS data it became apparent that 52% or approximately 55.4Km of the storm sewer collection system did not have an entry for pipe age. The total length of CCTV data that was provided was only approximately 5% of the entire system, such that a detailed city-wide condition assessment could not be completed.

An additional component of the DMP was an assessment of the five the major ravines that was completed by Thurber Engineering and AECOM. Thurber's full report is included in **Appendix A** of this report. The major findings were on Coldicott Creek where there was evidence of a recent landslide and it was recommended by Thurber that a detailed assessment be completed for this area as well as confirmation that private property owners on the south side of Marine Drive in the vicinity of the ravine are not discharging rainwater runoff to the slopes.

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

Year	Approximate Length to be Replaced (m)	Total Cost Estimate (incl. Engineering and Contingency)
2013	1,265	$ 1,493,170
2014	836	$ 1,377,348
2015	808	$ 2,124,608
2016	766	$ 2,125,154
2017	1,064	$ 1,399,903
2018 - 2023	6,640	$ 1,625,999
Total	**11,379**	**$ 20,146,181**

Following are some of the additional recommendation for the City with respect to GIS data completion, improving the efficiency of drainage system and capital works planning;

- City should update the GIS data to include all pipe attributes (i.e. pipe diameter, inverts, material and age) possible. This can be achieved by further review of available record drawings, existing and future CCTV assessments and field measurements.

- City should catalogue all existing CCTV data and complete a full condition assessment then proceed with conducting assessment for approximately 10% of the system per year.
- According to discussions with City staff, some of the outfalls south of Marine Drive are buried under the boulders along the BNSF railway tracks. Discussions should be carried with BNSF staff to re-align the boulders so that they don’t act as obstruction to peak flows during major storms.
- Field review shows that 600mm diameter outfall south of Oxford Pump Station is clogged by debris and tideflex valve is separated from the outfall pipe. This outfall and all other outfalls should be inspected and cleared of any debris to prevent any flooding in the upstream area during a major storm.
- The City should conduct a condition assessment of all the outfalls to Semiahmoo Bay. For any outfall not in the current model, additional information such as inverts, diameters, upstream creek cross-section (where applicable) should be collected for capacity assessment.
- Both the drainage pump stations are old and detailed pump station condition assessment should be conducted prior to upgrading the pump stations.
- Prior to upsizing the storm sewers on Nicole Road (between North Bluff Road and Marine Drive), cost sharing discussions should be carried with the City of Surrey as it drains approximately 64ha catchment within City of Surrey boundary.

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Appendices

1 Introduction

1.1 Background Information

The City of White Rock retained AECOM to complete a Drainage Master Plan (DMP) Update including an update to their 10-Year Capital Plan for planning and budgeting purposes. Prior to this, the City had completed a drainage study assessment and a Capital Plan in 1999 that was later revised in 2004. Since then, development has continued in the City and the construction of new drainage infrastructure has not been reflected in the 2004 drainage model.

Recently in 2010, the City finalized an Integrated Stormwater Management Plan (ISMP) as part of their commitment to Metro Vancouver's Liquid Waste Management Plan (LWMP). This ISMP included a drainage system assessment that was based on the previous 2004 study. As a result, the City still required an update to their model and DMP which would validate recommendations provided in the previous studies, include newly constructed infrastructure, and incorporate existing and future land use changes for development of a new Capital Plan.

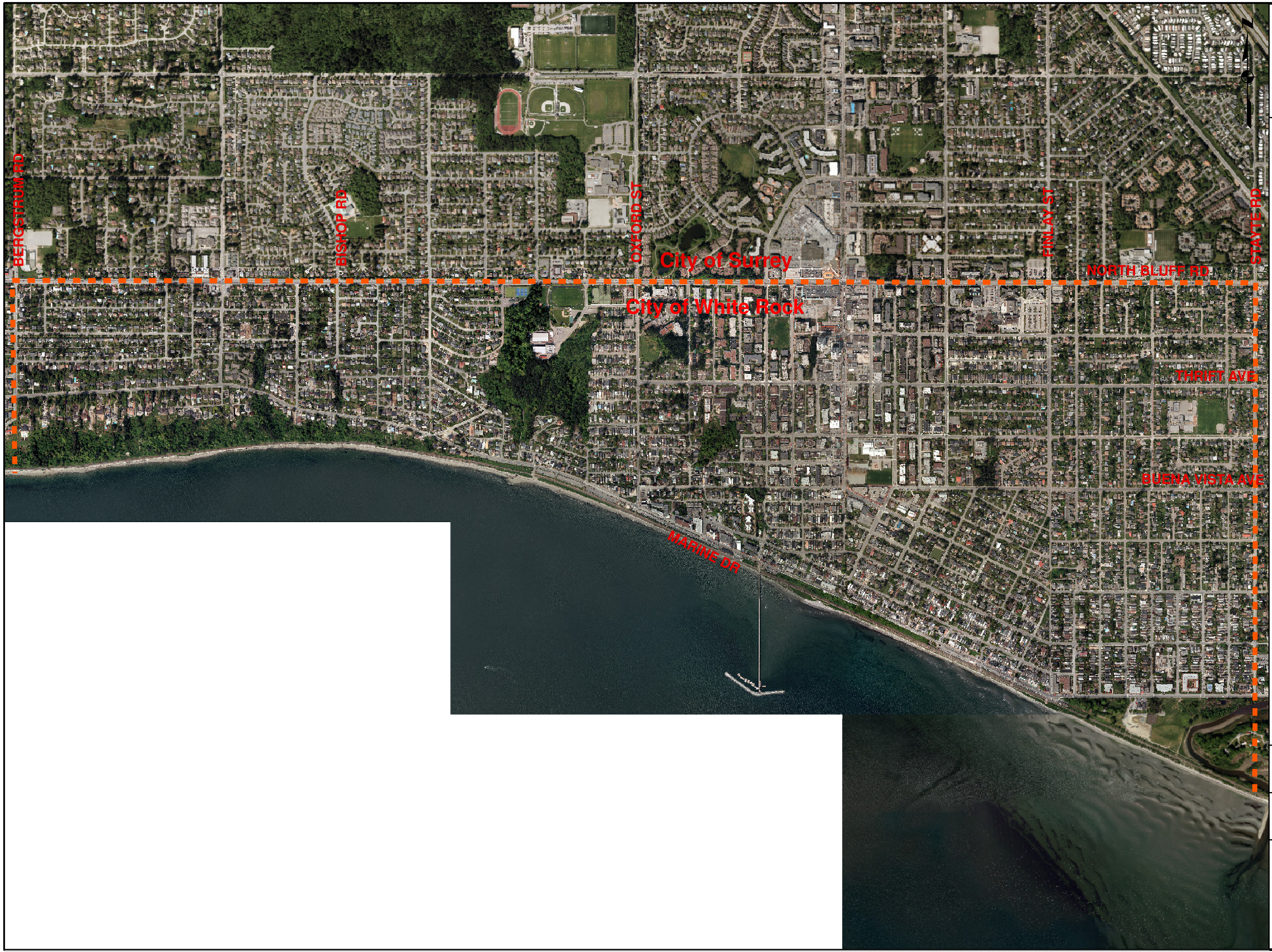
The new 2012 DMP uses the city-wide hydro-dynamic XPSWMM model previously developed by Urban System Ltd (USL), with the model updated to reflect current land use conditions and full build out of the Official Community Plan (OCP). As part of this project, the following tasks were completed:

1) A review of existing information has been summarized including specific updates to the model and how it was validated;
2) A new flow monitoring program to obtain current flow data was conducted;
3) An assessment of ravine slope stability was conducted;
4) A capacity assessment and review of existing condition assessment data for existing drainage infrastructure was completed; and,
5) Recommendations for improvements to support OCP land-use conditions are included in the updated 10-Year Capital Plan.

1.2 Study Area

The City of White Rock is home to approximately 19,240 people and covers an area of approximately 473 hectares. The City 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**. There are approximately 99.3 km of storm sewers in the City. Most of the storm sewers drain directly to Semiahmoo Bay or Little Campbell River via the 25 outfalls or two drainage pump stations. All the remaining storm sewers drain into the City of Surrey's storm sewer network. Drainage infrastructure includes pipes ranging in diameter from 100mm to 1350mm and is primarily concrete pipe. There are several creeks and ravines within the City limits with the major ones being Duprez Ravine, Coldicott Creek, Anderson Creek and Everall Creek.

Development in the form of densification has occurred in certain areas of the City – primarily the Town Centre area. This is placing increased pressure on existing drainage systems, increasing erosion and degradation of watercourses, and increasing operations and maintenance costs. There are also a number of storm sewers and catchment areas in the City of White Rock that drain towards the City of Surrey as well as areas of Surrey that drain to White Rock. The flow conditions at these locations are important to review as there can be downstream implications from increased development activity.



The City of White Rock

Master Drainage Plan Update

Legend

City Boundary

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Study Area

Figure 1.1

1.3 Previous Studies

Previous drainage studies that provided valuable background information include:

Comprehensive Drainage Assessment Study, Urban Systems Limited (USL) 1999 (Updated in 2004)
City of White Rock Integrated Stormwater Management Plan (ISMP), USL 2010

The *Comprehensive Drainage Assessment Study* (USL 2004) includes a City-wide capacity assessment for the 10-Year and 100-Year design storm events and a prioritised list of capital upgrades with particular emphasis on high flow trunk sewer diversions for conveyance of the 100-Year peak flows. In contrast, the current DMP and Capital Plan focus more on conveyance of 1 in 10-Year design storm event peak flows in the piped system with exception of roadway culverts, railway crossings and outfalls which were sized for 1 in 100-Year design storm event.

Drainage infrastructure issues noted in the ISMP (USL 2010) include the potential for increased ocean water levels and storm surges resulting in increased risk of flooding along Marine Drive. This is also denoted by floodplain zones in the OCP and worsened by concerns that the drainage pump stations are under capacity. Other items noted in the ISMP include risk of basement flooding due to 100-Year water levels or hydraulic grade line (HGL); rights-of-way for drainage infrastructure upgrades with specific mention of outfalls under the Burlington Northern Santa Fe (BNSF) Railway; the lack of a blanket works agreement for infrastructure renewal; the need for increased condition assessment for storm sewers; and implementation of measures to improve stormwater runoff quality.

1.4 Key Issues & Objectives

This DMP 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 DMP outlines a phased Capital Plan for storm sewer infrastructure improvements and replacement that is within the current budget expenditures. In addition to the development of a comprehensive 10-Year Capital Plan, there are a number of key drainage issues that the City currently faces that include management of future development plans, trans-boundary issues with the City of Surrey, review of previously proposed trunk sewer diversions (including the proposed new Stayte Road Outfall), ravine slope stability, and addressing concerns noted in the ISMP.

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 White Rock drainage system and ravines collect runoff from the City of Surrey at three locations and also discharges to Surrey at four locations which results in "trans-boundary" conditions that must be agreed upon between municipalities. This DMP reviews each occurrence for trans-boundary flows and provides guidance on how to manage these flows for the future.

A review of the trunk sewer diversions previously proposed for management of the major system was undertaken in the system analysis. The peak flows at several locations including the Oxford/Everall Trunk System, Fir/Columbia/Martin Trunk System, and Stayte/Pacific/Kent trunk System were assessed with the goal of maintaining base flows while providing a viable alternative for high flows.

In trans-boundary areas where drainage from White Rock discharges into the City of Surrey, it was agreed that base flows into Surrey would be maintained in order to preserve flows in watercourses such as McNally Creek. Flows up to existing conditions would continue to enter into Surrey; however any additional flows above this condition would need to be managed within White Rock's boundaries (either through detention facilities or diversions). This approach was developed through discussions between both municipalities and it mimics the approach of managing trans-boundary drainage flows from Surrey into White Rock across North Bluff Road.

Another key issue was to give particular attention to the Stayte Road storm sewer system where previously recommended infrastructure works have already commenced, however there are challenges with gaining approvals for the proposed new outfall to Little Campbell River which is located within land owned by the Semiahmoo First Nation. While continuing discussions with the First Nation are encouraged we have focused our efforts on determining alternative solutions for the City to manage stormwater through outfalls located within the City of White Rock.

In summary, the main objectives of the 2012 Drainage Master Plan are as follows:

1) Update the 2004 model developed by USL and validate the model to reflect existing conditions;
2) Develop a strategy that maintains low to existing flows draining into the City of Surrey while managing high flows and any additional flows within the City of White Rock;
3) Assess the capacity of the existing drainage system and review options to divert flows away from Little Campbell River and the previously proposed Stayte Road diversion;
4) Assess the condition of the existing drainage infrastructure and provide a plan for a continued CCTV condition assessment program;
5) Develop a Capital Plan for the City using a phased approach for short, medium, and long term projects that are considerate of the City's annual budget for drainage works; and
6) Conduct slope stability review of 5 ravines in the City of White Rock including Coldicott Ravine, Collingwood Ravine, Duprez Ravine, Anderson Ravine and Everall Ravine.

2 Existing Drainage System and Criteria

This section provides a summary of the existing drainage system characteristics and design criteria for storm sewer servicing used in the analysis and preparation of the DMP update. A discussion of the drainage catchment areas, watercourses and ravines, outfalls and drainage infrastructure, as well as existing and future land use (OCP conditions) is provided.

2.1 Drainage Infrastructure

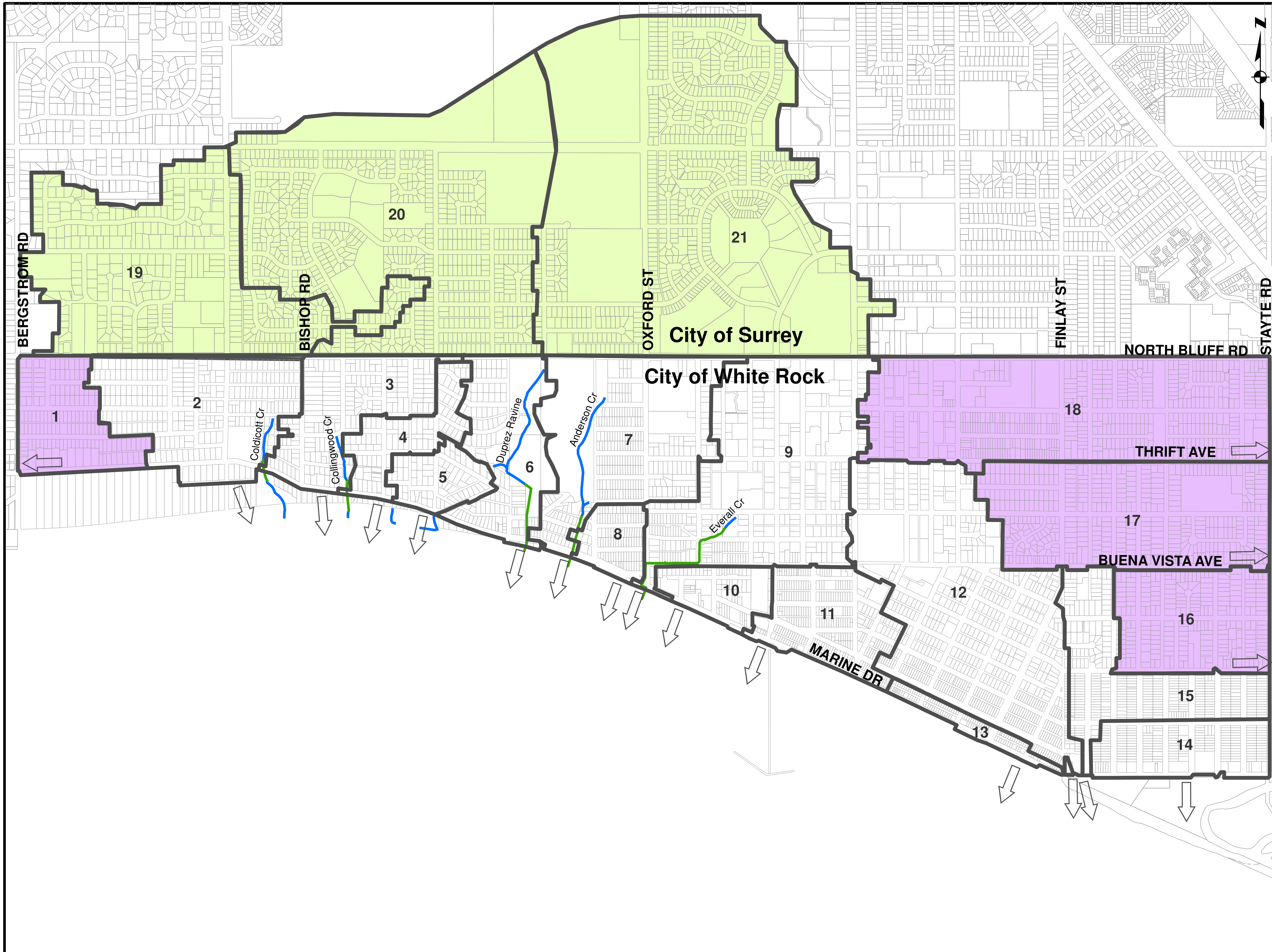
There are 18 major drainage catchment areas that make up the 473 Ha within the City limits and an additional 305 Ha of land in the City of Surrey that drains into White Rock. Of the 473 Ha within the City of White Rock there are 148 Ha that drain toward to the City of Surrey through storm sewer connections in the west at Bergstrom Road and in the east along Stayte Road. These major catchment areas are shown in **Figure 2.1** along with the locations of the major creeks and outfalls. The major catchment areas are further defined into 876 sub-catchment areas for modelling purposes and a summary of the assumed imperviousness and hydrologic parameters used for the model is provided in **Section 3.2** below.

The majority of the storm sewer infrastructure in the City drains directly to Semiahmoo Bay either through piped outfalls along the waterfront or culvert crossings under the BNSF Railway. Outfalls range in diameter from 300mm to 1200mm. There are two pump stations in the City: Oxford PS is located west of the foot of Oxford Street and discharges directly to Semiahmoo Bay; and Habgood PS is located at the foot of Habgood Street and discharges to Little Campbell River. No as-built information was available for both the pump stations. Pump curves were obtained from Xylem (formerly Flygt ITT). The catchment areas for Oxford PS and Habgood Pump are 6.5 Ha and 14 Ha respectively.

Including the major trunk sewers and flow diversions there are approximately 99.3 km of storm sewers in the City. An overview of the modelled trunk sewer system and locations of outfalls is shown in **Figure 2.2.** A breakdown of the GIS pipe data showing the diameter size range and material is listed in **Tables 2.1 and 2.2** below. As shown in the tables, a number of GIS data gaps were found (discussed in **Section 3.1**) in the storm sewers diameters and material fields (as well as age which is discussed in the condition assessment section). In such case the hydraulic model does not include all pipes within the City. **Figure 2.2** shows pipes that are included in the model.

Table 2.1 Storm Sewer Diameters

Diameter	Length (m)
100mm to 250mm	49,837
300mm to 450mm	28,542
600mm to 900mm	9,420
Greater than 1050mm	1,714
Unknown	9,877
Total Length =	**99,390**



The City of White Rock

Master Drainage Plan Update

Legend

- Storm Catchments
- Enclosed Watercourse
- Open Channel
- Direction of Catchment Outflow
- Storm Catchment draining to Surrey
- Storm Catchment draining from Surrey

Catchment #	Hectares
1	16.5
2	31.9
3	21.0
4	11.1
5	7.9
6	21.0
7	39.4
8	6.7
9	52.8
10	8.4
11	18.1
12	63.3
13	4.6
14	14.8
15	23.8
16	23.7
17	44.8
18	62.9
City of Surrey Contributing Catchments	
19	64.2
20	101.8
21	138.2

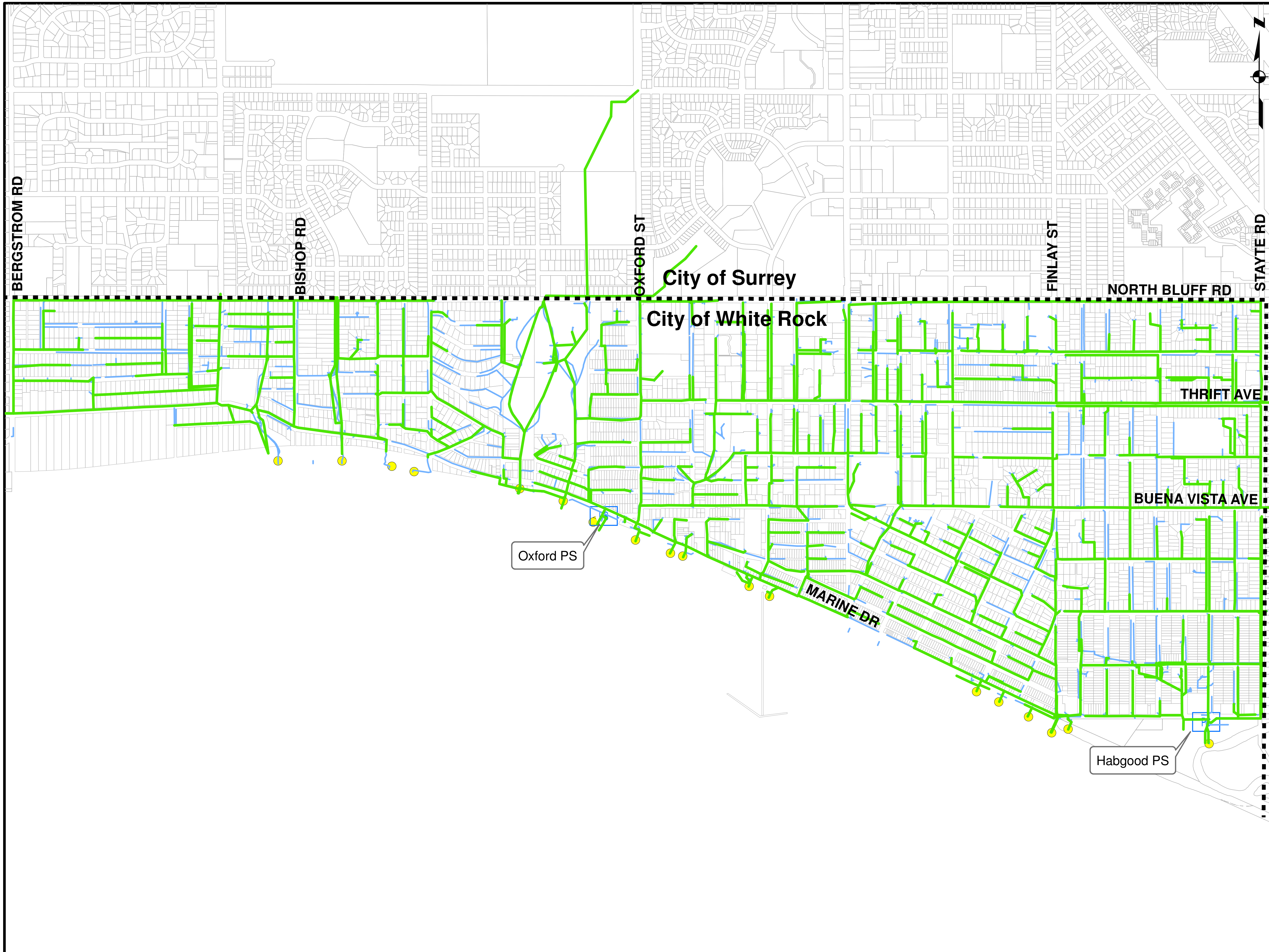
AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Major Catchment Areas & Watercourses

Figure 2.1



The City of White Rock

Master Drainage Plan Update

Legend

- Modelled Storm Sewers
- Pipes Not in Model
- PS Pump Station
- Ocean Outfalls

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Modelled Storm Sewer System

Figure 2.2

Table 2.2 Storm Sewer Materials

Material	Length
Concrete	51,153
PVC	34,934
Steel	704
Asbestos Concrete	1,995
Concrete Lined	485
HDPE	577
Unknown	9,540
Total Length =	**99,390**

2.2 Watercourses and Slope Stability Review

The major watercourses and ravines are the Duprez Ravine, Coldicott Creek, Anderson Creek, Everall Creek and Collingwood Creek. Significant features for each creek/ravine is discussed below with additional detailed information provided in **Appendix A – Ravine Slope Stability Review** which is a report completed by Thurber Engineering Ltd as part of the DMP.

Duprez Ravine is located in the center of White Rock and originates at North Bluff Road continuing south to Duprez Street just north of Marine Drive. There are known historical stability issues along Duprez Ravine that resulted in the June 1999 flood event at Marine Drive and remedial works have since been completed including significant bank stabilization and construction of a 1050mm diameter HDPE storm sewer bypass for flows from the City of Surrey. Results of the recent reconnaissance indicate that the creek channel slopes are relatively stable with little evidence of sloughing or current instability.

Coldicott Creek includes an upper reach north of Marine Drive and lower reach south of Marine Drive that discharges to Semiahmoo Bay. To minimize the potential for creek channel erosion drainage from Marine Drive is conveyed down the lower reach in a 750mm diameter HDPE pipe. During the field reconnaissance it was noted that the ravine is deeply incised with steep slopes downstream of Marine Drive and there was evidence of a landslide that extends from the crest of the slope to the channel. A detailed assessment of the slide area was recommended by Thurber as well as confirmation that private property owners on the south side of Marine Drive in the vicinity of the ravine are not discharging rainwater runoff to the slopes.

Collingwood Ravine extends from Malabar Avenue north of Marine Drive to the ocean discharge. Adjacent to the open channel section of the creek there is a 900mm storm sewer that originates in the City of Surrey and follows the creek ROW. A review of the ravine south of Marine Drive was not completed due to access limitations (e.g. steep slopes and vegetation).

Anderson Ravine extends from Vine Avenue at the north end and terminates near Upper Roper Avenue where it enters an inlet structure into drainage system. The ravine slopes are high and steep in some locations but there was no evidence of major instabilities. There is also a 600mm diameter storm sewer adjacent to the creek that discharges to the Bay via an outfall at Marine Drive.

Everall Ravine extends from just north of Roper Avenue west of Blackwood Street to an inlet structure at Prospect Avenue. The majority of the open channel portion of the creek is within private property and the side slopes appear to be well constructed with rip rap and gabion walls. At Prospect Avenue construction of an inlet headwall structure and overflow weir have been completed that have alleviated drainage concerns in this area.

2.3 Disconnected Roof Leaders

The practice of disconnecting building roof leaders plays a significant role in reducing the amount of stormwater runoff volume and peak flows that enter into the drainage system. Disconnection involves cutting off the downspout and installing an elbow to let stormwater run onto lawns, gardens, rock pits or into a rain barrel. This has many benefits such as reducing runoff volume, minimizing pollutants that may enter the sewer, and recharges groundwater. Areas within the City where a significant number of houses have disconnected roof leaders and this practice is permissible are shown in **Figure 2.3**.

It should be noted that site specific soil conditions and immediate surface grades should be taken into consideration when allowing disconnection of roof leaders such that there are no adverse impacts to neighbouring property. Further discussion on the specific catchment area imperviousness values applied for areas with disconnected roof leaders is presented in **Section 3.2.**

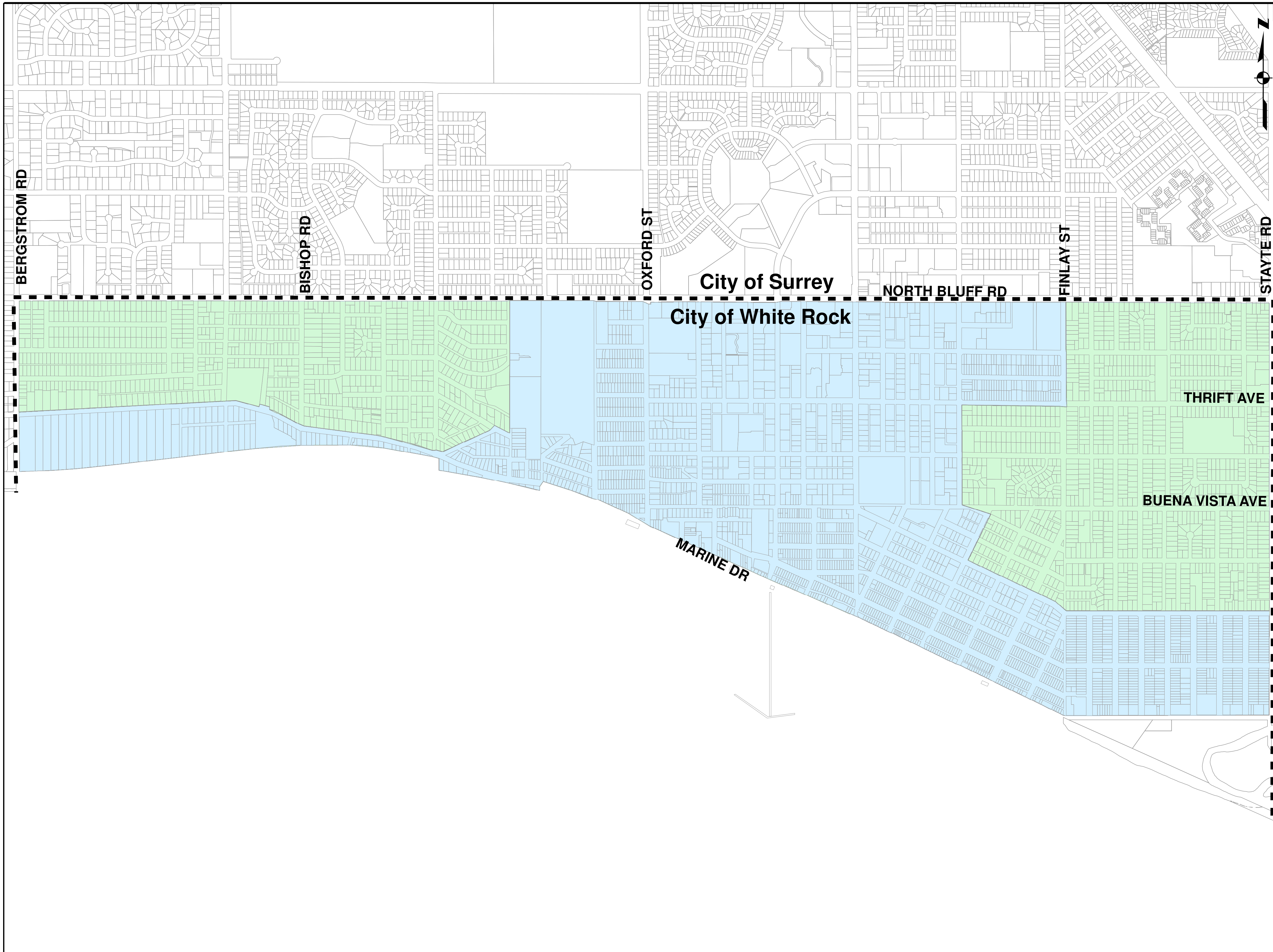
2.4 Existing and Future Land Use

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. 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. OCP land use is shown in **Figure 2.4**.



THE CITY OF WHITE ROCK

Master Drainage Plan Update

Legend

- Lots
- Zone of Disconnected Roof Leaders
- Zone of Permissible Roof Connections
- City Boundary

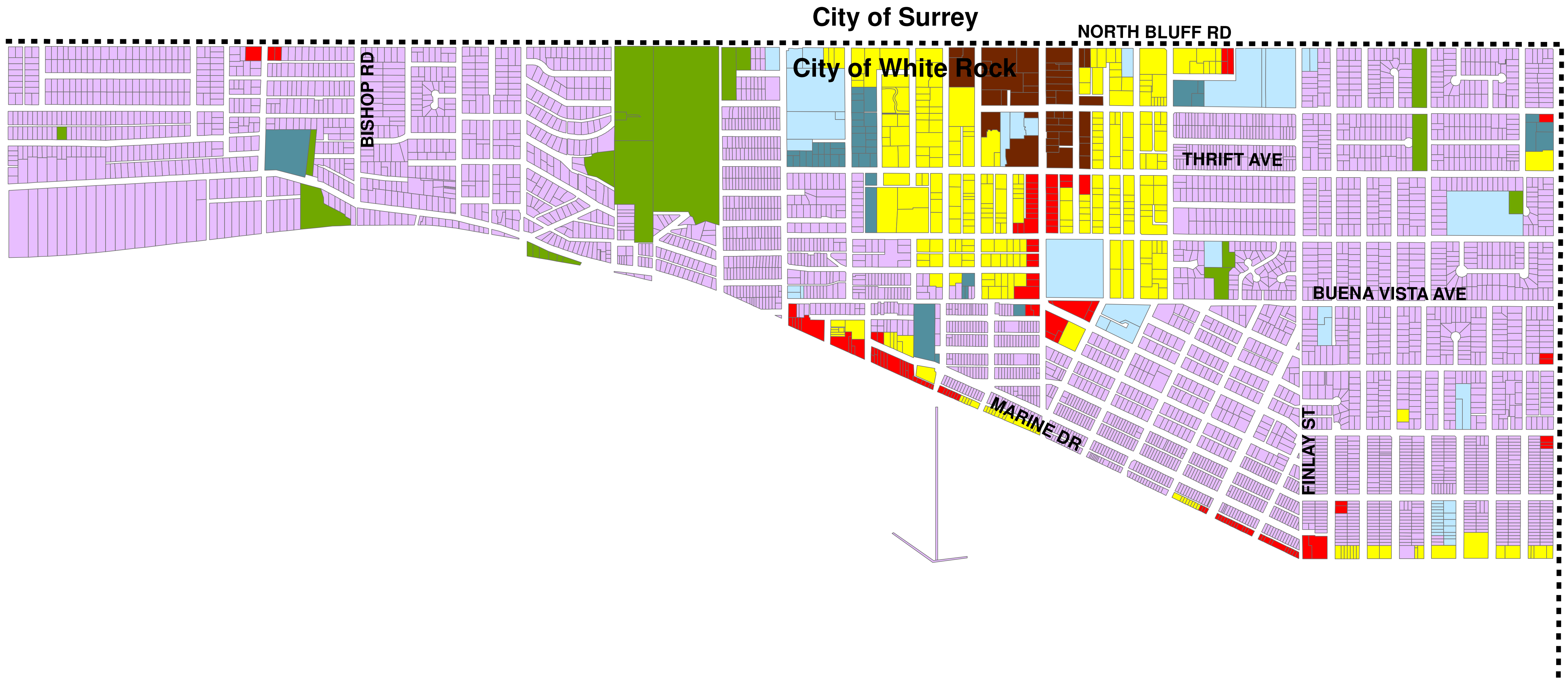
AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Areas of Disconnected Roof Leaders

Figure 2.3



The City of White Rock

Master Drainage Plan Update

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

AECOM

0 100 200 400 600 800
Meters

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OCP Land Use

Figure 2.4

2.5 Design Criteria

Stormwater drainage design criteria are based on providing a level of service to the public to mitigate flooding and damage to properties. Currently, the City requires that the minor system be designed for the 10-Year storm event, while the major system is to be designed for the 100-Year event as per the Subdivision Bylaw # 777 Schedule B. In general, the 10-Year event is conveyed through the drainage system via sewer pipes while the 100-Year is conveyed by overland flood routes that are designed to minimize property damage and protect the public. If the 100-Year system cannot be safely conveyed through overland flood routes and has a negative effect on private properties, then a 100-Year piped system should be designed in order protect the subjected properties.

In the 2004 USL study, the governing storm events were found to be the 10-Year 1-Hour and the 100-Year 2-Hour events. As well, the short duration AES hyetographs were used as design storm events in the model analysis. Prior to initiating our assessment we reviewed these criteria by completing a verification exercise and to also determined what the governing events were for our model. We began our verification by running the model for the 10-Year and 100-Year events for 1, 2, 6, 12, and 24-Hour durations and compared the peak flow rates experienced by all the pipes in the system. We then identified which storm event and duration these peaks occurred under. From our review, the majority of the existing sewers in White Rock experienced peak flows during the 10-Year 1-Hour and the 100-Year 2-Hour events, thereby verifying the information from the 2004 study.

The design storm information is included in **Appendix B**.

3 Model Update and Calibration

The previous drainage model was developed using XP-SWMM software and, as such, we elected to keep the model in XP-SWMM format and update it accordingly. The primary tasks associated with the model update included the following tasks:

- Unifying the existing drainage model files into one complete model for the entire City from the previously separated East and West models for the 10-Year and 100-Year design events;
- Reviewing the percent of directly connected impervious area in those catchments with disconnected roof leaders and adjusting the percentage accordingly to reflect current conditions;
- Incorporating new drainage infrastructure into the model based on GIS data and as-built drawing information;
- Updating the model to reflect current base flow conditions based on information obtained from the 2012 flow monitoring program; and,
- Reviewing existing catchment areas and verifying that they are assigned to the correct drainage node in the model.

3.1 GIS Data Review

A review of the GIS drainage system data was completed to identify any "data gaps" between the City's current GIS information and the existing XPSWMM hydraulic model and highlight pipes that had been constructed since the model was last updated. As-built drawings were also used to input pipes recently constructed and not yet in the GIS database. A few of the GIS data gaps that were noted include the following items:

- Pipes were listed in GIS with no inverts, these pipes were only added to the model if required for connectivity and in general these were small diameter local storm sewers;
- The GIS shapefiles provided by City showed conflicting information in some pipe diameters when compared to those in the model. In some instances the model diameter was assumed correct based on downstream pipe diameter; and
- A significant number of pipes in GIS (52%) do not have an entry for the pipe age or installation date. This does not impact the hydraulic modelling but reduces the data available for the asset condition assessment.

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 drainage 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. All newly imported links and nodes were then tagged as "2012" in the object description in order to quickly distinguish them from other links and nodes already present in the model.

A total number of 2731 links and 3401 nodes were identified in the updated model, of which 48 new links and 93 new nodes were added. The model was then further updated with information provided by the City and recent as-built drawings for new storm sewers constructed along Stayte Road, Victoria Avenue, and Finlay Street that have not been updated in the latest GIS database.

3.2 Hydrologic & Hydraulic Model Parameters

The XPSWMM software program utilizes both runoff and hydraulic modules. The runoff module generates hydrographs based on rainfall (or hyetographs), soil characteristics, catchment widths, depression storage, impervious area, and infiltration rates. The hydraulic module routes these hydrographs through the drainage system, on a real time basis, from the start to the end of the rainfall event. The drainage system is represented as links (pipes or other conduits) and nodes (manholes or other junctions). Therefore, the hydro-dynamic model provides simulated results that emulate the real flow pattern and parameters such as flow rates, velocities, water depths, and volumes.

The runoff module in XPSWMM requires input of various hydrologic parameters that define the catchment characteristics to be assigned to the nodes. These parameters include:

- catchment area, percent imperviousness, width, and overland slope;
- initial abstractions (impervious and pervious areas);
- soil infiltrations (Horton’s infiltration); and
- rainfall hyetographs.

Table 3.1 summarizes the hydrologic parameters that were used for the model under winter infiltration conditions and compares them to the previous models parameters. In general, there was a good correlation in model parameters between the 2004 and 2012 models, with the exception to the parameters used to model percent impervious from lots with disconnected roof leaders.

Table 3.1 Comparison of Hydrologic Parameters

Parameter	Value Used (2004)	Value Used (2012)
Horton’s Max Infiltration Rate	35 mm/hr	45 mm/hr
Horton’s Min Infiltration Rate	5 mm/hr	2.5 mm/hr
Horton’s Decay	0.00115 /sec	0.0009 /sec
Depression Storage (Impervious)	0.5 mm	0.5 mm
Depression Storage (Pervious)	5.0 mm	5.0 mm
Impervious ‘n’	0.11	0.11
Pervious ‘n’	0.30	0.30

Table 3.2 summarizes the land use imperviousness ratios that were used for the model based on the calibration results. Note that the single family residential imperviousness (ie. directly connected) is much lower for lots where the houses have disconnected roof leaders.

Table 3.2 Land Use Imperviousness

Land Use	Imperviousness (%) (Directly Connected)
Single Family Residential	30-55
Single Family Residential Roof Leaders Disconnected	18
Multi-Family Residential	60
Commercial & Institutional	90
Open Space and Recreation Areas	10
Roads	90

The hydraulic model represents the storm sewer network and conveyance systems within the watershed. Ground and invert elevations must be input into the model to assist in simulating a hydraulic grade line (HGL) throughout the entire system, as well as identify locations of surcharging and surface flooding. The hydraulic model has the capability to model detention ponds, orifices, weirs, pumps, backwater effects and varying downstream boundary conditions (e.g. free outfall, constant water level, tidal). As a result, peak flow attenuation and lag time from detention ponds and surcharged systems are accounted for. A list of the key parameters used in the hydraulic module is summarized below:

- pipe and culvert size;
- channel shape and dimension;
- Manning's "n" (pipes, culverts and channels);
- ground and invert elevations;
- baseflows at various locations; and
- Outfall boundary conditions.

As part of the validation process, both the runoff and hydraulic parameters are adjusted in order to replicate metered results.

3.3 Rainfall and Flow Monitoring Data

Previously, the 2004 USL model was calibrated using flow monitoring data obtained from February to April 2001. For the 2012 model update, we utilized this model as a base and completed a new flow monitoring program with SFE Global to further validate the model to ensure it was reflective of the most current conditions in White Rock.

In order to 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 3.1** and details of each site are summarized in **Table 3.3** below.

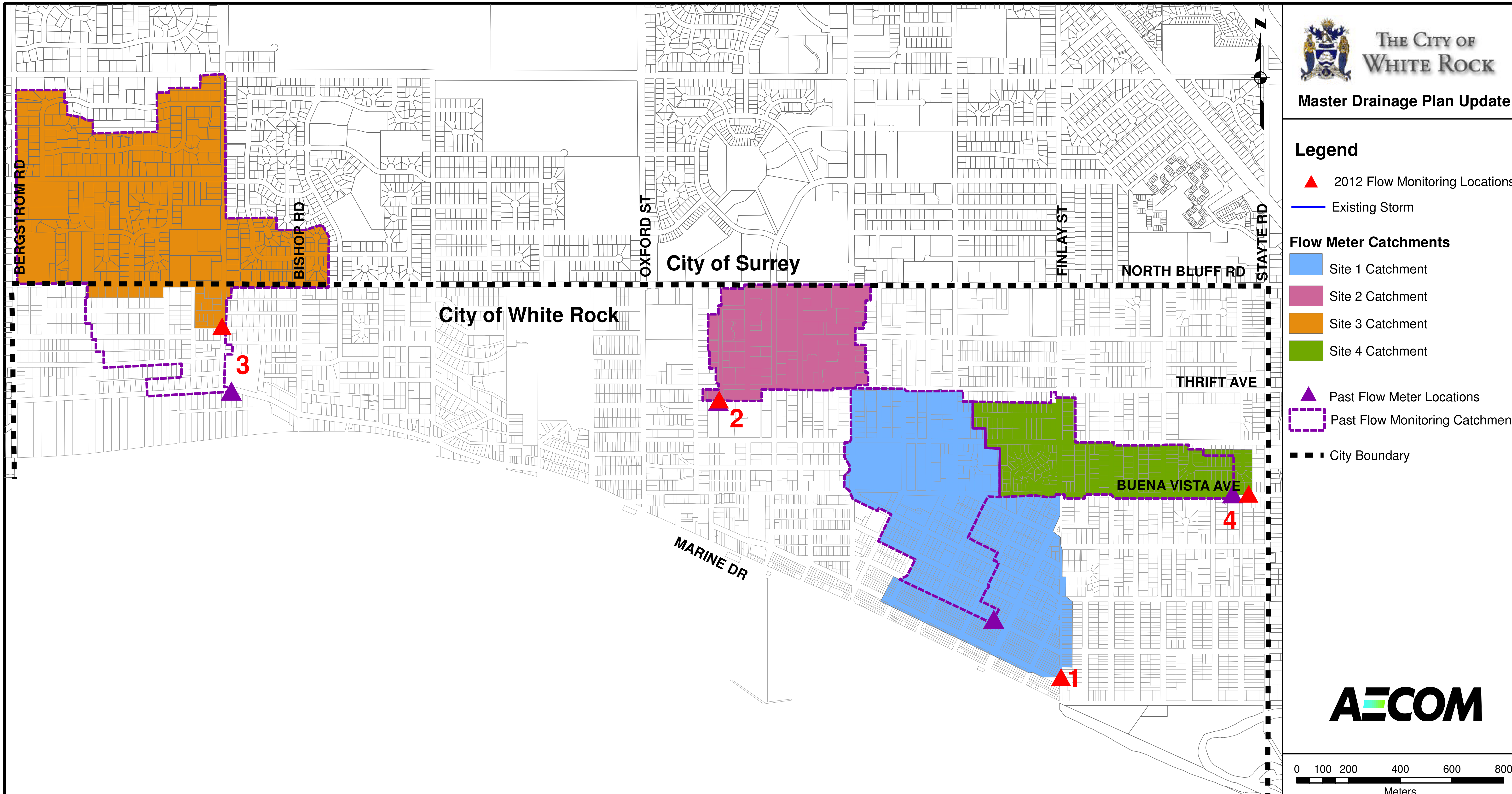
Table 3.3 Summary of Installed Flow Meters

Site No.	Location	Catchment Area (Ha)	Flow Meter Type	Inlet Pipe Dia. (mm)	Outlet Pipe Dia. (mm)
1	Finlay St. at Victoria Ave.	62	Area Velocity	1200	1200
2	Vidal St. south of Thrift Ave.	25	Area Velocity	600	600
3*	Laurel Ave. at Nichol Rd.	67	Area Velocity	750	750
4	15957 Buena Vista Rd.	26	Weir	600	600

*Approximately 4.2 Ha of the Site 3 catchment is located within the City of White Rock with the remainder in Surrey

The City of White Rock

Master Drainage Plan Update



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Project No.	Date
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Storm Flow Monitoring Sites and Catchments

Figure 3.1

Flow data was received for the months of January, February, and March of 2012. Upon review, the month of January showed inconsistency in the data, specifically for Sites 2 and 4. We communicated this with SFE and as a result they replaced the Isco velocity meter with a Sigma AV meter at Site 2 on February 8 and the AV meter at Site 4 was replaced with a 350mm weir on February16. After these modifications the data collected from Site 4 had improved but Site 2 remained problematic. Upon further review of Site #2 it was determined that during dry weather conditions the manhole experienced pooling and also became very turbulent during storm events affecting the quality of data collected. Consequently, we focused our analysis on utilizing the data obtained for Sites 1, 3, and 4 for the model calibration and validation.

3.4 Model Validation

First validation event was selected to be February 16 to 19, 2012 which included the largest rainfall event during the flow monitoring period. Although this event was less than a 2-Year event as per the City's IDF curve it was largest event recorded and data was available for each of the three monitoring sites (data from Site 2 was discarded for reasons noted above). The flows generated from the model were graphed and compared to the corresponding flow data to determine if the model accurately represented existing conditions for each site.

Second validation event was selected to be March 8 to 18 as there were two significant rain events during this period. This event was also less than 2-Year event. Both the February and March events are plotted against the City IDF curve for the White Rock rain gauge in **Figure 3.2** below. Although the accuracy of the model can be further improved by calibrating to a 5 to 10-Year event, unfortunately no big event occurred during the flow monitoring period.

Figure 3.2 IDF Curve with Calibration/Validation Events

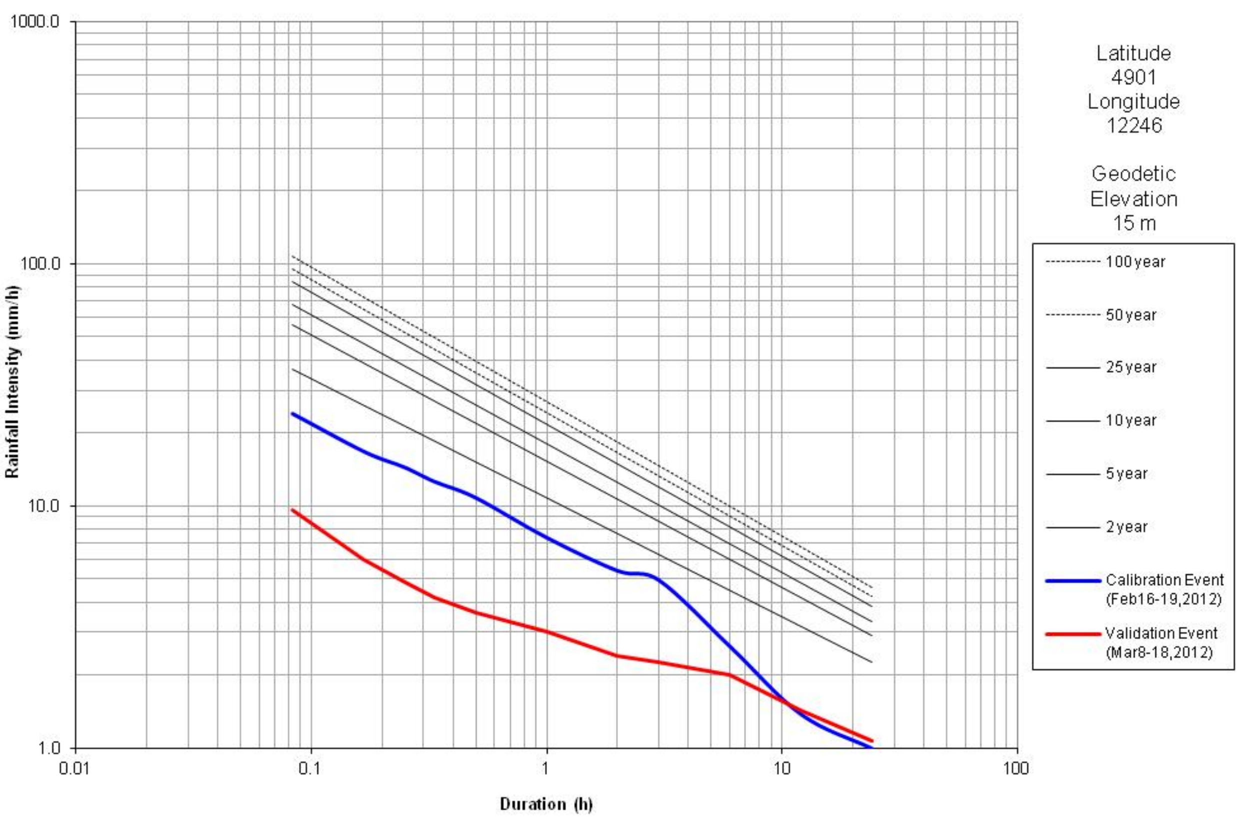


Table 3.4 summarizes the comparison of volume and peak flows between the metered data versus the modelled data for each flow monitoring site under the validation events. Individual validation event plots are included in **Appendix C**.

The validation results for storm volume, runoff coefficient and peak flow at all Sites show acceptable levels of model accuracy. Typically we recommend the percent difference is within 10% which can be difficult to achieve under all circumstances. For instance the percent difference values for Site 3 are slightly higher than 10% for both volume and peak flow and can be attributed to the model flows dropping at a slower rate than the meter flows after the February 17 event resulting in a greater volume difference. The metered flows for Site 1 and 3 also show possible malfunction in the AV meter just after the peak rain event which also reduces the total meter volume used for the comparison. Overall, the results presented are slightly conservative but reasonable given the quality of the data.

Table 3.4 Modelled Versus Metered Peak Flows and Volumes

Event #1 (Feb.17 to 19, 2012)			**VOLUME COMPARISON**						**PEAK FLOW COMPARISON**		
			Metered Data			**Modelled Data**		**Percent Difference (Storm Volume)**	**Metered Data**	**Modelled Data**	**Percent Difference (Peak Flow)**
Site No. and Location	Catchment Area	Calibration Event	Cumulative Rainfall Depth	Storm Volume	Runoff Volume Coefficient	Storm Volume	Runoff Volume Coefficient		Peak Flow	Peak Flow	
	(Ha)		(mm)	(m^3)		(m^3)		**(%)**	(L/s)	(L/s)	**(%)**
Site 1 Finlay St.	62	Feb.17-19 (2 Days)	25.80	6,352	0.40	6,919	0.43	**8.2**	575.4	622.0	**7.5**
Site 3 Laurel Ave.	67	Feb.17-19 (2 Days)	25.80	9,428	0.52	11,004	0.61	**14.3**	313.9	354.0	**11.3**
Site 4 Buena Vista	26	Feb.17-19 (2 Days)	25.80	917	0.14	1,007	0.15	**8.9**	149.4	155.0	**3.6**

Event #2 (March.8 to 18, 2012)			**VOLUME COMPARISON**						**PEAK FLOW COMPARISON**		
			Metered Data			**Modelled Data**		**Percent Difference (Storm Volume)**	**Metered Data**	**Modelled Data**	**Percent Difference (Peak Flow)**
Site No. and Location	Catchment Area	Validation Event	Cumulative Rainfall Depth	Storm Volume	Runoff Volume Coefficient	Storm Volume	Runoff Volume Coefficient		Peak Flow	Peak Flow	
	(Ha)		(mm)	(m^3)		(m^3)		**(%)**	(L/s)	(L/s)	**(%)**
Site 1 Finlay St.	62	Mar.8-15 (7 Days)*	44.0	15,533	0.57	13,438	0.49	**13.5**	153.0	161.0	**4.9**
Site 3 Laurel Ave.	67	Mar.8-18 (10 Days)	58.4	36,801	0.90	33,710	0.82	**8.4**	241.0	266.0	**9.4**
Site 4 Buena Vista	26	Mar.8-18 (10 Days)	58.4	6,155	0.41	5,913	0.39	**3.9**	149.4	145.0	**2.9**

* Flow meter at Site 1 was out of service on March 16 during the peak of the rain event and was not repaired until March 30

At Site 4, the runoff coefficient for both meter data and model data is low compared to Sites 1 and 3. This is a direct result of the disconnected roof leaders within the eastern part of the City particularly in the catchment area for Site #4.

Results for the validation events are generally within the 10% percent difference guideline but the model volumes are lower than recorded values at each Site. Predicted peak flows are greater than measured flows at Sites 1 and 3 and slightly lower at Site 4.

In addition to the validation results, the model was run for historical rain events from January and February 2001 using flow data provided by SFE for the City and again produced favourable results.

4 Hydraulic Analysis

4.1 Sewer System Capacity

Keeping in mind that the primary goals of the model are to develop a 10-Year Capital Asset Plan and OCP review for the City, the future OCP land-use scenario was analyzed to determine key upgrades and improvements needed to support growth. There are very few greenfields that can be developed in the City of White Rock, so opportunities for drainage system expansion are rare. Most of the City's drainage system is in a state of infrastructure renewal.

We compared existing land use designations with OCP land use and determined which model sub-catchments are expected to have significant changes in footprint. Impervious rates were increased in these areas to simulate the higher runoff. Catchment widths were also increased for the future scenario as increase in impervious areas and renewal/replacement of old inefficient infrastructure will result in shorter times of concentration and an increase in peak flows. The model was utilized to perform an analysis of the hydraulic capacity for the existing drainage system under future OCP land use conditions. A total of 278 out of 1,466 modelled existing storm pipes were found to surcharge under the 10-year storm event. This means that 19% of the modelled pipes within the system are unable to convey the 10-year storm event without surcharging. The corresponding total length of surcharged pipes is 14,391m.

Figure 4.1 shows the existing storm system capacity assessment for the 10-year event under future OCP land use conditions. The figure highlights modelled pipes that have a Qmax/Qcapacity greater than 0.85 which is a prediction of the pipes that may surcharge within the model accuracy. The hydraulic grade line (HGL) at manholes or model nodes is also shown and includes the following gradation:

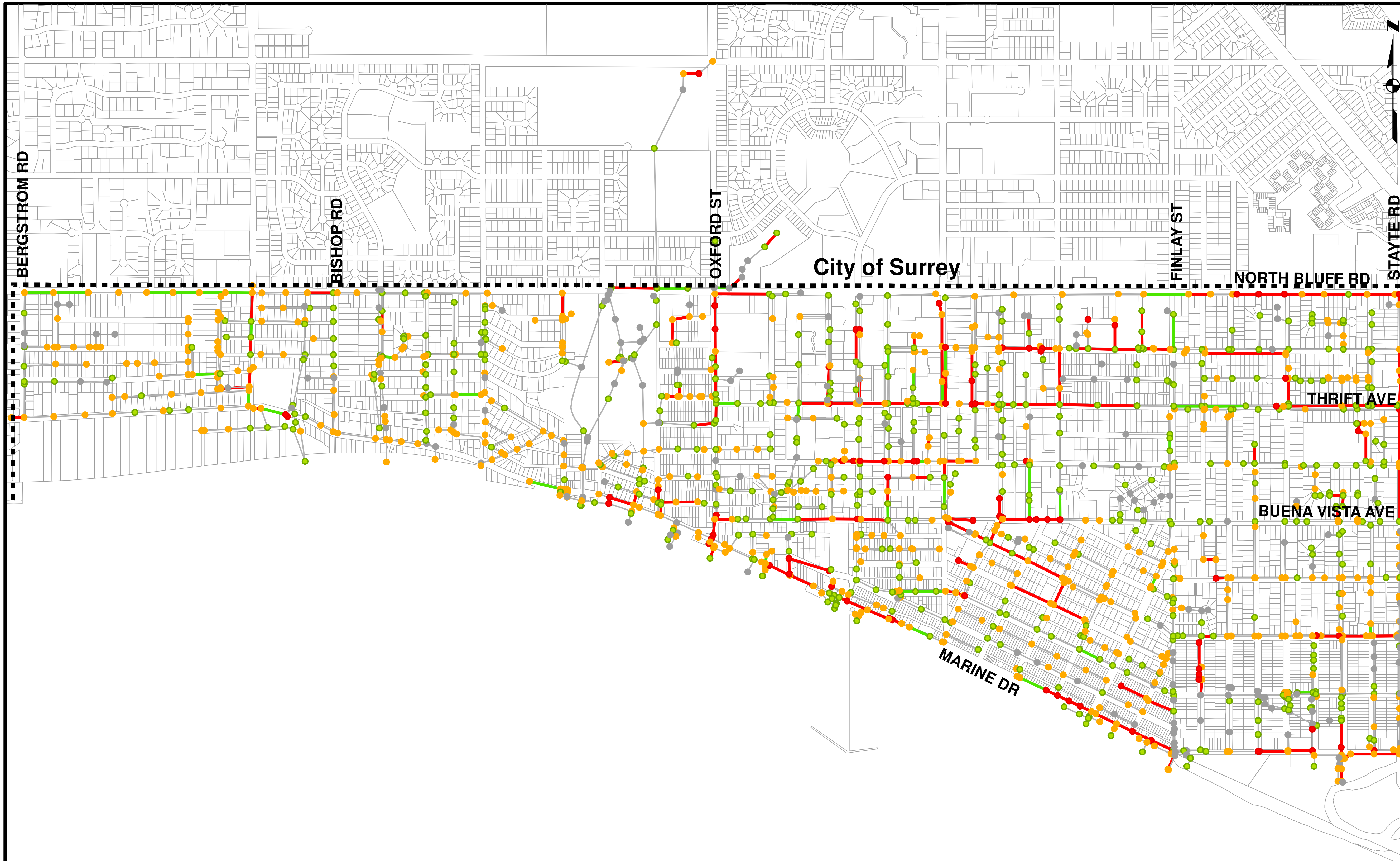
- HGL above ground: surface flooding potential
- HGL between 0 – 1.2m: crawl spaces flood potential
- HGL between 1.2 – 2.4m: basement flood potential
- HGL less than 2.4m: low risk

4.2 Pump Station Capacity

There are three floodplain zones along Marine Drive which fall within the Ocean/Campbell River floodplain. These zones are identified in Schedule G of the City's Official Community Plan (OCP). There are two drainage pump stations along Marine Drive serving these flood zones. The Oxford PS is located west of the foot of Oxford Street and discharges directly to Semiahmoo Bay; and Habgood PS is located at the foot of Habgood Street and discharges to Little Campbell River. The catchment areas for Oxford PS and Habgood Pump are 6.5 Ha and 14 Ha respectively.

A summary of the pump station capacity and model predicted inflows to the pump stations is shown in **Table 4.1.** Model results shows that both the pump stations are undersized to convey 10-year peak flows. Based on the discussions with the City staff, both pump stations are very old and no as built information is available for them. Capacities of both the pump stations were based on available pump curves and assuming one pump running.

Habgood PS has two submersible Flygt pumps. One pump was replaced about 2 year ago with a new pump of higher capacity. Since then the City staff has not noticed any flooding upstream of pump station along Marine Drive. Field review of the pump station shows that there are two separate discharge pipes exiting the wet well. Due to overgrown



THE CITY OF WHITE ROCK

Master Drainage Plan Update

Legend

Existing Manholes Freeboard

- HGL>2.4m below ground
- HGL between 1.2m - 2.4m
- HGL between 0 - 1.2m
- HGL above ground

Pipe Flow/Capacity, q/Q

- Less than 0.85
- 0.85 - 1.0
- Greater than 1.0

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Existing System with OCP Landuse for 10-Year 1-Hour Event

Figure 4.1

vegetation in the vicinity of the pump station, the size and location of outfall pipe into Little Campbell River can't be determined. Discussion with City staff also indicate that there is no gravity overflow pipe at this location and all flows are routed through the pump station. It is highly likely that both the pumps are running during minor events such as 2-Year to 5-Year return period.

Oxford PS has two 5 HP submersible Flygt pumps. Low flows from the upstream manhole just west of the wet well are diverted to the pump station. There is a 600mm diameter pipe running south from the same manhole and outfalls to the beach. GIS data and field review show that this pipe is partly at reverse grade with a flap gate installed on the downstream side inside the manhole. It is likely that during a major storm, when the flows exceed the capacity of the pump station, it overflows via this 600mm diameter pipe thereby preventing flooding in the parking lot area. Model shows the pipes upstream of the pump stations surcharging during a 10-Year event until the water level reaches the overflow pipe invert and discharges via overflow pipe. The capacity of overflow pipes is also estimated assuming free flow conditions (low tide) as shown in **Table 4.1.** However, in the event of a major storm occurring during high tide could result in flooding in the parking area.

Table 4.1 Pump Station Summary

Pump Station Name	Existing Pump Capacity[1] L/s	Overflow Pipe Capacity[3]	10-Year Peak Inflow to Pump Station[2] L/s	100-Year Peak Inflow to Pump Station[2] L/s	Upgrade Required
Oxford	40	523	330	430	Yes
Habgood	160	No Overflow	708	1000	Yes

Notes:

1. *Existing capacity based on One Pump Running.*
2. *Peak Inflow to Pump Station assuming high tide in Semiahoo Bay, resulting in no flow from overflow pipe and all the inflow routed through the pump station.*
3. *Overflow pipe capacity based on tide elevation below outfall invert. Pipe inverts based on GIS provided by City.*

Since both the pump station are very old, we recommend that prior to any pump station upgrades, the City should carry out a detailed assessment of both the pump stations that includes structural as well as electrical components inspection. The final upgrade strategy should also look into the effects of high tide on the overall capacity of the drainage system.

4.3 100-Year Overland Flow Assessment

The major system and overland flow routes were reviewed for a 1 in 100-year event. The design event was simulated to predict 100-year peak flows and HGL’s in the existing system (with no upgrades).

Runoff from the roof leaders and building drains is piped (through service connections) into the local sewer network. Runoff from the road surface flows overland and enters the sewer network through catch basins and lawn drains. For minor storm events (less than a 10-year), almost all the runoff from the road surface is collected through catch basins and drained into the sewers. During a major event (1 in 25-year or greater), the catch basins may not have sufficient inlet capacity (especially on steep slopes) and not all of the road runoff can enter the sewer system. This causes excess runoff to continue flowing overland, usually along the curb, and either get collected by other catch basins or continue downhill to a low-point where it ponds potentially causing localized flooding.

It is important to be aware of the overland flow routes and the localized low points, should a major event occur or even if a series of catch basins gets plugged with ice, silt or branches. We reviewed the major drainage system and overland flow by:

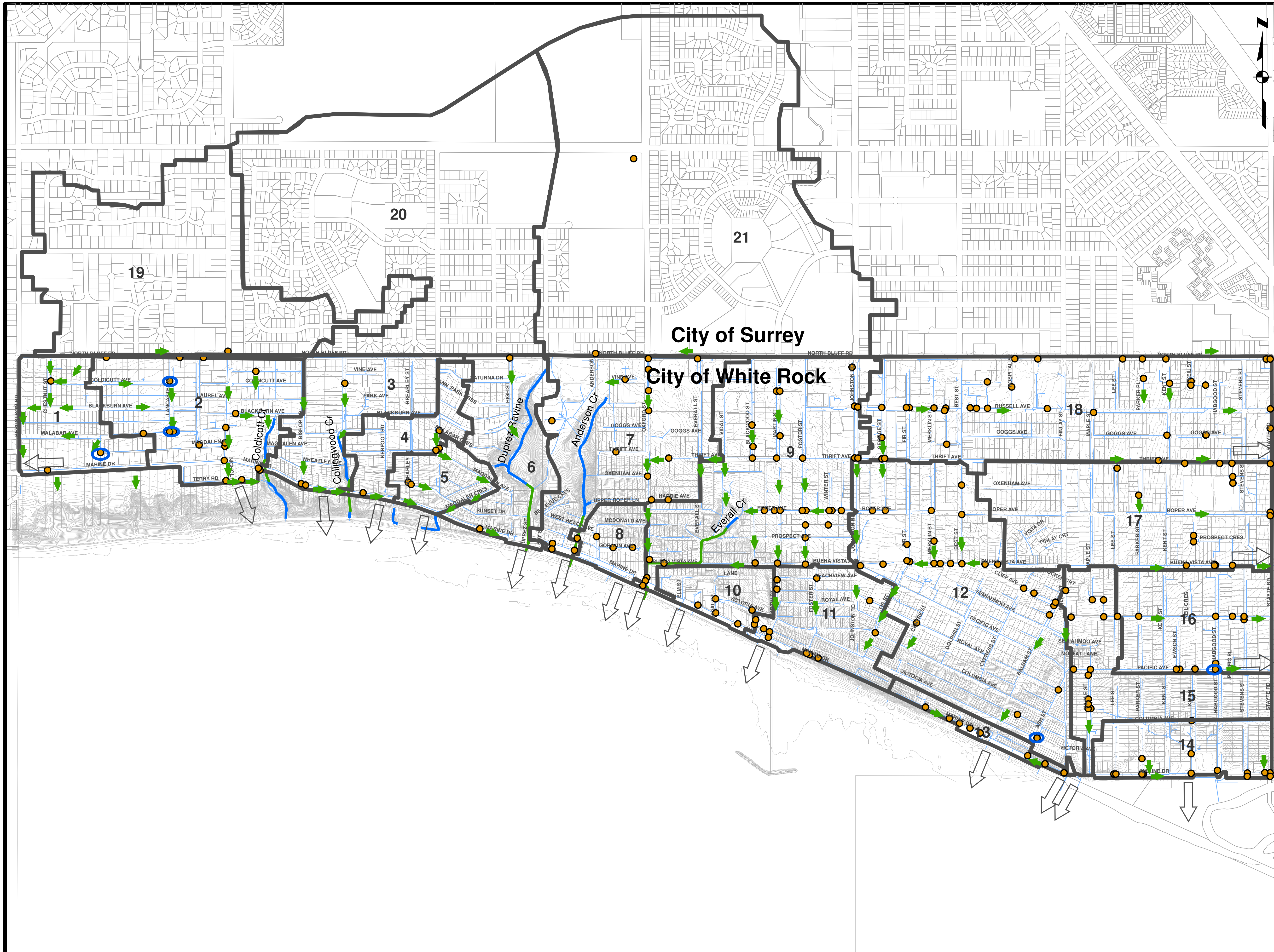
- Simulating a 100-year event and identifying where manholes are predicted to surcharge to the road surface; and
- Reviewing topographic maps to identify overland flow routes and low points;

Figure 4.2 illustrates the locations where the existing storm sewer system is predicted to surcharge to the road surface during a 100-year event and major low points.

4.4 Trans-boundary Flows

There are four locations where storm sewers drain from White Rock to Surrey. Three are located along Stayte Road and the fourth is at Bergstrum Road. The three at Stayte Road take flows from White Rock into Surrey from Thrift, Buena Vista, and Pacific Avenues. Flows from the Buena Vista and Pacific connections drain into McNally Creek (and on to Little Campbell River) which is known to have erosion issue. No increase in flows at these locations is recommended, and the proposed strategy is to maintain base flows up to the existing 10-Year peak flows into Surrey while managing the high 100-Year flows as well as any additional flows within the City of White Rock.

There are also three locations where flows from Surrey enter into White Rock. These three are all located along North Bluff Road, and one takes in discharge from Surrey's Southmere Detention Ponds. The three corresponding catchment areas located within Surrey are illustrated on **Figure 2.1**. With exception of Nichol Road, the catchments from Surrey enter into dedicated diversion sewers in White Rock.



The City of White Rock

Master Drainage Plan Update

Legend

- Storm Catchments
- Enclosed Watercourse
- Open Channel
- Direction of Catchment Outflow
- Overland Flow Routes
- 100YR_2HR HGL Above Ground (Existing System)
- Potential Local Flooding during 100 YR Storm

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Overland Flow Routes

Figure 4.2

5 Sewer Condition Assessment

To assist with the long range capital plan we have provided a review of the available pipe age information and CCTV data.

5.1 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 it became apparent that 52% or approximately 55.4Km of the storm sewer collection system did not have an entry for pipe age. For the remaining pipes that did have age attributes the earliest year of installation is noted at 1966. A summary of the pipe age or year of installation is shown in **Figure 5.1**. In general, the following key items are required for a municipality to properly plan for existing infrastructure replacement:

When the sewer was installed;
Expected life cycle of the sewer;
Estimated time to rehabilitate or replace; and,
Is the asset technologically or commercially obsolete?

From available GIS data, **Table 5.1** categorizes the existing sewers into their corresponding year of installation.

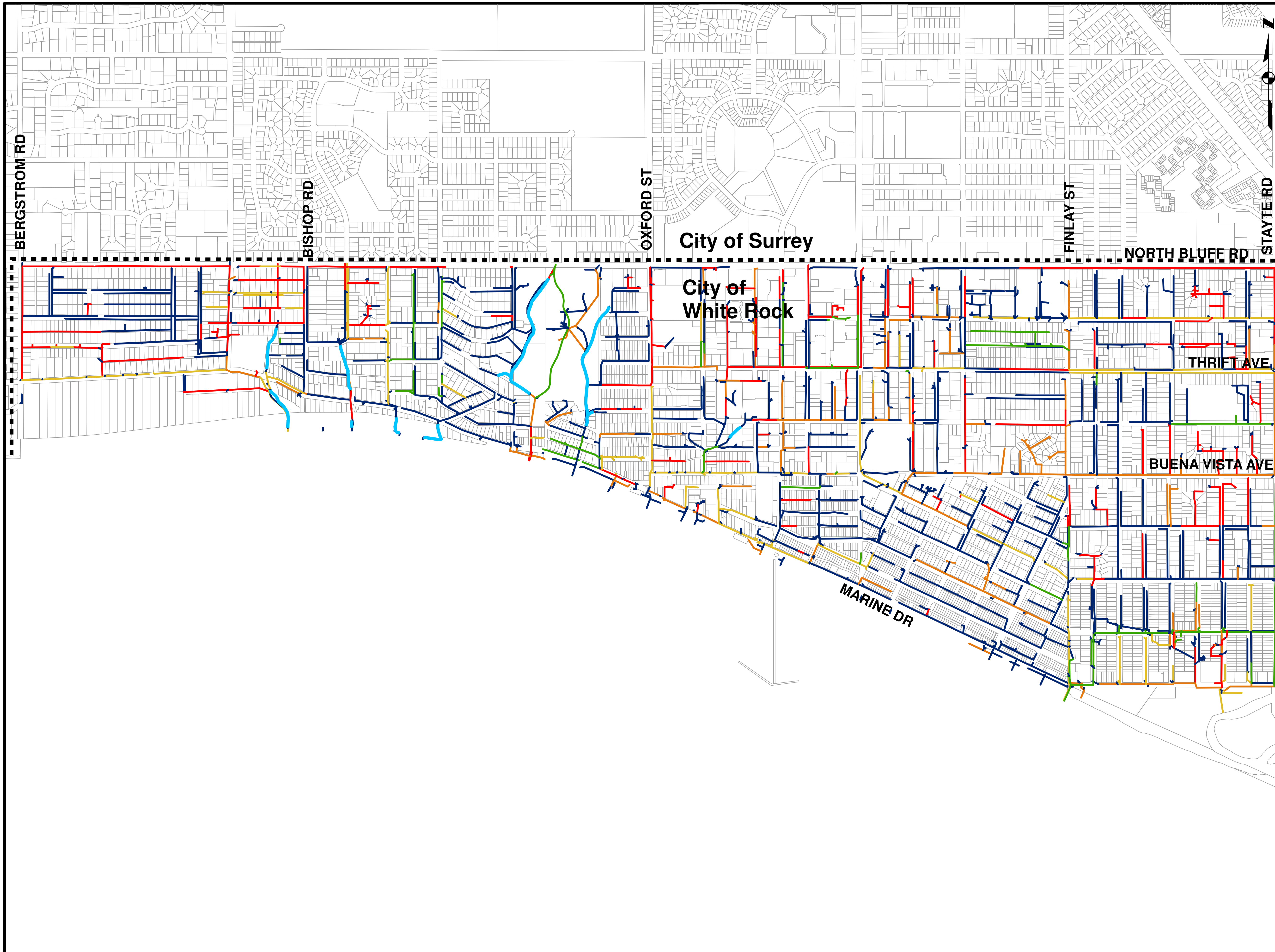
Table 5.1 Summary of Pipe Year of Installation

Year Installed	Length of Pipe	Number of Pipes
2012-2001	7,419	189
2000-1991	11,436	236
1990-1981	12,244	267
1980-1971	16,708	338
1970-1966	2,872	52
Unspecified	55,418	1,650
Total	**106,097**	**2,732**

5.2 CCTV Inspections

A summary of the CCTV data provided is also included and reviewed in conjunction with the system capacity assessment data for the Capital Plan. The data that was reviewed includes the following information provided by the City:

- May 16 to June 23, 2011 by McRaes Environmental Services Ltd – 189 survey videos were recorded for a total length of 4,154 metres of storm sewers inspected.
- May 2 and 3, 2012 by McRaes Environmental Services Ltd – Marine Drive from Bishop Road to High Street
- March 23, 2012 by ABC Pipe Cleaning Services Ltd. – Victoria Avenue from Fir Street to Balsam Street



The City of White Rock

Master Drainage Plan Update

Legend

- Stream

Storm Pipe Year of Installation

- Unknown
- 1966 - 1980
- 1981 - 1990
- 1991 - 2000
- 2001 - 2009

0 100 200 400 600 800
Meters

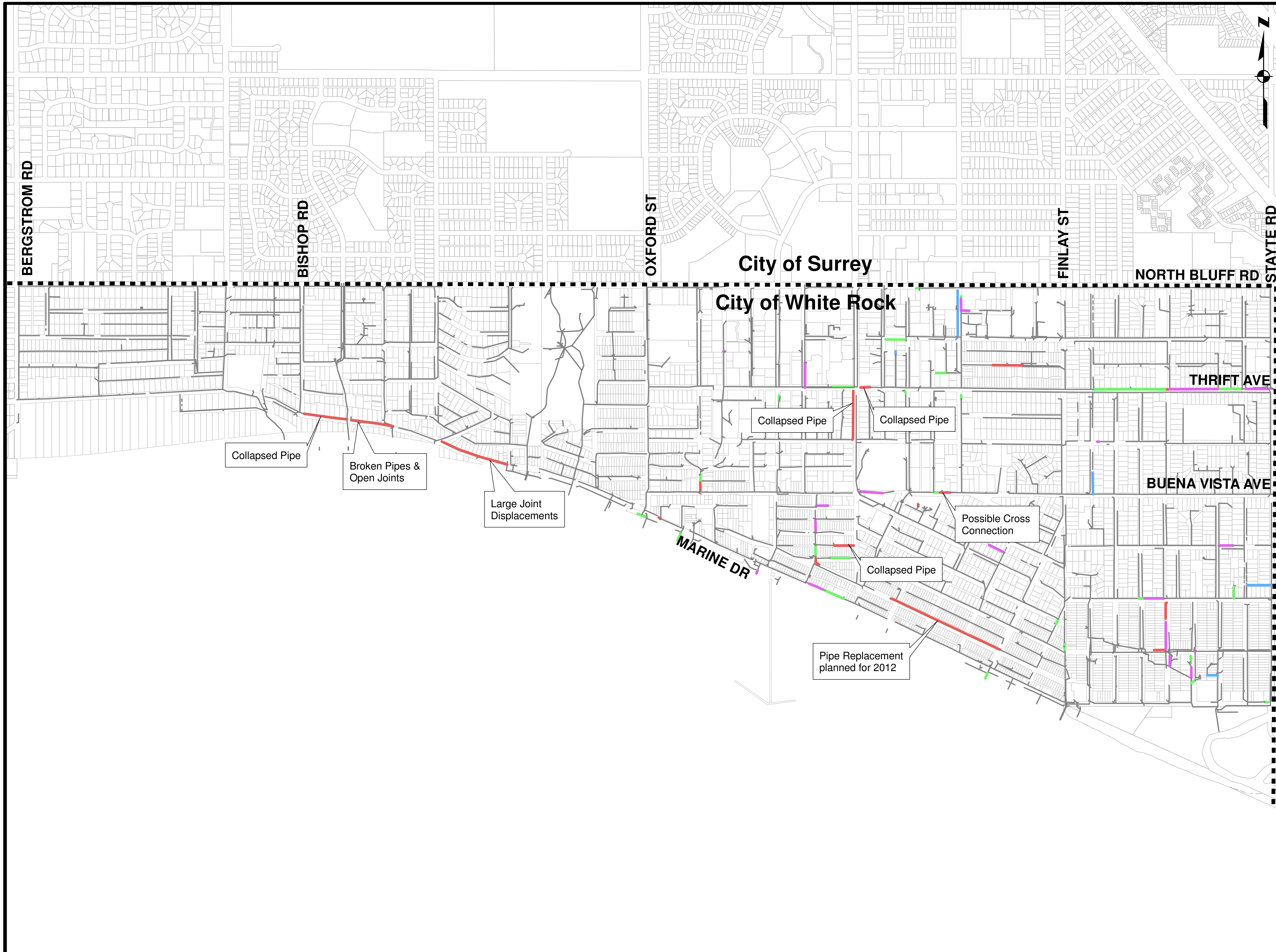
Project No.	Date
60239267	June 2012

Existing Storm Pipe Year of Installation

Figure 5.1

A summary of the location of pipes with structural and service defects as per the CCTV assessments is shown in **Figure 5.2**. Note that detailed reviews of the WRc condition ratings provided by the CCTV contractors have not been completed for the CCTV data collected in 2011 as this was not part of the scope of works for the DMP. Detailed reviews for the Marine Drive and Victoria Avenue assessments were completed under separate contracts with the City.

The total length of the system CCTV data that was available and reviewed is only approximately 5% of the entire system. In such case we recommend the City continue to collect conduct condition assessments of the storm sewer system by initiating contracts for approximately 10% (or 10 Km) of the system per year.



THE CITY OF WHITE ROCK

Master Drainage Plan Update

Legend

- Lots
- Storm Pipe
- Structural and Service Defect
- Unable to survey
- Service Defect
- Structural Defect

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Storm Sewer Condition Assessment Results

Figure 5.2

6 Recommendations

This section summarizes our recommendations for the Updated Drainage Master Plan after completing the system 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**.

The proposed upgrades were derived based on the following fundamental criteria:

- Maximizing existing outfall capacities;
- Minimizing flows to Little Campbell river; and
- Maintaining base flows and existing flows to Surrey (i.e McNally Creek) and diverting increases in peak flows from new town centre developments away from the City of Surrey.

6.1 Comparison with Previous Recommendations

Following are the some of the important differences between the 2004 USL study and this DMP:

- The USL study was completed in 2004 and since then, development has continued in the City. As a result the construction of new drainage infrastructure has not been reflected in the 2004 drainage model.
- In the 2004 study, trans-boundary flows issues with City of Surrey were not clearly described. The flow conditions at these locations are important to review as there can be downstream implications from increased development activity. The proposed strategy in this DMP is to maintain low flows to Surrey while managing the high flows within the City of White Rock.
- All the trunk sewers in 2004 study were designed for containment of 100-Year peak flows. It recommended a total of 18.6km of upgrades for storm sewers and proposed diversion sewers. In comparison, in this DMP we have assessed the system using the 10-Year peak flows for all pipes (with exception of outfalls and major creek/road crossings). In total we recommended 11.4km of storm sewer upgrades. In comparison the City of Surrey uses 5-year design storm for sizing the minor systems including piped trunk sewers with catchment area greater than 20ha.
- The flow routing and diversions provided in 2004 study are different from this DMP. For example, USL proposed diverting major flows to Little Campbell River via the proposed new outfall at Stayte Road, whereas in this study we proposed diversion of flows away from Little Campbell River due to issues with Semaihmoo First Nations.
- The previous study used 3-tier priority ranking for all the proposed upgrades in the city. In this study a 10-year capital upgrade plan was prepared keeping in mind that the City's allowable annual budget of approximately $1 million a year for drainage related capital improvements. We prioritized the upgrades for each year for first five years. All the remaining upgrades can be completed in the next five years.

6.2 Proposed Upgrades

Due to steep terrain, limited undeveloped land and high land cost, neither detention ponds or underground storage tanks were considered feasible. So diversions and sewer upgrades were considered. The proposed upgrades are shown in **Figure 6.1.** The figure includes all the proposed upgrades based on the capacity assessment criteria mentioned above and also includes the existing modelled diameter for comparison. Additional detail on the major upgrades and diversions discussed below and provided on **Figures 6.2 and 6.3**.

THE CITY OF WHITE ROCK

Master Drainage Plan Update

Legend

- Prop Dia — Proposed Upgrade
- Exist Dia — Existing Sewer
- Upgrade Based On Condition
- Pump Station Upgrade

AECOM

0 100 200 400 600 800
Meters

Project No.	Date
60239267	June 2012

Proposed Upgrades

Figure 6.1

Pacific and Habgood Drainage Improvements – System improvements required to further reduce the likelihood of flooding at this intersection and reduce high flows to Surrey (i.e. McNally Creek).

Parker Street Diversion – This project is an alternative to the Stayte Road outfall that was previously proposed and has been abandoned until a solid agreement can be made with the Semiahmoo First Nation for construction of a new outfall. The objective is to control peak flows within White Rock and only discharge existing flows and base flows to the City of Surrey and McNally Creek. The diversion sewer is proposed on Parker Street between Thrift Avenue and Pacific Avenue and shown in **Figure 6.2**. Flow diversion manholes will be required at Thrift Avenue and Buena Vista Avenue to divert base flows to Surrey and high flows south towards Pacific Avenue. Flows intersecting Parker Street at Cliff Avenue and Ropar Avenue would be diverted south towards Pacific Ave. Once at Pacific Avenue all flows would be diverted west towards Maple Street and finally outfall into Semiahmoo Bay via Finlay Street outfall.

Foster Street Diversion at McDonald Ave - Storm sewers on Roper Ave (west of Foster) are undersized to convey 10-year peak flows. We recommend a new diversion manhole at intersection of Roper Ave and Foster Street (**Figure 6.3)**, that would divert high flows south on Foster Street and allow base flows to the Creek on Everall Street.

6.3 Outfall & Culvert Summary

A review of the outfalls and major culverts under the 100-Year design storm event was completed. Of the total 25 outfalls and major culverts that are shown in the GIS, there are only 17 in the hydraulic model as shown in **Figure 6.4**. The remainder are downstream of open channel sections, within park areas or under the BNSF railway and limited information is available for them.

A summary of the outfall/culvert capacity and model results for the 100-Year event with no boundary condition is provided in **Table 6.1**. We have modelled a free outfall boundary condition as the combination of a 100-Year event combined with a high tide is very conservative and would result in most outfalls surcharging back into the system. A total of the 5 outfalls were found to be undersized to convey 100-Year flows as shown in **Figure 6.4**.

The capacity of two major culvert crossing Marine Drive (at Coldicott Creek and Collingwood Creek) was also assessed.

The City of White Rock

Master Drainage Plan Update

Legend

- Proposed Diversions
- Diversion Manhole
- Existing Sewer
- Lots

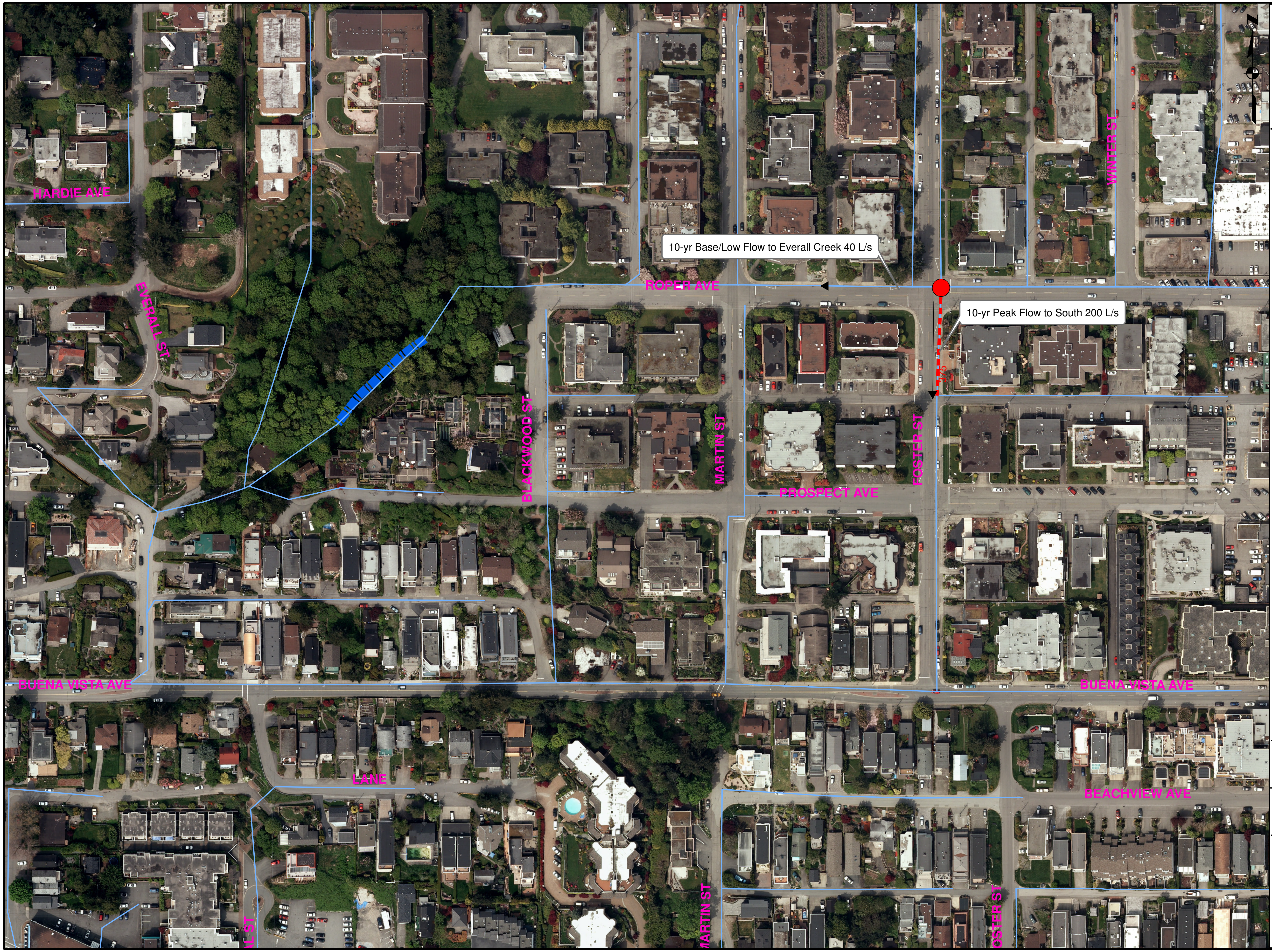
AECOM

0 75 150 300
Meters

Project No.	Date
60239267	June 2012

Proposed Diversions Parker/Pacific and Pacific/Habgood

Figure 6.2



The City of White Rock

Master Drainage Plan Update

Legend

- Proposed Diversions
- Diversion Manhole
- Existing Sewer
- Stream

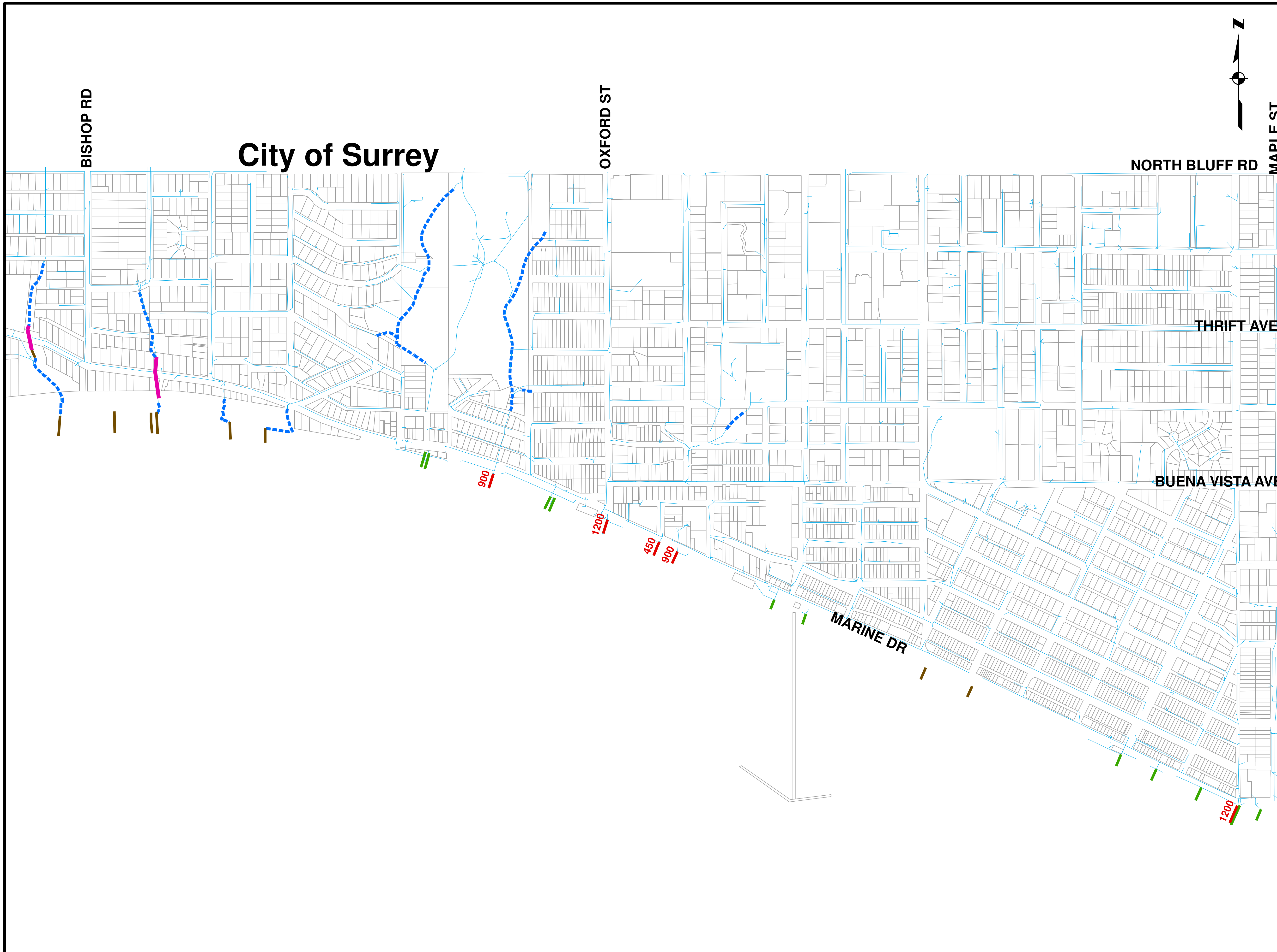
AECOM

0 25 50 100
Meters

Project No.	Date
60239267	June 2012

Proposed Diversions Foster Street and Roper Ave

Figure 6.3



The City of White Rock

Master Drainage Plan Update

Legend

Storm System Type

- Creek
- Pipe
- Existing Major Culvert
- Existing Outfall
- Prop Dia — Proposed Outfall Upgrade
- Outfall Not In Model

AECOM

0 50 100 200 300 400 500
Meters

Project No.	Date
60239267	June 2012

Oufall/Culvert Upgrades

Figure 6.4

Table 6.1 Outfall and Culvert Summary

Storm_Id	Us_Inv_Ele	Ds_Inv_Ele	Length, m	Slope	Pipe Material	Existing Diameter mm	Existing Pipe Capacity L/s**	100yr 2hr Flow* L/s	Undersized	Proposed Diamter mm	Comment
Outfall Summary											
6756	2.04	1.26	37	2.10%	Concrete	300	140	9	No		
6353	1.50	1.47	23	0.13%	Concrete	750	401	491	Yes	900	Upgrade Required for 100-Year Flow. Only minor surcharging expected with existing size.
7143	3.90	3.07	19	4.39%	PVC	375	434	24	No		
6228	0.93	0.79	19	0.73%	Concrete	600	523	386	No		
6713	2.07	1.48	22	2.73%	Concrete	600	1014	250	No		
6716	2.09	1.10	26	3.76%	Concrete	600	1190	178	No		
6449	2.53	1.84	12	5.88%	Concrete	600	1489	1349	No		
7264	0.97	0.47	54	0.92%	Concrete	900	1740	2582	Yes	1200	Upgrade Required for 100-Year Flow
6164	3.11	0.41	33	8.27%	Concrete	600	1766	2073	Yes	900	Upgrade Required for 100-Year Flow
7485	0.72	0.34	55	0.69%	Concrete	1050	2275	2617	Yes	No Upgrade	Divert High Flows to Parallel Outfall (GIS ID-7264) at Manhole upstream of railway track
6292	0.92	0.15	42	1.80%	Concrete	900	2450	3680	Yes	1200	Upgrade Required for 10-Year Flow
6758	2.82	0.97	31	5.96%	Concrete	900	4421	38	No		
6082	3.03	1.41	30	5.44%	Concrete	1050	6367	3540	No		
7322	4.50	1.38	33	9.43%	na	375	538	12	No		
7440	1.33	1.20	35	0.38%	PVC	375	127	180	Yes	450	Upgrade Required for 100-Year Flow
7437	4.09	0.00	15	27.41%	Concrete	450	1493	-	-		Not in Model-In Park
7436	4.09	3.38	15	4.76%	Concrete	500	824	-	-		Not in Model- In Park
7356	3.56	2.56	7	15.04%	Concrete	600	2381	-	-		Not in Model
3707	3.34	1.62	11	15.14%	Concrete	850	6048	-	-		Not in Model
3180	4.05	1.54	12	21.81%	Concrete	1200	18206	-	-		Not in Model
3178	3.36	1.73	12	14.17%	na	na	-	-	-		Not in Model
7313	3.69	2.83	8	10.50%	na	na	-	-	-		Not in Model
7314	3.17	2.50	11	6.09%	na	na	-	-	-		Not in Model
Culvert Summary											
7010	37.54	10.00	118	23.27%	Concrete	1200	18807 (see comment)	5828	No		Capacity based on Mannings full pipe flow. Field verification of culvert inverts required to accurately estimate the culvert capacity using inlet/outlet control nomograms
6945	53.61	51.00	54	4.82%	Concrete	900	3974 (see comment)	400	No		Capacity based on Mannings full pipe flow. Field verification of culvert inverts required to accurately estimate the culvert capacity using inlet/outlet control nomograms

Notes
* 100-Year Flow Based on Free Flowing Outfall
** Capacity Based on Inverts and Slopes in GIS data provided by City

6.4 Ravine Assessment Recommendations

An assessment of the five major ravines was completed by Thurber Engineering and AECOM. A brief summary of the findings of this study is provided in **Section 2.2** above and Thurber's full report is included in **Appendix A** of this report. The major findings were on Coldicott Creek where there was evidence of a recent landslide and it was recommended by Thurber that a detailed assessment be completed for this area as well as confirmation that private property owners on the south side of Marine Drive in the vicinity of the ravine are not discharging rainwater runoff to the slopes.

The estimated cost of completing the detailed assessment of the landslide area in Coldicott Ravine is approximately $40,000 and would include the following tasks:

- Complete a thorough examination of the existing conditions and assessment of the risk of further instability along the ravine crest;
- Provide alternative methods for improving slope stability at concept design level along with approximate costs; and
- Prepare a document for distribution to residents providing advice regarding the risks of poor property management (e.g. disposal of garden waste etc., sprinkler systems at slope crest, etc.).

This cost estimate is for the City of White Rock to include allowance for the investigation in the 2013 Capital Works program and includes allowance for drilling, topographic survey of the slope, analyses, consideration of alternatives and costing them, and meeting with the City to discuss the various issues and possible recommendations.

6.5 Additional Recommendations

Following are some of the additional recommendation for the City with respect to GIS data completion, improving the efficiency of drainage system and capital works planning;

- City should update the GIS data to include all pipe attributes (i.e. pipe diameter, inverts, material and age) possible. This can be achieved by further review of available record drawings, existing and future CCTV assessments and field measurements.
- City should conduct storm sewer condition assessment for approximately 10% of the system per year.
- According to discussions with City staff, some of the outfalls south of Marine Drive are buried under the boulders along the BNSF railway tracks. Discussions should be carried with BNSF staff to re-align the boulders so that they don't act as obstruction to peak flows during major storms.
- Field review shows that the 600mm diameter outfall south of Oxford Pump Station is clogged by debris and the Tideflex valve is separated from the outfall pipe. This outfall and all other outfalls should be inspected and cleared of any debris to prevent any flooding in the upstream area during a major storm.
- The City should conduct a condition assessment of all the outfalls to Semiahmoo Bay. For any outfall not in the current model, additional information such as inverts, diameters, upstream creek cross-section (where applicable) should be collected for capacity assessment.
- Both the drainage pump stations are old and detailed pump station condition assessment should be conducted prior to upgrading the pump stations.
- Prior to upsizing the storm sewers on Nicole Road (between North Bluff Road and Marine Drive), cost sharing discussions should be carried with the City of Surrey as it drains approximately 64ha catchment within City of Surrey boundary.

7 10-Year Capital Plan

This section details the 10-year capital plan for the City of White Rock to complete the proposed drainage improvements. The capital plan has also been divided into priority items in consideration of White Rock's allowable annual budget for drainage 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 storm 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 7.1**) summarizes the unit cost used, and it is important to note these costs include sewers, manholes and road restoration.

Table 7.1 – 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
750mm sewer (c/w backfill & asphalt restoration)	m	$1,441
900mm sewer (c/w backfill & asphalt restoration)	m	$1,793
1050mm sewer (c/w backfill & asphalt restoration)	m	$2,068
1200mm sewer (c/w backfill & asphalt restoration)	m	$2,310
1350mm sewer (c/w backfill & asphalt restoration)	m	$2,585
1500mm sewer (c/w backfill & asphalt restoration)	m	$2,860

A summary of the total expenditure for capital improvements per year is provided in **Table 7.2** below.

Table 7.2 Summary of Capital Plan

Year	Approximate Length to be Replaced (m)	Total Cost Estimate (incl. Engineering and Contingency)
2013	1,265	$ 1,493,170
2014	836	$ 1,377,348
2015	808	$ 2,124,608
2016	766	$ 2,125,154
2017	1,064	$ 1,399,903
2018 - 2023	6,640	$ 1,625,999
Total	**11,379**	**$ 20,146,181**

Keeping in mind that The City's allowable annual budget is approximately $1 million a year for drainage related capital improvements, we prioritized the upgrades for each year for first five years. All the remaining upgrades can be completed in the next five years. **Figure 7.1** shows the proposed drainage upgrades based on the year required. A detail breakdown of the proposed capital improvements for each phase is shown in **Table 7.3.** In addition to these upgrades, the City should be completing CCTV inspections for 10% of their storm sewers each year. This equates to approximately 10km of sewers and manholes at an annual cost of $75,000 (for flushing/cleaning and CCTV only).

For some pipe upgrade projects the existing diameters are the same as proposed but the proposed pipes slope have changed to increase pipe capacity.

THE CITY OF WHITE ROCK

Master Drainage Plan Update

Legend

Proposed Upgrade

- 2013 (Very Poor Condition Sewers)
- 2014
- 2015
- 2016
- 2017
- 2018 - 2023

Oxford Pump Station Upgrade

Habgood Pump Station Upgrade

AECOM

0 100 200 400 600 800

Meters

Project No.	Date
60239267	June 2012

Capital Projects

Figure 7.1

Table 7.3 - Details for 10-Year Capital Plan

Year 2013 - Replacement of Storm Sewers in Very Poor Condition

ITEM #	Project Location	Street/Location Detail of Pipes	Model ID	Exist Dia (mm)	Prop Dia (mm	10-Year Peak Flow (L/s)	Length (m)	Unit Rate	Total Cost
1.1	Victoria Ave between Balsam and Centre Street	Victoria Ave b/w 15287 Victoria Ave and Centre St	6552	200	250	24	50	$743	$36,927
		Victoria Ave b/w 15369 Victoria Ave and 15287 Vic	6579	200	250	70	187	$743	$139,201
		Victoria Ave b/w 15423 Victoria Ave and 15369 Vic	6600	200	300	103	113	$798	$90,094
		Victoria Ave b/w Balsam St and 15423 Victoria Ave	5148	300	375	130	96	$886	$85,371
1.2	Marine Drive From Bishop Road to Kerfoot Road	Marine Dr from 14213 Marine Dr to Bishop St	6965	250	250	69	134	$743	$99,398
		Marine Dr from Outfall at 14230 Marine Dr to 1421	6966	250	250	69	32	$743	$24,109
		Marine Dr from 14259 Marine Dr to 14239 Marine	6969	200	250	12	41	$743	$30,413
		Marine Dr from 14283 Marine Dr to 14259 Marine	6971	250	250	30	59	$743	$43,577
		Marine Dr from 14321 Marine Dr to 14283 Marine	6975	375	375	37	64	$886	$56,362
		Marine Dr from 14321 Marine Dr to Kerfoot Rd	6974	450	450	201	34	$963	$32,561
1.3	14479 Marine Drive to High Street	Marine Dr from High St to 14479 Marine Dr	6006	200	250	69	114	$743	$85,001
1.4	Lane b/w Columbia Ave and Royal Ave West of Johnston Rd	Lane b/w Columbia Ave and Royal Ave West of Johnston Rd	6361	200	250	Not in Model	76	$743	$56,222
1.5	Johnston Rd from Thrift Ave to Roper Ave	Johnston Rd from Thrift Ave to Roper Ave	5894	150	250	Not in Model	189	$743	$140,095
1.6	Johnston Rd and Thrift Ave Intersection	Johnston Rd and Thrift Ave Intersection	5692	150	250	Not in Model	37	$743	$27,547
1.7	15367 Buena Vista Ave	15367 Buena Vista Ave	6128	300	525	209	39	$1,122	$44,173
1.8	Coldicut Ravine	Conduct Geotechnical Investigation of the Coldicott Ravine Slide							$40,000
1.9		Annual CCTV Inspection (Approx. 10km of Storm						$7.5	$75,000
						Total	**1265**		**$1,106,052**
						Engineering-10%			**$110,605**
						Contingency -25%			**$276,513**
						Total			**$1,493,170**

Year 2014 - Habgood Diversion plus Replacement of Storm Sewers on Pacific Street and Maple Street

ITEM #	Project Location	Street/Location	Model ID	Exist Dia (mm)	Prop Dia (mm	10-Year Peak Flow (L/s)	Length (m)	Unit Rate	Total Cost
2.1	fic Ave from Maple St to Parke	Pacific Ave from Maple St to 15663 Pacific Ave	6528	250	600	714	42	$1,199	$50,282
		Pacific Ave from 15663 Pacific Ave to Lee St	6530	250	600	707	49	$1,199	$58,755
		Pacific Ave and Lee St Intersection	6531	300	600	591	14	$1,199	$16,909
		Pacific Ave from Lee St to Parker St	6533	200	600	591	82	$1,199	$98,843
		Maple St from 991 Maple St to Intersection of Paci	6550	250	600	754	24	$1,199	$29,348
2.2	Maple St from Columbia Ave to Pacific Ave	Maple St from 949 Maple St to 991 Maple St	5174	200	600	786	92	$1,199	$110,308
		Maple St from 939 Maple St to 949 Maple St	6591	250	600	786	19	$1,199	$22,781
		Maple St from 929 Maple St to 939 Maple St	6596	200	600	798	18	$1,199	$21,598
		Maple St from 928 Maple St to 929 Maple St	6598	250	600	798	11	$1,199	$12,889
		Maple St and Columbia Ave Intersection to 928 Ma	7525	150	600	858	56	$1,199	$67,091
2.3	Habgood St from Pacific Ave to Columbia Ave	Pacific Ave and Habgood St Intersection to 15869 P	5202	375	450	140	63	$963	$60,277
		Proposed Diversion at Pacific Ave and Habgood St I	Divert	NA	450	158	18	$963	$17,334
		Columbia Ave and Habgood St Intersection	7599	375	450	219	8	$963	$8,163
		Habgood St from 905 Habgood St to 15896 Pacific /	6619	200	450	219	176	$963	$168,948
		Columbia Ave and Habgood St Intersection to 905 I	6618	200	450	219	7	$963	$6,434
2.4	Columbia Ave from Parker St to Habgood St	Columbia Ave from Keil St to Habgood St	7598	450	600	336	95	$1,199	$113,485
		Columbia Ave and Parker St Intersection	7493	300	375	99	14	$886	$12,130
		Columbia Ave at Kent St Intersection	7494	600	750	469	14	$1,441	$19,995
		Columbia Ave and Kent St Intersection to 15817 Co	7591	600	750	469	34	$1,441	$49,687
2.5		Annual CCTV Inspection (Approx. 10km of Storm Sewers)						$7.5	$75,000
						Total	**836**		**$1,020,258**
						Engineering-10%			**$102,026**
						Contingency -25%			**$255,064**
						Total			**$1,377,348**

Year 2015 - Parker Street Diversion from Thrift Ave to Pacific Ave

ITEM #	Project Location	Street/Location	Model ID	Exist Dia (mm)	Prop Dia (mm	10-Year Peak Flow (L/s)	Length (m)	Unit Rate	Total Cost
3.1	Parker St from Pacific Ave to Thrift Ave	Parker St and Thrift Ave Intersection	Prop_Pipe1	NA	600	362	13	$1,199	$14,992
		Parker St from 1356 Parker St to Thrift Ave	Link1483	NA	600	450	81	$1,199	$96,651
		Parker St from Roper Ave to 1356 Parker St	Link1484	NA	600	447	116	$1,199	$139,308
		Parker St from 1252 Parker St to Roper Ave	Link1485	NA	600	477	93	$1,199	$112,083
		Parker St from Buena Vista Ave to 1252 Parker St	Link1486	NA	600	477	103	$1,199	$123,160
		Parker St from Cliff Ave to Buena Vista Ave	Link1487	NA	600	526	207	$1,199	$247,688
		Parker St from 15767 Pacific Ave to Cliff Ave	Link1488	NA	600	560	185	$1,199	$221,805
		Pacific Ave and Parker St Intersection to 15767 Pac	Link1489	NA	600	560	11	$1,199	$13,095
		2 Diversion Manholes @ $15,000 each						$15,000	$30,000
3.2		Annual CCTV Inspection (Approx. 10km of Storm Sewers)						$7.5	$75,000
3.3		Oxford Pump Station Upgrade							$500,000
						Total	**808**		**$1,573,784**
						Engineering-10%			**$157,378**
						Contingency -25%			**$393,446**
						Total			**$2,124,608**

Table 7.3 - Details for 10-Year Capital Plan

Year 2016 - Replacement of Storm Sewers on Thrift and George Street									
ITEM #	**Project Location**	**Street/Location**	**Model ID**	**Exist Dia (mm)**	**Prop Dia (mm)**	**10-Year Peak Flow (L/s)**	**Length (m)**	**Unit Rate**	**Total Cost**
4.1	Thrift Ave from George St to 15539 Thrift Ave	Thrift Ave and George St Intersection	5087	300	525	215	11	$1,122	$12,422
		Thrift Ave from 15291 Thrift Ave to George St	5690	300	525	258	44	$1,122	$49,266
		Thrift Ave and Fir St Intersection to 15291 Thrift Av	5697	300	525	291	40	$1,122	$44,700
		Thrift Ave at Thrift Ave and Fir St Intersection	5698	375	525	313	12	$1,122	$13,780
		Thrift Ave from 15317 Thrift Ave to Fir St	5700	375	525	344	38	$1,122	$42,884
		Thrift Ave from 1429 Merklin St to 15317 Thrift Ave	5701	375	525	370	44	$1,122	$49,898
		Thrift Ave from 15403 Thrift Ave to 1429 Merklin S	5703	450	525	444	119	$1,122	$133,448
		Thrift Ave from 15473 Thrift Ave to 15403 Thrift Av	5704	600	750	844	133	$1,441	$191,448
		Thrift Ave from 15539 Thrift Ave to 15473 Thrift Av	5710	600	675	872	138	$1,320	$182,798
4.2	George St from Thrift Ave to Russell Ave	George St from Thrift Ave and Geirge St Intersectio	5608	250	450	182	82	$963	$78,444
		George St from 1455 George St to Russell Ave	5609	375	450	172	104	$963	$100,100
4.3		Annual CCTV Inspection (Approx. 10km of Storm Sewers)						$7.5	$75,000
4.4		Habgood Pump Station Upgrade							$600,000
						Total	**766**		**$1,574,188**
						Engineering-10%			**$157,419**
						Contingency -25%			**$393,547**
						Total			**$2,125,154**

Year 2017 - Replacement of Storm Sewers on Best Street, Russel Ave, Merklin Street and Hospital Street									
ITEM #	**Project Location**	**Street/Location**	**Model ID**	**Exist Dia (mm)**	**Prop Dia (mm)**	**10-Year Peak Flow (L/s)**	**Length (m)**	**Unit Rate**	**Total Cost**
5.1	Russell Ave from Fir St to Best St	Russell Ave from 15383 Russell Ave to Merklin St	5100	300	375	185	46	$886	$40,708
		Russell Ave from Russell Ave and Best St Intersectio	5101	300	375	223	47	$886	$41,704
		Russell Ave and Best St Intersection	5102	375	450	258	13	$963	$12,663
		Russell Ave and Fir St Intersection	5447	300	375	62	12	$886	$10,737
		Russell Ave from Merklin St to Fir St	5452	300	375	112	95	$886	$84,321
5.2	Lane b/w Hospital and Finlay north of Russell Ave.	Lane from 15521 Russell Ave to North Bluff Rd	5386	250	300	67	73	$798	$58,218
		Lane from Russell Ave to 15521 Russell Ave	5460	300	375	82	57	$886	$50,475
5.3	Hospital St from Russell Ave to Vine Ave	Hospital St from Russell Ave to Vine Ave	5369	250	300	109	85	$798	$67,390
5.4	Best St from Thrift to Russell Ave	Best St from Thrift Ave to 1428 Best St	5632	375	525	366	58	$1,122	$65,077
		Best St from 1428 Best St to Russell Ave	5634	375	450	309	139	$963	$133,307
5.5	15456 Russell Ave to Finlay St	Russell Ave from 15570 Russel Ave to 15560 Russel	5451	375	450	125	39	$963	$37,430
		Russell Ave from 15496 Russel Ave to 15456 Russel	5120	300	375	151	94	$886	$82,934
		Russell Ave from 15550 Russel Ave to 15496 Russel	5121	375	450	273	84	$963	$81,096
		Russell Ave from 15560 Russel Ave to 15550 Russel	5123	375	450	273	16	$963	$15,699
		Russell Ave from 15570 Russel Ave to 15560 Russel	5462	375	450	281	37	$963	$35,432
		Russell Ave and Finlay St Intersection to 15570 Rus	5466	375	450	457	56	$963	$54,240
5.6	15334 Merklin St to Russell Ave	Merklin St from Russell Ave to 1530 Merklin St	5391	250	300	68	53	$798	$41,874
		Merklin St from 1530 Merklin St to Multi 8	5392	250	300	62	61	$798	$48,660
5.7		Annual CCTV Inspection (Approx. 10km of Storm Sewers)						$7.5	$75,000
						Total	**1064**		**$1,036,965**
						Engineering-10%			**$103,696**
						Contingency -25%			**$259,241**
						Total			**$1,399,903**

Table 7.3 - Details for 10-Year Capital Plan

ITEM #	Project Location	Street/Location	Model ID	Exist Dia (mm	Prop Dia (mm	10-Year Peak Flow (L/s)	Length (m)	Unit Rate	Total Cost
Years 2018-2023 - Replacement of Remaining Undersized Storm Sewers									
1	Vidal St from Marine Dr to Victoria A	Vidal St from Marine Dr to Victoria Ave	6346	600	600	220	55	$1,199	$65,951
		Vidal St at Vidal St and Marine Dr Intersection	6351	600	600	218	5	$1,199	$5,919
2	Thrift Ave from Vidal St to Martin St	Thrift Ave from Vidal St to Martin St	5666	600	900	1367	101	$1,793	$180,747
		Thrift Ave and Blackwood St Intersection	5672	600	750	1071	7	$1,441	$10,153
		Thrift Ave from Blackwood St to Martin St	5674	600	750	1069	94	$1,441	$135,094
3	Thrift Ave at Stevens St	Thrift Ave at Stevens St	5709	600	750	955	13	$1,441	$19,409
4	Thrift Ave at Johnston Rd	Thrift Ave and Johnston Rd Intersection	5683	250	300	122	13	$798	$10,127
5	Stayte Rd from Buena Vista Ave to R	Stayte Rd from Buena Vista Ave to Roper Ave	5193	375	525	305	191	$1,122	$214,302
6	Royal Ave from Centre St to Cypress St	Royal Ave from 15457 Royal Ave to Dolphin St	5144	600	750	1343	120	$1,441	$173,175
		Royal Ave from Cypres St to 15457 Royal Ave	6488	600	900	1343	7	$1,793	$12,040
		Royal Ave from Dolphin St to Centre St	6389	300	375	84	150	$886	$132,799
7	Roper Ave from Foster St to Winter St	Roper Ave from Foster St to 15133 Roper Ave	5888	250	375	196	44	$886	$38,984
		Roper Ave from 15133 Roper Ave to Winter St	5889	250	375	179	45	$886	$39,853
8	Roper Ave at Merklin St	Roper Ave at Roper Ave and Merklin St Intersectio	5898	150	300	51	14	$798	$10,908
9	Pacific Ave from Johnston Rd to Fir St	Pacific Ave from 1174 Fir St to 15259 Pacific Ave	5128	450	525	431	17	$1,122	$18,983
		Pacific Ave from MULTI 44 to 15201 Pacific Ave	6197	375	525	382	103	$1,122	$115,834
		Pacific Ave from 15259 Pacific Ave to MULTI 44	6232	375	525	418	44	$1,122	$49,116
10	Pacific Ave from Fir St to Dolphin St	Pacific Ave from Dolphin St to Centre St	5130	600	900	1204	136	$1,793	$243,263
		Pacific Ave from 15315 Pacific Ave to 1174 Fir St	6281	600	675	1088	86	$1,320	$113,078
		Pacific Ave from Centre St to 15315 Pacific Ave	6303	600	675	1161	62	$1,320	$81,779
		Pacific Ave and Dolphin St Intersection	6384	600	750	1232	18	$1,441	$25,706
11	Oxford St from Oxenham Ave to Russell Ave	Oxford St from 1454 Oxford St to 1550 Oxford St	5552	300	450	436	54	$963	$51,976
		Oxford St from 1444 Oxford St to 1454 Oxford St	5598	300	450	480	43	$963	$41,080
		Oxford St from 14809 Thrift Ave to 1444 Oxford St	5640	375	450	498	54	$963	$51,803
		Oxford St at 14809 Thrift Ave	5655	375	450	499	19	$963	$18,438
		Oxford St from Thrift Ave to 14809 Thrift Ave	5670	375	450	499	13	$963	$12,514
		Oxford St from 1384 Oxford St to Thrift Ave	5774	375	450	679	60	$963	$57,293
		Oxford St from Oxenham Ave to 1384 Oxford St	5783	375	450	679	12	$963	$11,194
12	Oxford St from Marine Dr to Buena V	Oxford St from 14785 Marine Dr to Buena Vista Av	6206	900	1050	2979	56	$2,068	$116,290
13	Oxford St at Marine Dr	Oxford St at Marine Dr	6224	900	1050	2998	14	$2,068	$28,702
		Oxford St at Marine Dr	6239	750	1050	3007	12	$2,068	$25,054
		Oxford St at Marine Dr	6239_2	750	1050	3007	8	$2,068	$15,671
14	Oxenham Ave at Anderson St	Oxenham Ave at Oxenham and Anderson St Inters	5225	600	675	1016	4	$1,320	$5,569
15	Outfall between Elm St and Vidal St (Below Railway Track)	Outfall between Elm St and Vidal St (Below Railway Track)	6353	750	900	373	23	$6,000	$135,486
16	Outfall between Bay St and Anderson St	Marine Drive at 14661 Marine Dr	5003p	600	900	1327	24	$1,793	$42,286
		Marine Drive at 14661 Marine Dr	6096	600	900	1343	8	$1,793	$13,976
		Outfall at 14661 Marine Dr (Below Railway Track)	6164	600	900	1343	33	$6,000	$195,960
17	Outfall at Oxford St	Outfall at Oxford St	5016p	750	1050	3021	4	$2,068	$7,538
		Outfall at Oxford St	6292_1	900	1050	3021	4	$2,068	$7,439
		Outfall at Oxford St (Below Railway Track)	6292_2	900	1200	3032	38	$7,000	$269,213
18	Outfall at Finlay St (Below Railway Tr	Outfall at Finlay St (Below Railway Track)	7264	900	1200	2034	54	$7,000	$379,197
19	Outfall at Elm St(Below Railway Trac	Outfall at Elm St (Below Railway Track)	6338	375	450	136	49	$4,500	$222,260
20	North Bluff Rd from Lee St to Stayte Rd	North Bluff Rd 1475 Kent St to 15704 North Bluff R	5242	250	300	160	77	$798	$61,159
		North Bluff Rd 1475 Kent St to 15748 North Bluff R	5243	250	300	177	91	$798	$72,299
		North Bluff Rd from 1595 Keil St to 1475 Kent St	5244	250	300	189	61	$798	$48,822
		North Bluff Rd from 15860 North Bluff Rd to 1595	5245P	250	300	192	78	$798	$61,881
		North Bluff Rd from 15904 North Bluff Rd to 15860	5248	300	375	199	86	$886	$76,292
		North Bluff Rd from 1593 Stevens St to 15904 Nort	5249	300	375	203	65	$886	$57,668
		North Bluff Rd from 1587 Stayte Rd to 1593 Stever	5250	300	375	219	98	$886	$86,613
		North Bluff Rd from Stayte Rd to 1587 Stayte Rd	5251	300	375	222	15	$886	$13,670
21	Nichol Rd from North Bluff Rd to Marine Dr	Nichol Rd from 1481 Nichol Rd to Blackburn Ave	6874	750	900	1758	54	$1,793	$95,981
		Nichol Rd from Marine Dr to Magdalen Ave	6928	750	900	1941	55	$1,793	$98,685
		Nichol Rd from Blackburn Ave to Laurel Ave	7004	750	900	1751	61	$1,793	$108,498
		Nichol Rd from 1461 Nichol Rd to 1481 Nichol Rd	7045	750	900	1766	59	$1,793	$105,902
		Nichol Rd from 13987 Marine Dr to 1461 Nichol Rd	7046	750	900	1927	13	$1,793	$23,370
		Nichol Rd from Coldicutt Ave to North Bluff Rd	7053	750	900	1731	100	$1,793	$179,641
		Nichol Rd from Laurel Ave to Coldicutt Ave	7055	750	900	1744	56	$1,793	$100,440
22	Marine Dr from Parker St to Keil St	Marine Dr from Keil St to Kent	6737	375	450	297	85	$963	$82,263
		Marine Dr and Kent St Intersection	6743	300	450	-244	4	$963	$3,850
		Marine Dr and Kent St Intersection to 15777 Marin	6744	300	450	244	100	$963	$95,774
		Marine Dr and Parker St Intersection	7328	300	450	221	6	$963	$5,457
23	Marine Dr from Martin St to Foster St	Marine Dr at Marine Dr and Foster St Intersection	5068	375	900	715	3	$1,793	$5,705
		Marine Dr from Martin St to 15073 Marine Dr	6453	375	525	749	78	$1,122	$87,504
		Marine Dr from 15073 Marine Dr to Foster St	6454	375	525	731	81	$1,122	$91,325
24	Marine Dr from Habgood St to Stevens St	Marine Dr and Stevens St Intersection	5222	250	375	91	13	$886	$11,522
		Marine Dr and Habgood St	7204	300	375	-168	11	$886	$9,321
		Marine Dr from 820 Habgood St to Stevens St	7206	250	375	168	83	$886	$73,248
		Marine Dr and Habgood St	6754	300	375	0	11	$886	$9,601
		Marine Dr and Habgood St	7207	250	375	-168	23	$886	$20,475
25	Marine Dr from Cypress St to Balsam St	Marine Dr from 15445 Marine Dr to 15423 Marine	5149	250	375	171	43	$886	$38,265
		Marine Dr from Balsam St to 15445 Marine Dr	5150P	250	375	188	45	$886	$39,438
		Marine Dr from 15423 Marine Dr to 15415 Marine	6657	250	300	145	44	$798	$35,277
26	Marine Dr from Balsam St to Finlay St	Marine Dr from 15501 Marine Dr to Ash St	5155	200	375	82	62	$886	$54,586
		Marine Dr from 15501 Marine Dr to Balsam St	6703	200	250	11	80	$743	$59,636
		Marine Dr from Ash St to MULTI 7	6731	200	300	51	74	$798	$58,975
		Marine Dr from MULTI 7 to Finlay St	7168	200	250	33	81	$743	$60,340
27	Marine Dr at Oxford St	Marine Dr at Marine Dr and Oxford St	6243	750	1050	3020	20	$2,068	$41,563

Table 7.3 - Details for 10-Year Capital Plan

Years 2018-2023 - Replacement of Remaining Undersized Storm Sewers									
ITEM #	Project Location	Street/Location	Model ID	Exist Dia (mm	Prop Dia (mm	10-Year Peak Flow (L/s)	Length (m)	Unit Rate	Total Cost
28	Marine Dr at Nichol Rd	Marine Dr from 14020 Marine Dr to Nichol Rd	6930	750	900	1966	27	$1,793	$47,916
		Marine Dr at Marine Dr and Nichol Rd Intersection	6931	750	900	1966	20	$1,793	$35,589
29	Marine Dr at Elm St	Marine Dr at Marine Dr and Elm St Intersection	6296	300	375	63	7	$886	$6,064
30	Marine Dr at Balsam St	Outfall at Marine Dr and Balsam St	6690	250	375	181	20	$886	$17,698
31	Marine Dr and Keil St	Marine Dr and Keil St	6752	375	450	437	18	$963	$17,001
32	Lane of 14934 Thrift Ave	Lane of 14934 Thrift Ave	5768	600	675	1493	7	$1,320	$9,169
		Lane of 14733 Thrift Ave	7570	375	450	181	6	$963	$6,083
33	Lane of 1354 Winter St	Lane of 1354 Winter St	5838	200	250	68	30	$743	$22,466
34	Kent St from Thrift Ave to Goggs Ave	Kent St from Thrift Ave to 15805 Thrift Ave	5688	250	375	96	8	$886	$7,084
		Kent St from 15805 Thrift Ave to Goggs Ave	5689	250	375	95	90	$886	$79,253
35	Johnson Rd from Russell Ave to North Bluff Rd	Johnson Rd at 15176 North Bluff Rd	5076p	150	250	43	41	$743	$30,128
		Johnson Rd from Russell Ave to 15176 North Bluff	5436	250	300	71	111	$798	$88,523
36	Johnson Rd at Buena Vista Ave	Johnson Rd from 15201 Pacific Ave to Buena Vista	5078	450	525	326	19	$1,122	$20,806
37	Foster St from Buena Vista Ave to Roper Ave	Foster St from 1270 Foster St to 1280 Foster St	5029	150	375	260	49	$886	$43,391
		Foster St from 15080 Prospect Ave to 1270 Foster	6076	250	375	330	44	$886	$38,962
		Foster St from Buena Vista Ave to 15080 Prospect	6077	250	375	351	58	$886	$51,360
38	Fir Street from Pacific Ave to Buena Vista Ave	Fir Street from lane to 15322 Buena Vista Ave	6169	300	525	611	20	$1,122	$22,411
		Fir Street from 15322 Buena Vista Ave to Fir St	6170	300	525	601	29	$1,122	$32,660
		Fir Street from Pacific Ave to lane of Buena Vista A	6237	300	525	618	46	$1,122	$51,052
39	Dolphin St from Royal Ave to Pacific	Dolphin St from Royal Ave to Pacific Ave	6444	600	750	1237	84	$1,441	$121,622
40	Columbia Ave from Johnston Rd to Fir St	Columbia Ave and Johnston Rd Intersectin to 1520	6403	250	375	261	36	$886	$32,047
		Columbia Ave from 15205 Columbia Ave to Fir St	6420	250	375	243	30	$886	$26,201
		Columbia Ave and Fir St Intersection	6423	250	375	91	3	$886	$3,037
41	Buena Vista Ave from Merklin St to Best St	Buena Vista Ave from Merklin St to 15367 Buena V	5127	250	525	229	24	$1,122	$27,460
		Buena Vista Ave from 15367 Buena Vista Ave to 15	6128	300	525	209	39	$1,122	$44,179
		Buena Vista Ave from 15391 Buena Vista Ave to Be	6129	250	375	177	44	$886	$39,213
42	Buena Vista Ave from Fir St to Merkl	Buena Vista Ave from Fir St to Merklin St	5097p	300	525	411	94	$1,122	$105,789
43	Buena Vista Ave from Everall St to Foster St	Buena Vista Ave from MULTI 33 to Foster St	5017	250	375	423	94	$886	$83,488
		Buena Vista Ave from Martin St to MULTI 33	5019	300	375	458	14	$886	$12,184
		Buena Vista Ave from Everall St to 14941 Buena Vi	5020	375	450	701	96	$963	$92,001
		Buena Vista Ave from 15021 Buena Vista Ave to M	6114	250	375	569	87	$886	$76,735
		Buena Vista Ave from 14941 Buena Vista Ave to 15	6118	300	375	683	123	$886	$108,545
44	Best St from Thrift Ave and Russell A	Best St from 1244 Best St to Kyle Ct	5112	250	300	116	7	$798	$5,583
45	Anderson St and Gordon Ave to Mar	Anderson St from Marine Dr to Gordon Ave	7630	300	375	149	39	$886	$34,775
		Anderson St from Marine Dr and Gordon Ave Intersection to 14707 Gordon Ave	6072	300	375	66	7.851	885.5	$6,952
		Anderson St from Gordon Ave to West Beach Ave	6014	250	300	116	45.67	797.5	$36,422
46	1587 Stayte Rd to North Bluff Rd	Stayte Rd from 1587 Stayte Rd to North Bluff Rd	5180	200	375	222	37	$886	$32,401
47	15505 Marine Dr	Outfall at 15505 Marine Dr	5156	200	450	133	19	$963	$18,673
48	15381 Marine Dr to Cypress St	Marine Dr from 15415 Marine Dr to 15381 Marine	6639	200	250	103	44	$743	$32,839
49	1528 Phoenix St to Vine Ave	Phoenix St from 1528 Phoenix St to Vine Ave	6983	250	375	83	73	$886	$64,642
50	15241 Roper Ave to George St	Roper Ave from 15241 Roper Ave to George St	5883	300	375	90	50	$886	$44,193
51	15226 Royal Ave to Fir St	Royal Ave from 15262 Pacific Ave to 15219 Royal A	6324	200	300	92	47	$798	$37,099
52	15226 Royal Ave to Fir St	Royal Ave from Fir St to 15262 Pacific Ave	6332	200	300	92	11	$798	$9,088
53	15019 Marine Dr to Martin St	Marine Dr from 15019 Marine Dr to Martin St	5070	375	600	1056	30.762	1199	$36,884
54	14985 Victoria Ave to Vidal St& 14881 Marine Dr	Victora Ave from Vidal St to 14985 Victoria Ave	6336	200	250	147	148	$743	$109,562
		Marine Drive from 14899 Marine Dr to Vidal St	5073p	200	600	272	79	$1,199	$94,743
		Marine Dr from 14881 Marine Dr to 14899 Marine	6320	300	600	299	15	$1,199	$18,473
		Marine Drive at 14881 Marine Dr	6325	375	600	354	11	$1,199	$12,595
55	14811 Buena Vista Ave to Oxford St	Buena Vista Ave from Oxford St to 14831 Buena Vi	6116	750	900	2729	61	$1,793	$109,671
59	14767 Gordan Ave to Anderson St	Gordon Ave from 14733 Gordan Ave to 14767 Gor	7357	300	375	67	76	$886	$67,349
		Gordon Ave from Anderson St to 14733 Gordan Av	7359	300	375	66	54	$886	$47,579
60	14760 Oxenham Ave to Oxford St	Oxenham Ave from 14760 Oxenham Ave to Oxford	5792	375	450	709	72	$963	$68,923
61	14711 to 14733 Thrift Ave	Thrift Ave from 14711 Thrift Ave to 14733 Thrift Av	5645	375	450	258	39	$963	$37,508
		Thrift Ave at 14733 Thrift Ave	5646	375	450	246	6	$963	$5,895
62	1456 Johnson Rd to Thrift Ave	Johnston Rd from Thrift Ave to 1456 Johnson Rd	5077	250	300	84	105	$798	$84,034
63	1453 Stayte Rd to Thrift Ave	Stayte Rd from 1453 Stayte Rd to 1471 Stayte Rd	5613	600	750	1064	37	$1,441	$53,647
		Stayte Rd from Thrift Ave to 1453 Stayte Rd	5686	600	750	1075	80	$1,441	$115,404
66	1281 Johnson Rd to Roper Ave	Johnson Rd from 1281 Johnson Rd to Roper Ave	5979	300	375	213	58	$886	$51,359
67	1262 Buena Vista Ave to Johnston Ro	Buena Vista Ave from Johnston Rd to 1273 Fir St	6137	200	250	26	87	$743	$64,575
68	1234 Merklin St	Merklin St at 1234 Merklin St	6068	250	375	138	21	$886	$18,153
69	1111 Cliff Ave to Lee St	Cliff Ave from Lee St to 1111 Lee St	5134	200	375	117	41	$886	$36,510
70		Annual CCTV Inspection (Approx. 50km of Storm Sewers in 5 Years)						$7.5	$375,000
						Total	**6640**		**$8,611,851**
						Engineering-10%			**$861,185**
						Contingency -25%			**$2,152,963**
						Total			**$11,625,999**

Appendix A

Ravine Slope Stability Review (January 23, 2012)

- Thurber Engineering Ltd.

January 23, 2012 File: 19-5438-72

AECOM Canda Limited
4th Floor, 3292 Production Way
Burnaby, BC V5A 4R4

Attention: Mr. Steve Bridger, P.Eng.

WHITE ROCK MASTER DRAINAGE PLAN UPDATE
RAVINE SLOPE STABILITY REVIEW

Dear Steve:

As requested, Thurber Engineering Ltd. (TEL) has completed a reconnaissance of 5 ravines in White Rock. This letter presents the results of the reconnaissance and provides our comments on the condition of the existing slopes and our recommendations for further work.

Use of this report is subject to the attached Statement of Limitations and Conditions.

1. PROGRAM OF WORK

As described in our proposal dated November 1, 2011, we conducted a reconnaissance of the ravines by traversing the creek channels and adjacent trails by foot. The reconnaissance was completed on November 10 and December 8, 2011. As this study is related to the Master Drainage Plan project, our observations focussed on the creek channels and adjacent creek bank slopes although any major instability features observed on the slopes above the channels were also recorded. Access to some reaches of the creek channels was not possible due to thick blackberry vegetation and/or very steep terrain. Photographs of areas of concern were taken and used to assist in the description of particular features that were worthy of note.

We also reviewed our files regarding creek stability issues along Duprez Ravine where we have had significant prior involvement subsequent to the June 1999 flood event. Comparative photographs of previous areas of concern were taken and are presented herein.

2. OBSERVATIONS

2.1 Coldicott Ravine

Coldicott Ravine is the most westerly of the 5 ravines and extends both above (north) and below (south) Marine Drive. We understand that stormwater from the upper reach of the ravine (north of Marine Drive) and from the Marine Drive storm drainage system is intercepted at Marine Drive and conveyed down the lower ravine in a 750mm diameter HDPE pipe, thereby reducing the potential for creek channel erosion in the lower ravine. The geotechnical features of the ravine are markedly different above and below Marine Drive. Hence, the description of the ravine has been presented in two sections below.

2.1.1 Lower Coldicott Ravine

South of Marine Drive, the ravine is deeply incised with steep slopes that are both forest and brush covered. The aforementioned storm pipe discharges into the creek channel via an energy dissipation structure located about 50 m upstream of the 1200 mm diameter culvert below the BNSF railway line. The invert of the culvert is lined with concrete filled sandbags. At the time of the reconnaissance, there was very low water flow in the creek channel above the structure but a steady flow, estimated to be about 2 l/s, was discharging from the dissipation structure.

The lower half (approximately) of the creek channel below Marine Drive was covered by 150 to 1000 mm sized cobbles and boulders and there were areas where rock armouring had been placed in the channel bottom and on the adjacent banks.

At the south (downstream) end of the lower ravine, there were several areas on the east ravine bank where near-vertical, exposed soil slopes were visible at both creek level and on the ravine slopes above (Photo 1). A tree at the top of one of the faces part way up the ravine slope was tilted severely and is expected to topple in the near future. Due to the thick vegetation and very steep terrain, it was not possible to inspect the slopes above creek level. Examination of the near-vertical faces at creek level revealed very dense, fine silty sand. We expect that these faces will continue to slough for many years in the future until they reach a flatter slope, expected to be on the order of 1.2 to 1.4 H:1V at which time, natural re-vegetation can take hold and stabilize the surface of the slope. Fortunately, these faces are located at the south end of the ravine where there are no residences above.

Also on the east ravine slope at about midway between Marine Drive and the BNSF line, there was a landslide that extends from the crest of the slope to the channel. The attached Figure 1 shows the location of the slide and Photo 2 shows the slide as viewed from the base of the ravine. Due to the steep, challenging terrain at the ravine base and no access due to private property ownership at the crest, it was not possible to conduct a detailed assessment of this feature. Notwithstanding, we could see that some large trees had toppled and the root ball had slid down-slope into the channel. There was a Big-O, corrugated plastic pipe on the failed slope surface but it was not possible to determine the pipe alignment or source further up the slope. Review of the attached Figure indicates that the instability was likely a shallow slough triggered by excess soil moisture or possibly dumping of garden refuse at the crest. We recommend that TEL be retained to conduct a detailed assessment of this slide to determine if further slope movement should be expected and whether the presence of the slide presents a risk of damage to the adjacent private properties.

2.1.2 Upper Coldicott Ravine

The slopes and creek channel of Coldicott Ravine above Marine Drive were not as high or steep as below Marine but were also thickly treed. Some of the large coniferous trees had pistol-butted trunks indicating shallow surficial soil movement which is not uncommon on steep ravine slopes. However, there was no evidence of slope instability. The flow in the creek channel was almost imperceptible and there was no creek bank erosion on the channel margins. Dumping of garden refuse was observed along the crest of the ravine at some locations. We suggest that White Rock remind the homeowners along the crests of the ravines that dumping of refuse, including garden waste, is detrimental to the stability of the ravine slopes. TEL would be pleased to assist you with wording of a letter or with preparation of an information pamphlet, should you wish to proceed with this.

2.2 Collingwood Ravine

Collingwood Ravine extends from Malabar Avenue, north of Marine Drive, to the ocean discharge, to the south of Marine. The upper reach of the creek above Marine Drive from the storm pipe outlets (300mm and 900mm diameter concrete pipes) just south of Malabar Avenue to the inlet structure north of Marine Drive is a concrete lined channel. The channel is about 1 to 1.2 m deep and is trapezoidal in section. The slopes above the channel slope moderately. Since the channel is concrete lined, there was no creek bank erosion. Furthermore, the ravine slopes above the channel appeared to have moderate slopes and there was no evidence of slope instability. High flows are conveyed in a 1350mm diameter corrugated steel pipe just east of the concrete lined open channel.

There was no access to the lower reach of Collingwood Ravine, south of Marine Drive and an inspection was not possible. Extensive brush clearing will be required to conduct a visual assessment of this portion of the ravine.

2.3 Duprez Ravine

Duprez Ravine suffered severe channel erosion during major storm events in June 1999. TEL conducted a ravine slope assessment in 2000 and the results were presented in a report dated November 24, 2000. A small slide occurred on the right bank (east side) of the ravine directly east of the inlet structure in February 2002. Further creek damage was caused by another storm event in August 2002. The slide area was repaired in the fall of 2002 and subsequently, the channel slopes have been re-constructed using primarily gabion retaining walls/slopes and rip rap slope protection. Photo 3 shows the current condition of the repaired slope at the inlet.

During the summer of 2004, a storm water bypass was constructed to intercept storm flows from the City of Surrey storm system from discharging into Duprez Ravine. That flow is now conveyed by a 1050 mm diameter HPDE pipe which discharges into an energy dissipater at the downstream (south) end of the ravine. Hence, the base flow in the ravine is typically quite low, as was observed during the reconnaissance.

A significant slope instability feature was observed below 14517 Magdalen Avenue on the right ravine bank in 1999 and is described in detail in the aforementioned report. A slide was also reported at this location by Klohn Leonoff in 1982. Photo 4, taken in 2000, shows the area instability. In our report, it was postulated that the slide occurred as a result of dumping of waste at the crest of the slope, exacerbated by heavy rainfall. Photo 5 shows the current slope condition and it can be seen that vegetation has re-established on the slope. The lot at the crest has since been developed and we assume that the Owner is aware of the hazard posed by dumping refuse over the crest and that this no longer occurs. Since the crest of the slope is on private property, we were unable to confirm the condition of the slope crest.

The results of the recent reconnaissance indicate that the creek channel slopes are relatively stable with little evidence of sloughing or current instability. There are localized areas along the creek bank of low, near-vertical soil faces that will slough with time. However, due to the limited height of these faces, we believe the sloughing will have little to no impact on the overall ravine slopes above.

There is an approximately 1500 mm diameter corrugated steel pipe culvert which serves as a footbridge near the head of the ravine. The gabion wing wall on the upstream, left bank appears to be bulging and the culvert is slightly out of round (Photo 6). Gabion baskets are quite tolerant of deformation and we do not consider failure of the wall to be imminent. However, the condition of the gabion wall should be monitored and if further deformation occurs, it should be replaced. We also recommend that a structural assessment of the culvert be carried out to determine if remedial work is required.

2.4 Anderson Ravine

Anderson Ravine extends from Vine Avenue at the north end and terminates at about Upper Roper Avenue where it enters an inlet structure into the storm water system. The flow in the channel was minimal at the time of the reconnaissance.

The ravine slopes are high and steep in some locations. However, there was no evidence of major instabilities. There was an area where local instability of the creek channel bank, likely resulting from erosion, had caused some trees to topple but this is not a concern for overall ravine bank stability.

2.5 Everall Ravine

Everall Ravine extends from just north of Roper Avenue, west of Blackwood Street to an inlet structure at Prospect Avenue. A significant length of the ravine is located within private property and access to traverse this section was obtained from the Owner. TEL conducted an assessment of the ravine in 2000 in conjunction with the Duprez ravine assessment. There were numerous erosion features along the channel at the base of the ravine (Photo 7) and recommendations were provided for remediation. We understand that the works were implemented at the same time as the Duprez ravine repair.

Storm flow into the ravine from the uplands is delivered by a 900 mm (approximately) diameter corrugated steel pipe. The pipe extends some distance down the ravine and about 6 m of the pipe is exposed on the ravine floor (Photo 8). The pipe discharges into a cobble and boulder lined channel above the fence that delineates the private property (Photo 9). There was nominal flow in the creek channel at the time of the reconnaissance.

Through the private property, the channel walls have been protected by a mix of gabions, cobble and boulder armour and a short section of mortared stone retaining wall. There are some bare soil areas on the creek channel slopes but generally, the channel does not show signs of significant erosion.

Debris has accumulated in the channel against the south property line fence where the creek exits the private property. We recommend that the Owner be notified of the buildup and that the debris be removed before the weight of the debris causes the fence to topple and a potential sudden release of water and debris that could damage the creek channel downstream.

Below the private property, the lower reaches of the channel are lined with angular rip rap rock armour and a gabion wall has been constructed to maintain flow in the channel from impacting the house at the bottom of the ravine (Photo 10).

The overall ravine slopes above the channel from the head of the ravine to the southern end of the private property do not show evidence of instability. However, there is an abandoned house at the east end of Prospect Avenue with a series of timber crib retaining walls supporting site grading fill around the structure. We recommend that the house be demolished and the retaining walls and backfill be removed and the ravine slopes re-vegetated.

3. CLOSURE

This assessment comprised a traverse of 5 ravines in White Rock by foot and, while the majority of the slopes appeared to show no evidence of present or imminent instability, some areas of concern regarding current and future instability of the ravine slopes were noted and recommendations for further work provided. It should be noted that, due to the methodology, vegetation cover and limited access, there may be other areas where unfavourable conditions exist that were not observed during this study.

We believe that fill, composed of dumped grass, brush and/or trash may be present at or near the slope crests throughout the ravine system. Observation can be difficult as it may be masked by vegetation. Landslides can occur where these conditions exist, particularly during or after periods of heavy rain or rain on snow. Hence, the crest of the ravines should be treated with considerable caution by local residents and fill placement or dumping of waste on or near the crest should be forbidden.

We trust that this information is sufficient for your needs. Should you require clarification of any item or additional information, please contact us at your convenience.

Yours truly,
Thurber Engineering Ltd.
David Regehr, P.Eng.
Review Principal

Jan 23/12

David Hill, P.Eng.
Principal

Attachments: Statement of Limitations and Conditions
Photographs

900, 1281 West Georgia Street, Vancouver, BC V6E 3J7 T: 604 684 4384 F: 604 684 5124
thurber.ca

THURBER

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this Report expressly addresses proposed development, design objectives and purposes, and then only to the extent there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation or to consider such representations, information and instructions.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS WE MAY EXPRESSLY APPROVE. The contents of the Report remain our copyright property. The Client may not give, lend or, sell the Report, or otherwise make the Report, or any portion thereof, available to any person without our prior written permission. Any use which a third party makes of the Report, are the sole responsibility of such third parties. Unless expressly permitted by us, no person other than the Client is entitled to rely on this Report. We accept no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without our express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and this report is delivered on the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

(see over . . . .)

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c) Design Services: The Report may form part of the design and construction documents for information purposes even though it may have been issued prior to the final design being completed. We should be retained to review the final design, project plans and documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the report recommendations and the final design detailed in the contract documents should be reported to us immediately so that we can address potential conflicts.

d) Construction Services: During construction we must be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Client's benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

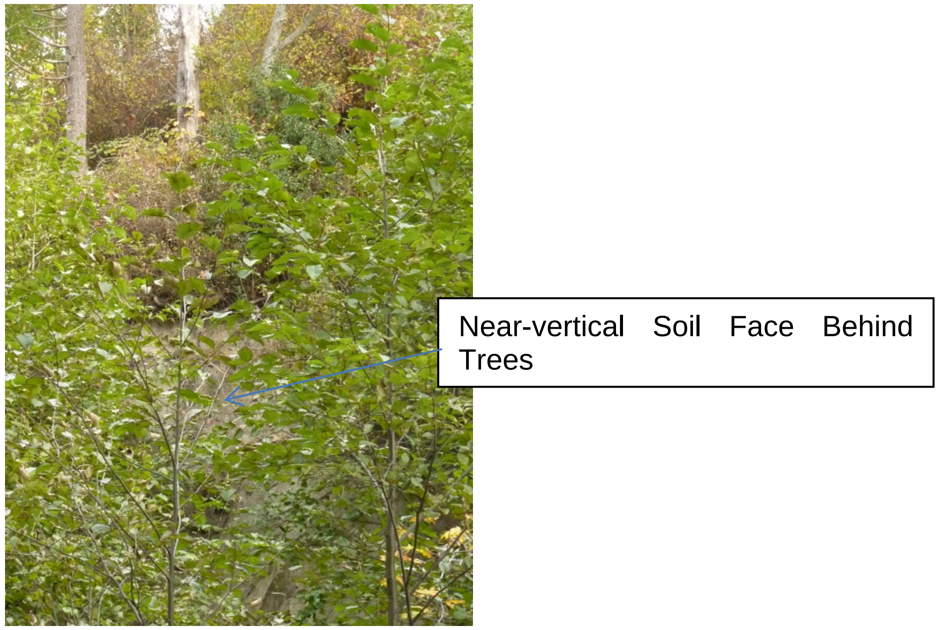

Photo 1 Lower Coldicott Ravine - Near-vertical Soil Face on West Ravine Slope

Photo 2 Lower Coldicott Ravine - Recent Landslide on West Ravine Slope Below Private Properties

Photo 3 Duprez Ravine – Current Condition of Repaired Slide at Inlet Structure

Photo 4 – Duprez Ravine– Landslide at Crest of Slope at 14517 Magdalen Avenue in 2000

Photo 5 – Duprez Ravine – Current Condition of Slope

Photo 6 Duprez Ravine – Gabion Wing Wall at Culvert Footbridge Inlet

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Photo 7 Everall Ravine – Creek Channel Damage in 2002

Photo 8 Everall Ravine – Storm Water Pipe Exposed in Ravine Floor

Photo 9 Everall Ravine – Cobble and Boulder Lined Channel within Private Property

Photo 10 Everall Ravine – Rip Rap Armour and Gabion Wall Lined Channel above Inlet Structure

S:\CAD Files\ACADDATA\PROJECTS\19\VED02837.dwg

NOTES:

AECOM CANADA LTD.

LOCATION OF LANDSLIDE ON COLDICOTT RAVINE

WHITE ROCK RAVINE
STABILITY ASSESSMENT

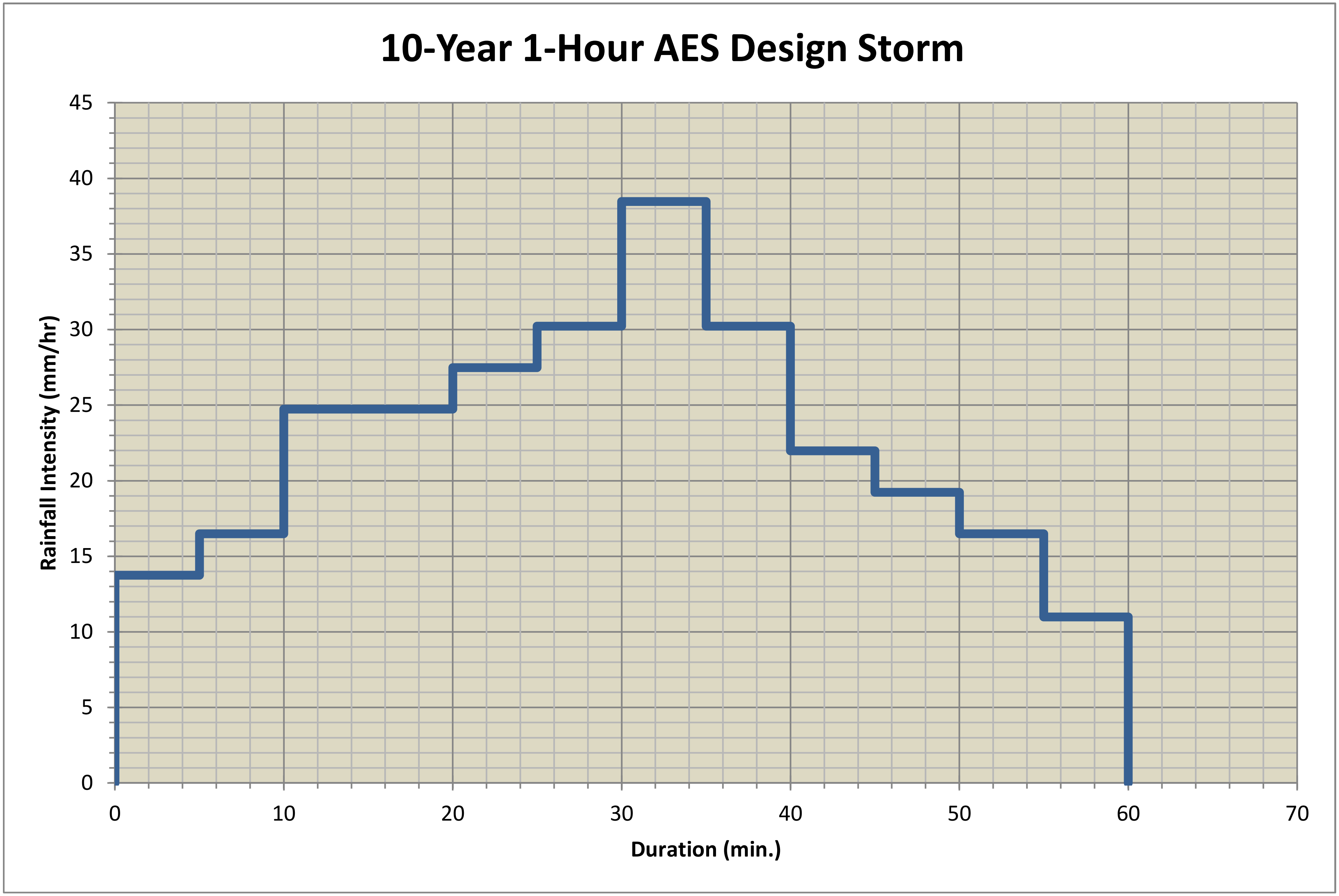
WHITE ROCK, B.C.

THURBER ENGINEERING LTD.

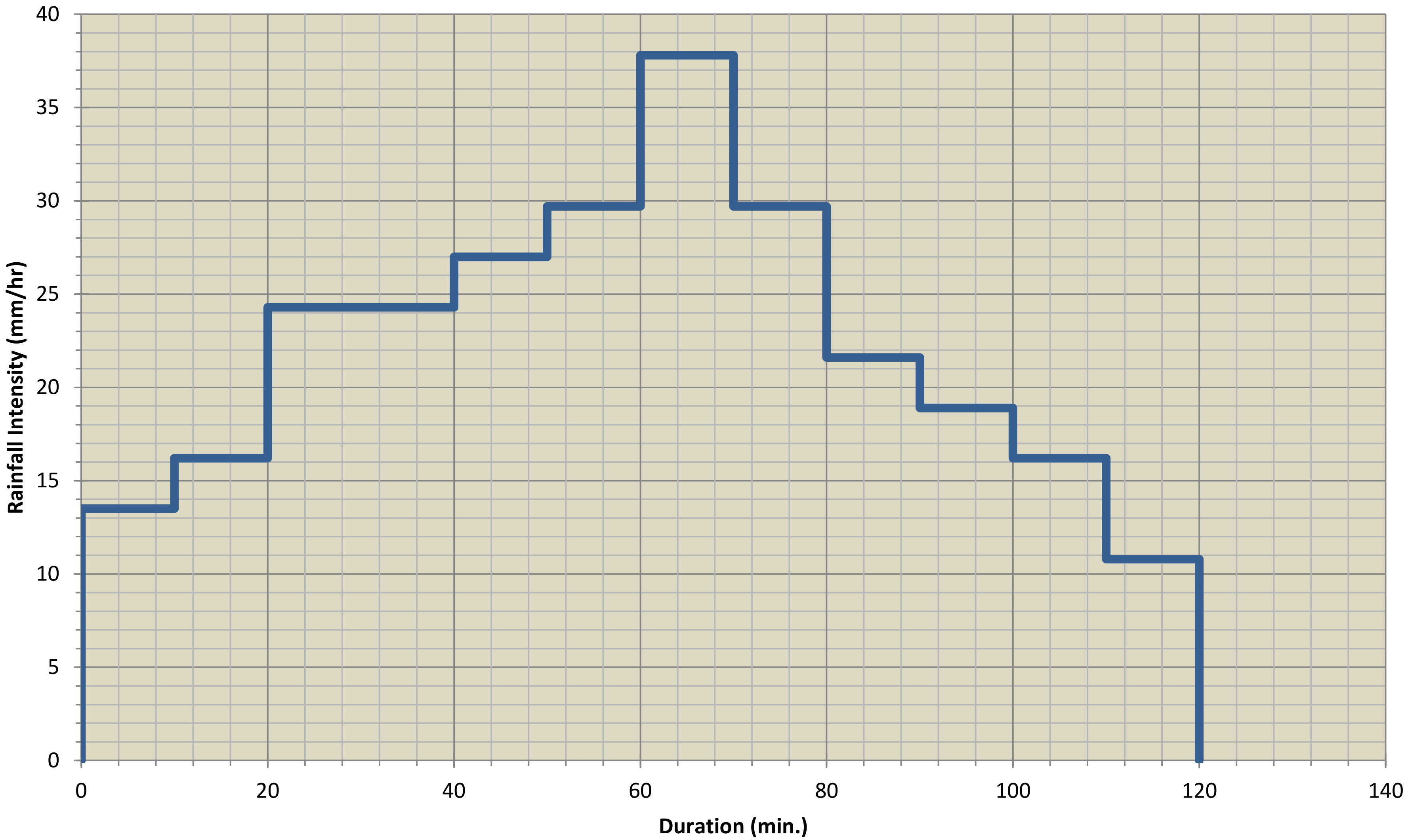
ENGINEER :	DRAWN :	APPROVED :
DH	JL	
DATE : JAN. 19, 2012	SCALE : 1:1000	FIGURE No. 1

Appendix B

- **AES Design Storm Hyetographs**


10-Year 1-Hour AES Design Storm
Rainfall Intensity (mm/hr)
45
40
35
30
25
20
15
10
5
0
0
10
20
30
40
50
60
70
Duration (min.)

100-Year 2-Hour AES Design Storm



Appendix C

- **Model Calibration and Validation Plots**

Figure 3.4 Site 1 Calibration (Finlay St.)

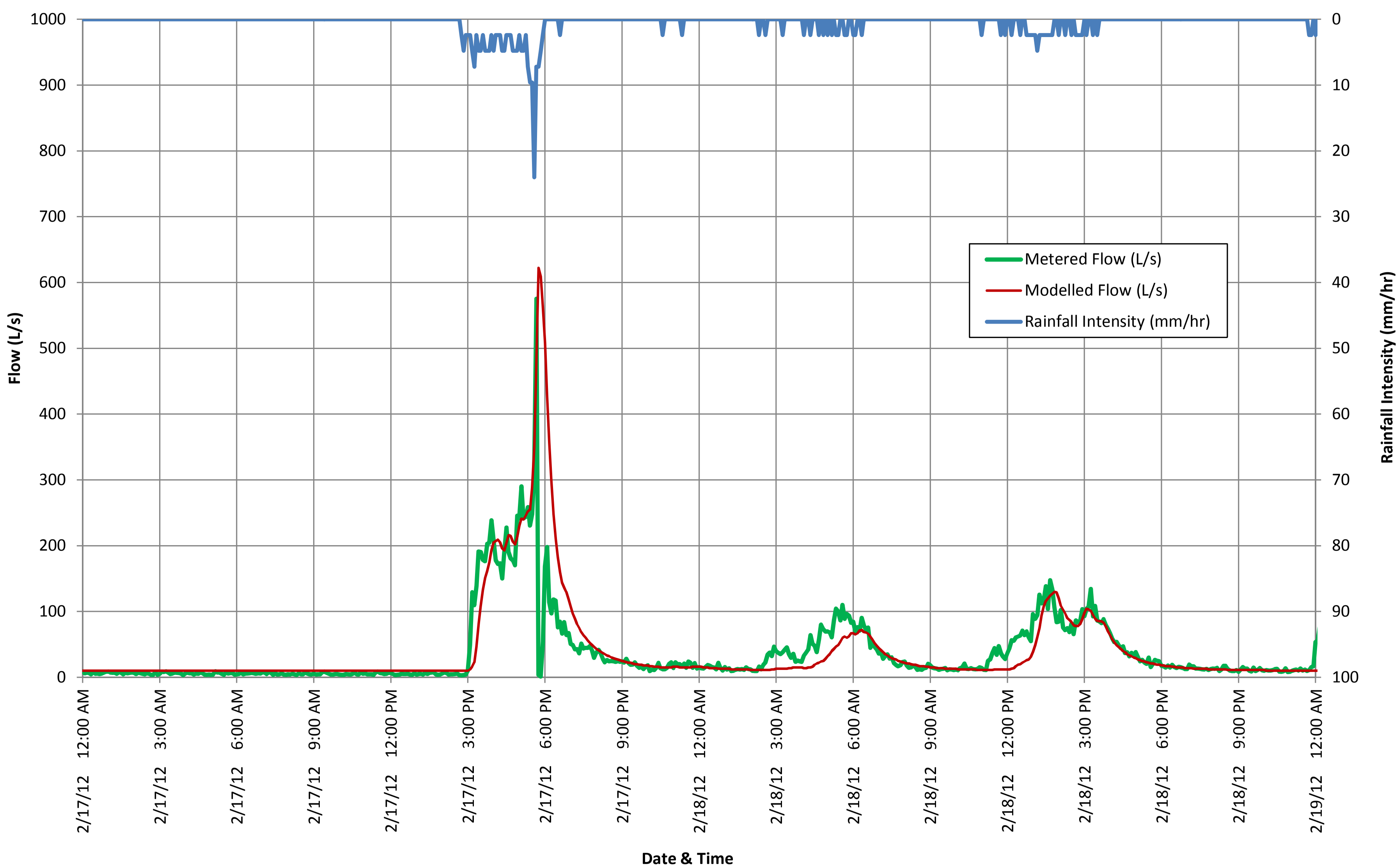


Figure 3.5 Site 3 Calibration (Laurel Ave.)

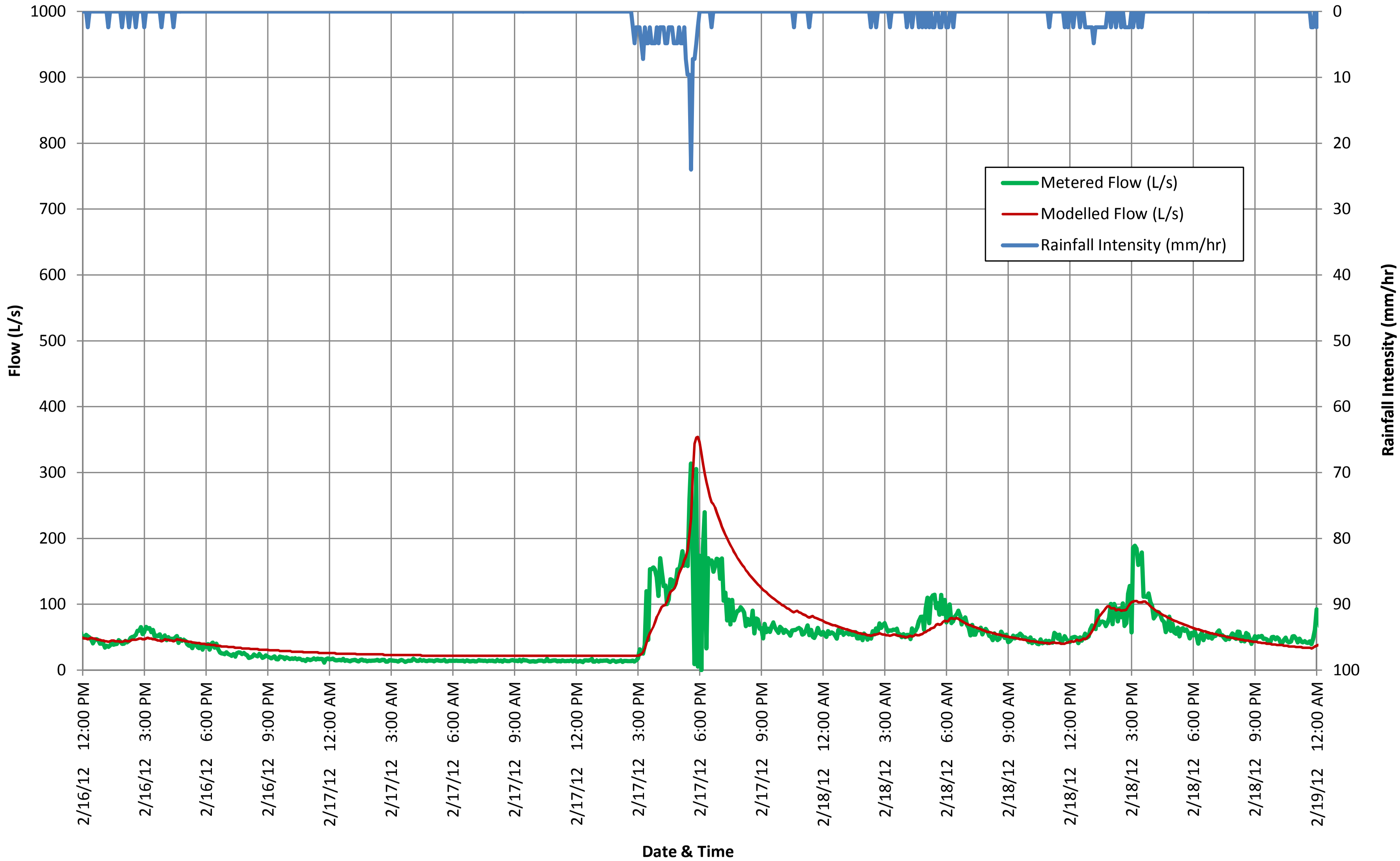


Figure 3.6 Site 4 Calibration (Buena Vista)

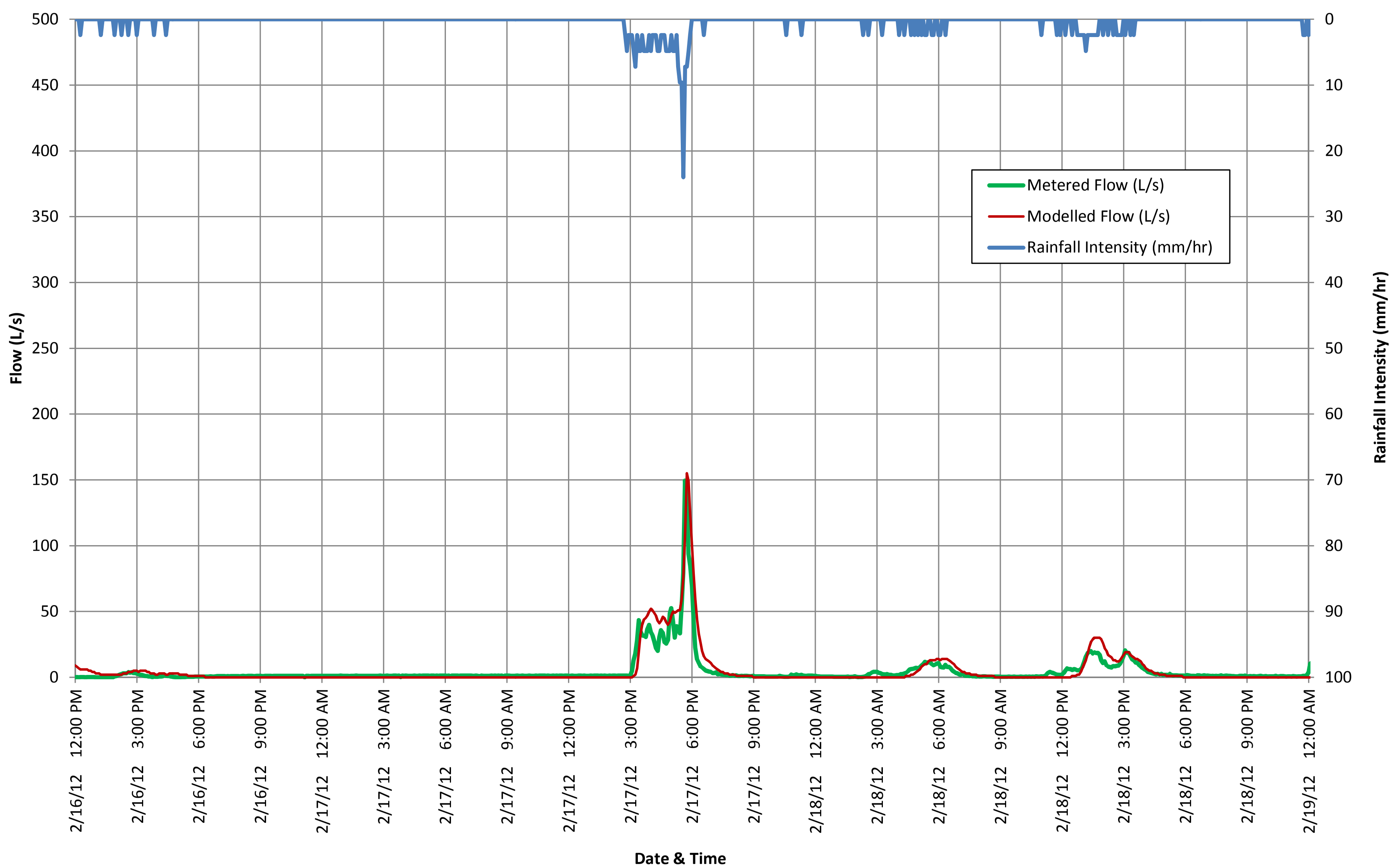


Figure 3.7   Site 1 Validation (Finlay St.)

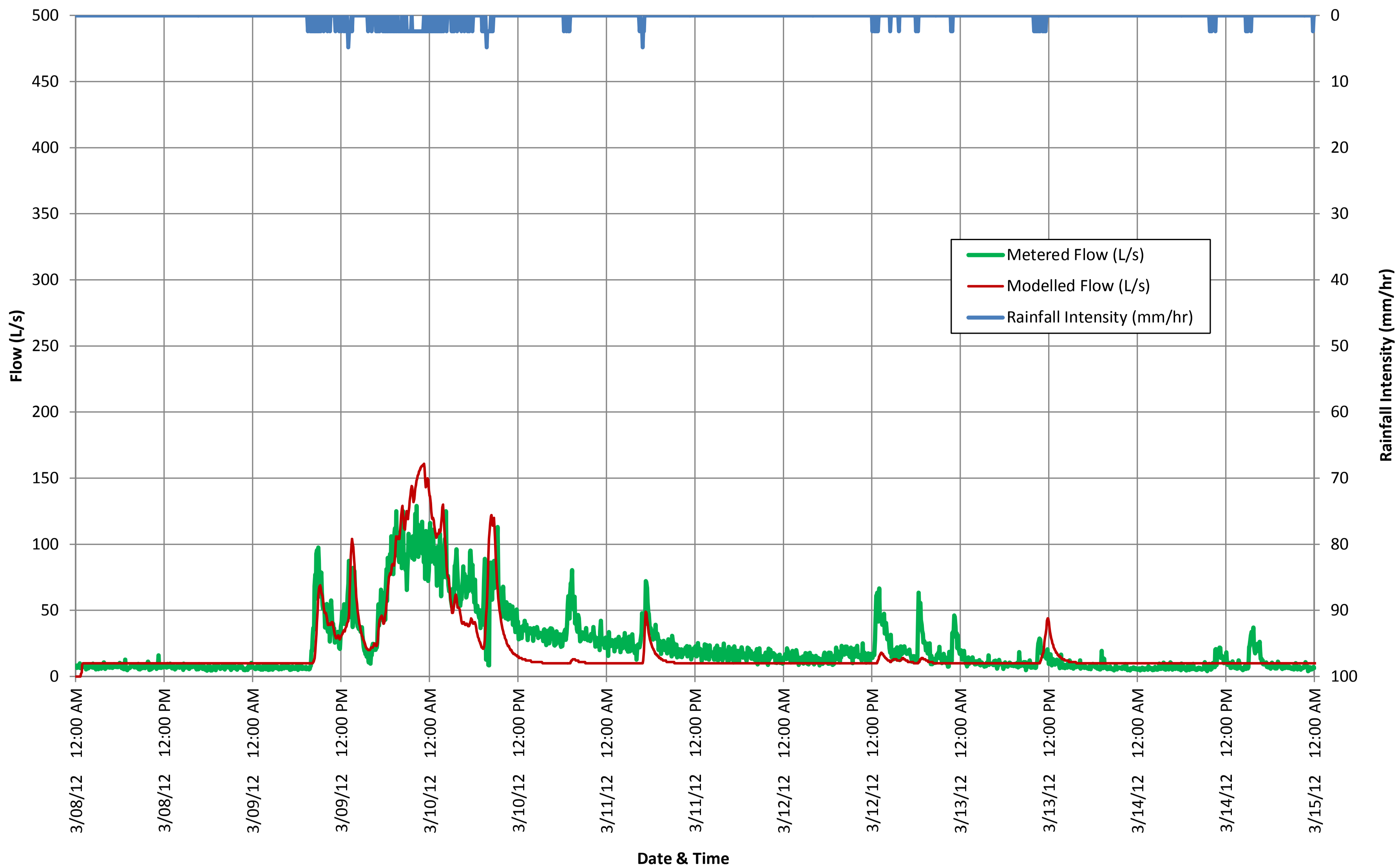


Figure 3.8 Site 3 Validation (Laurel Ave.)

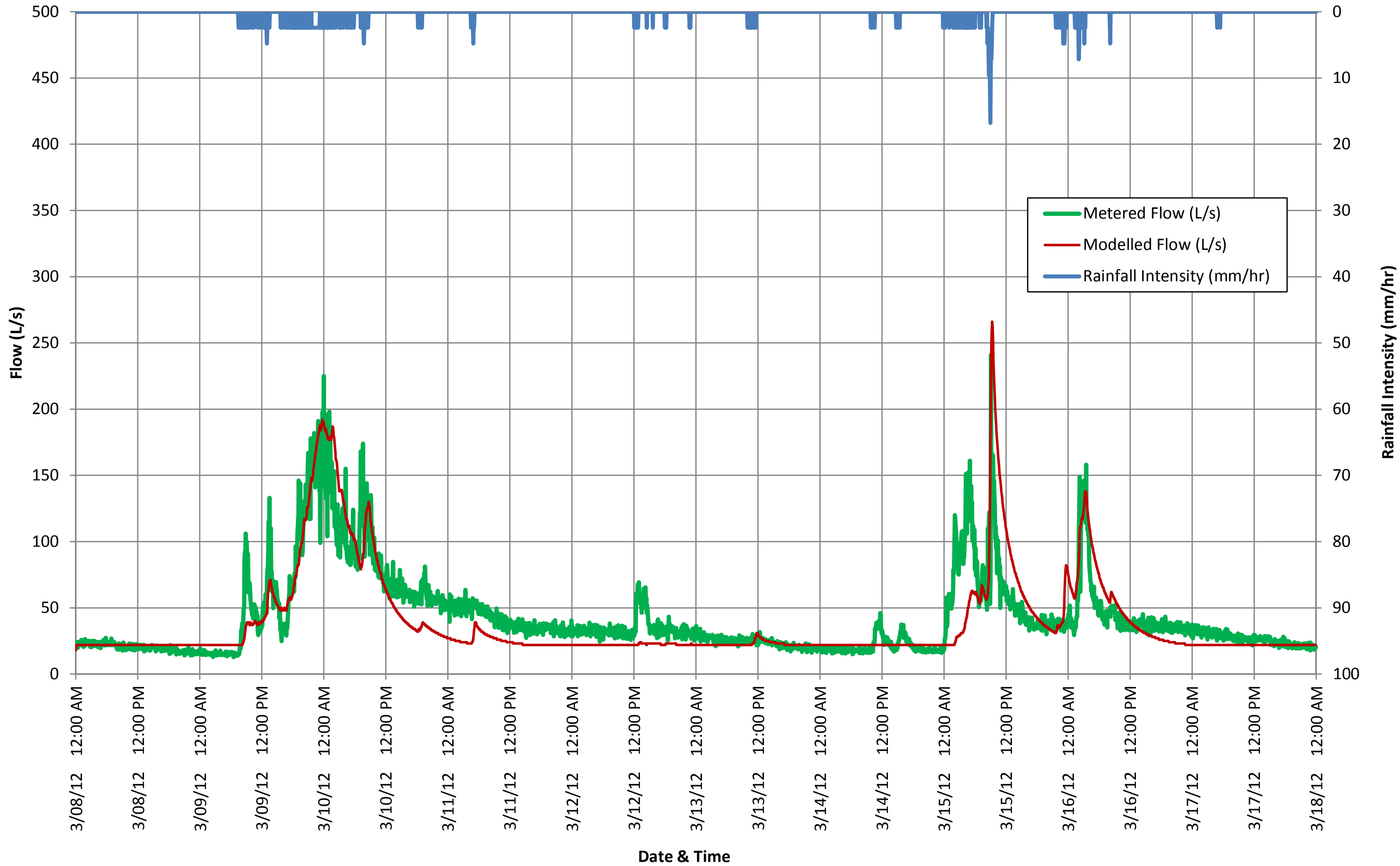
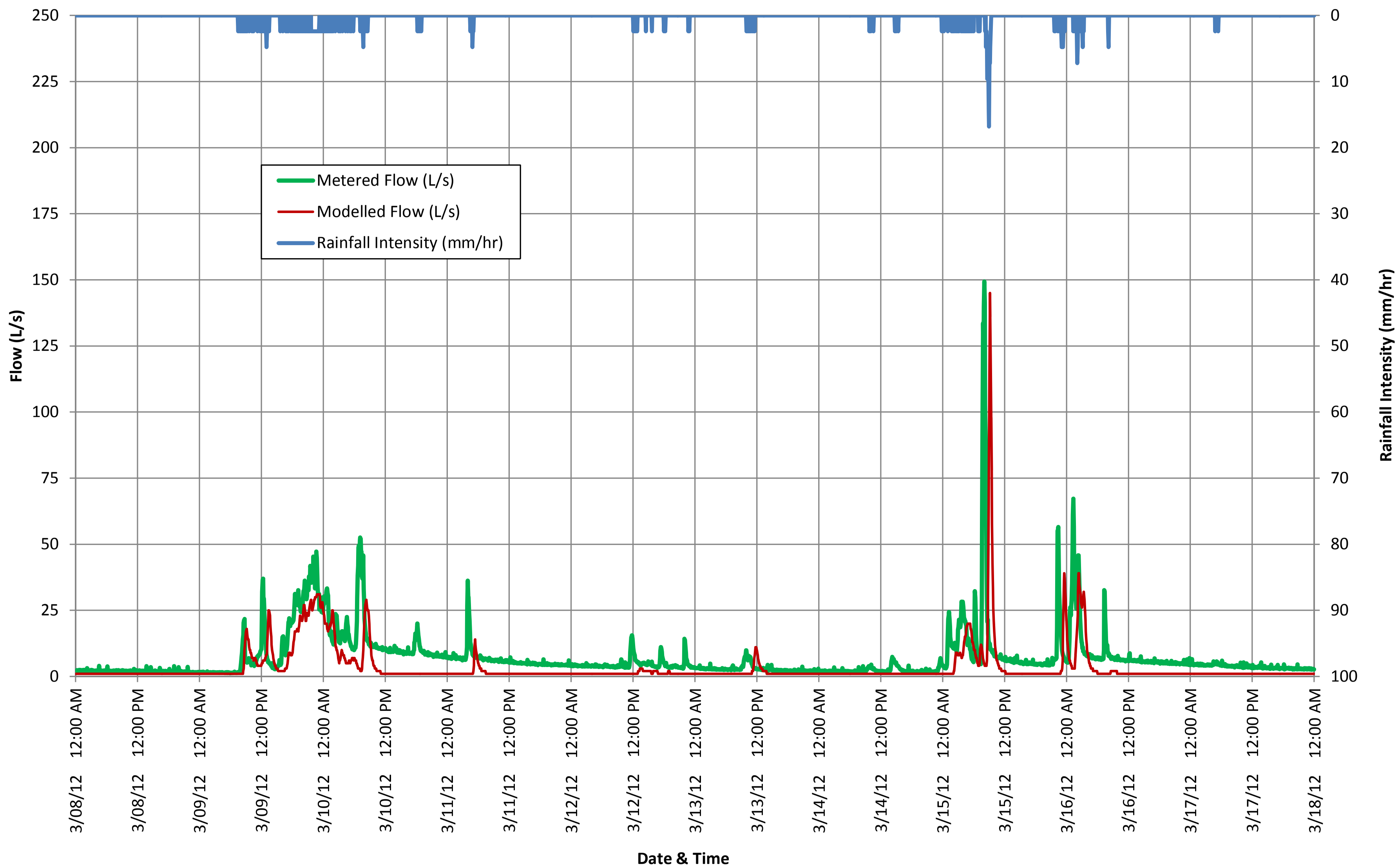


Figure 3.9 Site 4 Validation (Buena Vista)



Appendix B

Ravine Assessment Report (Thurber)

THURBER ENGINEERING LTD.

March 19, 2018 File: 20469

ISL Engengineering and Land Services Ltd.
4190 Lougheed Highway
Burnaby, BC
V5C 6A8

Attention: Angela Steward, P.Eng.

WHITE ROCK MASTER DRAINAGE PLAN 2018 UPDATE
RAVINE SLOPE STABILITY REVIEW

Dear Angela:

As requested, Thurber Engineering Ltd. has completed a reconnaissance of five ravines in White Rock. This letter presents the results of the reconnaissance and provides our comments on the condition of the existing slopes as well as our recommendations for further work.

It is a condition of this letter that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

1. PROGRAM OF WORK

As described in our proposal dated August 25, 2017, we conducted a reconnaissance of the ravines by traversing the creek channels and adjacent trails by foot. The reconnaissance was completed on February 19, and March 1, 2018. As this study is related to the Master Drainage Plan project, our observations focussed on the creek channels and adjacent creek bank slopes although any major instability features observed on the slopes above the channels were also recorded. Access to some reaches of the creek channels was not possible due to thick blackberry vegetation and/or very steep terrain. Photographs of areas of concern were taken and used to assist in the description of particular features that were worthy of note.

We also reviewed our files regarding creek stability issues noted during our January 2012 ravine assessment and along Duprez Ravine where we had significant prior involvement before the June 1999 flood event. Comparative photographs of previous areas of concern were taken and are presented herein.

2. OBSERVATIONS

2.1 Coldicott Ravine

Coldicott Ravine is the most westerly of the five ravines. The upper reach is located between Nichol and Bishop Roads and extends from Blackburn Avenue, to Marine Drive. The lower reach extends from Marine Drive to the BNSF Rail Line at the ocean. We understand that stormwater from both the upper reach of the ravine (north of Marine Drive) and from the Marine Drive storm drainage system is intercepted at Marine Drive and conveyed down the lower ravine in a 750 mm diameter HDPE pipe. As such the potential for creek channel erosion in the lower ravine is reduced. The geotechnical features of the ravine are markedly different above and below Marine Drive. Hence, the description of the ravine has been presented in two sections below.

2.1.1 Upper Coldicott Ravine

The slopes and creek channel of Coldicott Ravine above Marine Drive were about 4 m to 5 m high and sloped roughly at 2H:1V. Both banks were thickly treed. Some of the large coniferous trees had pistol-butted trunks indicating shallow surficial soil movement which is not uncommon on steep ravine slopes. However, we did not observe evidence of slope instability, such as scarps, or tension cracks. The flow in the creek channel was almost imperceptible and we did not oberseve bank erosion on the channel margins. Dumping of garden refuse was observed along the crest of the ravine at some locations. We suggest that White Rock remind the homeowners along the crests of the ravines that dumping of refuse, including garden waste, is detrimental to the stability of the ravine slopes. Thurber can assist you with wording of a letter or with preparation of an information pamphlet, should you wish to proceed with this.

2.1.2 Lower Coldicott Ravine

South of Marine Drive, the ravine is deeply incised with steep slopes that are both forest and brush covered. The 750 mm pipe mentioned previously discharges into the creek channel via an energy dissipation structure located about 50 m upstream of the 1200 mm diameter culvert below the BNSF railway line. The invert of the culvert is lined with concrete filled sandbags. At the time of the reconnaissance, there was very low water flow (about 2 to 5 L/s) in the creek channel above the structure but a steady flow (about 10 L/s) was discharging from the dissipation structure.

The lower half (approximately) of the creek channel below Marine Drive was covered by 150 mm to 1000 mm sized cobbles and boulders and there were areas where rock armouring had been placed in the channel bottom and on the adjacent banks.

At the south (downstream) end of the lower ravine, there were several areas on the left (east) bank of the ravine where near-vertical, exposed soil slopes were visible at both creek level and on the ravine slopes above, as shown in Photo 1. Several trees near the vertical faces of the ravine slope are very close to the exposed slope surface and are expected to topple in the near future, as shown in Photo 2. Due to the thick vegetation and very steep terrain, it was not possible to inspect the slopes above creek level. Examination of the near-vertical faces at creek level revealed very dense, fine silty sand. We expect that these faces will continue to slough for many years in the future until they reach a flatter slope, expected to be on the order of 1.2 to 1.4 H:1V at which time, natural re-vegetation can take hold and stabilize the surface of the slope. Fortunately, these faces are located at the south end of the ravine where there are no residences above.

Our 2012 assessment noted a landslide, extending form the crest of the slope to the channel on the east ravine slope below 14112 Marine Drive. Photo 3 shows the slide location as viewed from the opposite bank of the ravine. Due to the steep, challenging terrain at the ravine base and no access at the crest due to private property ownership, it was not possible to conduct a detailed assessment of this feature. Regardless, we observed that polyethylene sheeting has been used to cover a section of the slope near the crest, which could indicate some instability. We observed a corrugated plastic pipe connected to a PVC pipe on the failed slope surface. We could not locate where the pipe discharged on the slope or where it originated, but it appears to be from the house at the top of the slope. Our 2012 assessment suggested that the instability was likely a shallow slough triggered by excess soil moisture or possibly dumping of garden refuse at the crest. No signs of instability were observed that were not noted in our 2012 assessment. However, we recommend that an experienced geotechnical engineer conduct a detailed assessment of this slide location to assess the likelihood of further slope movement and the risk presented to the adjacent private properties.

2.2 Collingwood Ravine

Collingwood Ravine extends from Malabar Avenue, north of Marine Drive, to an ocean discharge, south of Marine Drive. The upper reach of the creek above Marine Drive from the storm pipe outlets (300 mm and 900 mm diameter concrete pipes) just south of Malabar Avenue to the inlet structure north of Marine Drive is a concrete lined channel. The channel is trapezoidal in section and is about 0.5 m deep, 0.6 m wide at the base and 1.8 m wide at the top. Since the channel is concrete lined, there was no creek bank erosion. Furthermore, the ravine slopes above the channel appeared to have moderate slopes and there was no evidence of slope instability. Some dumping of garden waste was present near the top of the ravine crest at some locations.

High flows are conveyed in a 1350 mm diameter corrugated steel pipe (CSP) just east of the concrete lined open channel. Accordingly, at the time of the inspection, the creek had very little flow near the storm pipe outlets at Malabar Avenue.

A large block of wood was lying across the creek near the end of Wheatley Avenue for use as a foot bridge, and was collecting debris, as shown in Photo 4. We recommend that the block be removed to reduce the likelihood of debris build-up

The flows appear to be conveyed under Marine Drive and to about 25 m up-slope of the BNSF rail line. The creekbanks were partially obscured by blackberries as shown in Photo 5. The trees on the bank near the culvert outlet showed signs of tilting and pistol butting. The banks in this area will need to be cleared to properly assess the stability of bank slopes.

2.3 Duprez Ravine

Duprez Ravine suffered severe channel erosion during major storm events in June 1999. Thurber conducted a ravine slope assessment in 2000 and the results were presented in a report dated November 24, 2000. A small slide occurred on the right (west) bank of the ravine directly east of the inlet structure in February 2002. Further creek damage was caused by another storm event in August 2002. The slide area was repaired in the fall of 2002 and subsequently, the channel slopes have been re-constructed using primarily gabion retaining walls/slopes and rip rap slope protection. Photo 6 shows the current condition of the repaired slope at the inlet.

During the summer of 2004, a storm water bypass was constructed to intercept storm flows from the City of Surrey storm system from discharging into Duprez Ravine. That flow is now conveyed by a 1050 mm diameter HPDE pipe which discharges into an energy dissipater at the downstream (south) end of the ravine. Hence, the base flow in the ravine is typically quite low, as was observed during the reconnaissance.

Below 14541 Magdalen Avenue, on the right (west) bank, we observed a sinkhole behind the gabion wall, which retains the walking path below this property as shown in Photo 7. We recommend that this sinkhole be repaired, to reduce the likelihood of damage to the walking path.

A significant slope instability feature was observed below 14517 Magdalen Avenue on the right ravine bank in 1999 and is described in detail in the aforementioned report. A slide was also reported at this location by Klohn Leonoff in 1982. Photo 8, taken in 2000, shows the area instability. In our report, we found it most likely that the slide occurred as a result of dumping of waste at the crest of the slope, exacerbated by heavy rainfall. Photo 9 shows the current slope condition and it can be seen that vegetation has re-established on the slope. The lot at the crest has since been developed and we assume that the Owner is aware of the hazard posed by dumping refuse over the crest and that this no longer occurs. Since the crest of the slope is on private property, we were unable to confirm the condition of the slope crest.

On the right (west) bank east of the corner of High Street and Blackburn Crescent, a localized, 5 m wide area of surface erosion extended from the creek level to the top of the ravine, as shown in Photo 10. The feature was bare of vegetation and sloped at about 35°. Near the top of the channel, 2.5 m to 3 m high, sub-vertical scarps were observed. No tension cracks were observed at the crest, but utility poles are present. The material at the base of the channel appeared to be hard/dense fine sand and silt. an HDPE drainage pipe was partially exposed throughout the length of the feature and no damage was observed where it was exposed. At the toe of the channel, a gabion wall is present, which does not show signs of distress. Surficial erosion of the feature will likely continue if not mitigated. Thurber can provide further recommendations for stabilizing this slope if requested.

Noticeable tilting, pistol-butting, and toppling of trees was observed throughout the ravine; however, no visible signs of recent instability were observed. Given the relatively low creek flow and gabion armouring present at many locations, we believe that the creek channel slopes are relatively stable, in general. There are localized areas along the creek bank of low, near-vertical soil faces that will slough with time. However, due to the limited height of these faces, we believe the sloughing will have little to no impact on the overall ravine slopes above.

There is an approximately 1500 mm diameter CSP culvert which serves as a footbridge near the head of the ravine. The gabion wing wall on the upstream, left bank appears to be bulging and the culvert is slightly out of round, as shown in Photo 11. Gabion baskets are quite tolerant of deformation and we do not consider failure of the wall to be imminent. However, the condition of the gabion wall should be monitored and if further deformation occurs, it should be replaced. We also recommend that a structural assessment of the culvert be carried out to determine if remedial work is required.

2.4 Anderson Ravine

Anderson Ravine extends from Vine Avenue at the north end and terminates at about Upper Roper Avenue where it enters an inlet structure into the storm water system. The flow in the channel was minimal at the time of the reconnaissance.

The ravine slopes are high and, in some locations, relatively steep, as shown on Photo 12. However, there was no evidence of major instabilities.

2.5 Everall Ravine

Everall Ravine has two branches, the west and east. The west branch extends from the south end of 1351 Vidal Street to about Everall Street at Prospect Avenue. The majority of the west branch is within private property, so it was accessed from the west side along the Roper Avenue right of way. A 300 mm diameter open-top CSP pipe exits the slope near Everall Lane and extends partway down the slope, as shown in Photo 13. An erosion feature is present below the outlet of the CSP pipe to the bottom of the ravine as shown in Photos 14 and 15, and Photo 16 shows the condition at the base of ravine in 2002. Flows in the ravine were minimal at the time of inspection. We recommend that the CSP pipe be extended to bottom of the ravine to reduce bank erosion.

The east branch extends from just north of Roper Avenue, west of Blackwood Street to an inlet structure at Prospect Avenue. A significant portion of the ravine is located within private property and access to this section was obtained from the Owner.

Storm flow into the east branch from the uplands is delivered by an approximately 900 mm diameter CSP. The pipe extends some distance down the ravine and about 6 m of the pipe is exposed on the ravine floor. The pipe discharges into a cobble and boulder lined channel inside of the fence that delineates the private property, as shown in Photo 17. There was nominal flow in the creek channel at the time of the reconnaissance.

Through the private property, the channel walls have been protected by a mix of gabions, cobble and boulder armour and a short section of mortared stone retaining wall. There are some bare soil areas on the creek channel slopes but generally, the channel does not show signs of significant erosion, as shown in shown in Photo 18.

Debris has accumulated in the channel against the south property line fence, which has a noticeable bulge in it where the creek exits the private property. It appears that the fence has been reinforced with rebar, as shown in Photo 19. We recommend that the Owner be notified of the buildup and that the debris be removed before the weight of the debris causes the fence to topple, which could cause a sudden release of water and debris that could damage the creek channel downstream.

Below the private property, the lower reaches of the channel are lined with angular rip rap rock armour and a gabion wall has been constructed to constrain flow in the channel near the house at the bottom of the ravine, as shown in Photo 20.

3. CLOSURE

This assessment comprised a traverse of five ravines in White Rock by foot. While the majority of the slopes appeared to show no evidence of present or imminent instability, some areas of concern regarding current and future instability of the ravine slopes were noted and recommendations for further work provided. Please note that, due to the methodology, vegetation cover and limited access, there may be other areas where unfavourable conditions exist that were not observed during this study.

We believe that fill comprising dumped grass, brush and/or trash may be present at or near the slope crests throughout the ravine system. Observation can be difficult as it may be masked by vegetation. Landslides can occur where these conditions exist, particularly during or after periods of heavy rain or rain on snow. Hence, the crest of the ravines should be treated with considerable caution by local residents and fill placement or dumping of waste on or near the crest should be forbidden.

We trust that this information is sufficient for your needs. Should you require clarification of any item or additional information, please contact us at your convenience.

Yours truly,

Thurber Engineering Ltd.
David Regehr, P.Eng.
Review Principal

Erik Stevenson, P.Eng.
Project Engineer

PROFESSIONAL
PROVINCE OF
E. P. STEVENSON
45439
BRITISH COLUMBIA
ENGINEER
March 19, 2018

Attachments:	Statement of Limitations and Conditions	(1 Page)
	Figure 1: Area Plan	(1 Page)
	Photographs	(11 Pages)

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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

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The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

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LEGEND:

Ravine Channel

6 Photograph Location

NOTES:

THURBER ENGINEERING LTD.

CLIENT: ISL ENGINEERING AND LAND SERVICES LTD.

WHITE ROCK RAVINES KEY PLAN

WHITE ROCK MASTER DRAINAGE PLAN 2018 UPDATE

WHITE ROCK, BC

DESIGNED:	DRAWN:	APPROVED:
EPS	EPS	DNR

DATE:	SCALE:	
March 19, 2018	NTS	
PROJECT No.:	FIGURE No.:	REV:
20469	1	0

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Photo 1 Lower Coldicott Ravine - Near-vertical Soil Face on East Ravine Slope

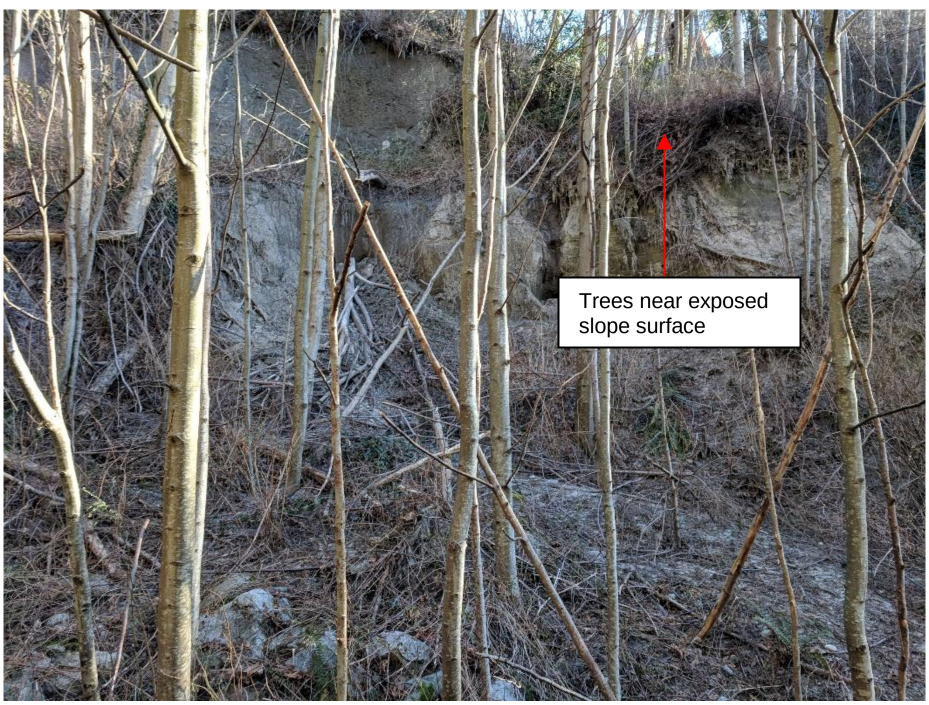

Photo 2 Lower Coldicott Ravine – Trees that may Topple

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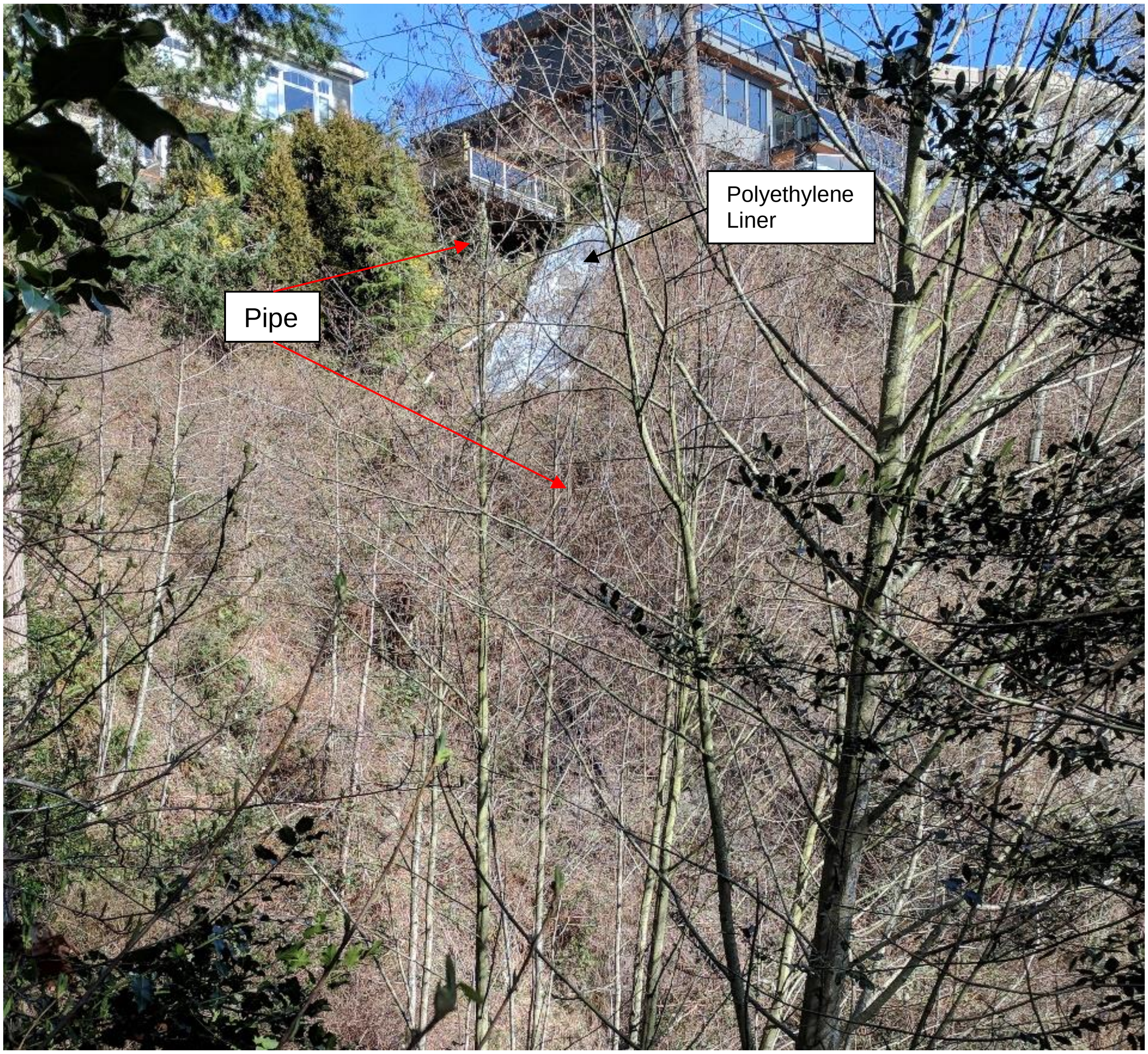

Photo 3 Lower Coldicott Ravine – Drainage Pipe and Poly Liner

Photo 4 Collingwood Ravine – Foot Bridge Collecting Debris

Photo 5 Collingwood Ravine – Outlet Slopes Covered with Blackberries

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Photo 6 Duprez Ravine – Current Condition of Repaired Slide at Inlet Structure

Photo 7 Duprez Ravine – Sinkhole Behind Gabion Wall at base of 14517

Photo 8 Duprez Ravine – Landslide at Crest of Slope at 14517 Magdalen Avenue in 2000

Photo 9 Duprez Ravine – Current Condition of Slope at 14517 Magdalen Avenue

Photo 10 Duprez Ravine – Steep Right Bank East of Blackburn Cr./High St.


Photo 11 Duprez Ravine – Gabion Wing Wall at Culvert Footbridge Inlet


Photo 12 Anderson Ravine – Steep Slope at End of Oxenham Avenue

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Photo 13 Everall Ravine West Branch – CSP Pipe near Crest

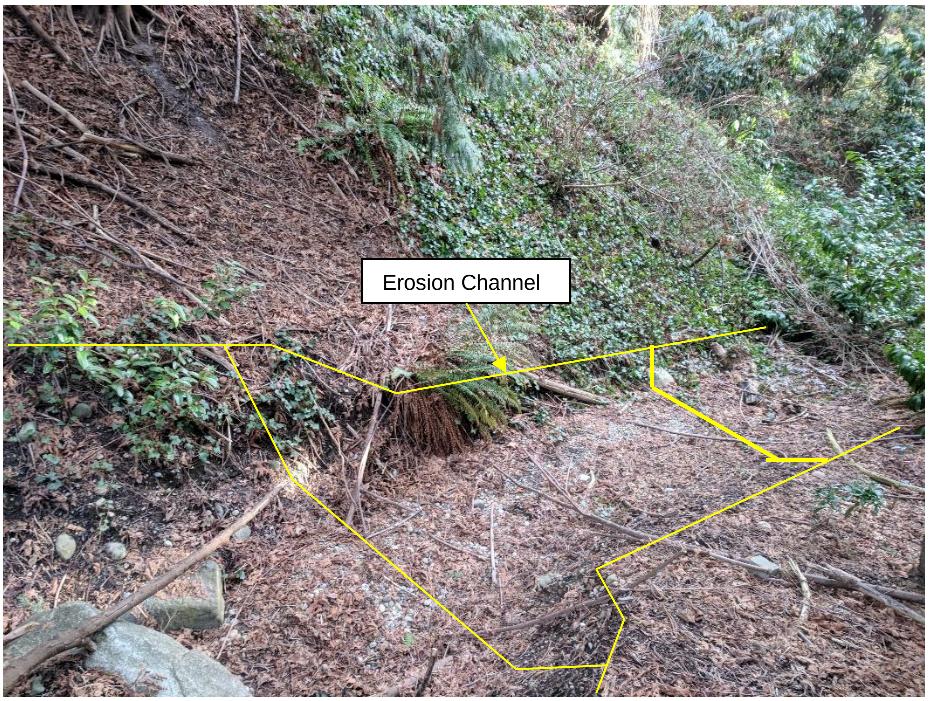

Photo 14 Everall Ravine West Brach – Erosion Channel below Pipe
(Looking downslope, 3 m below end of CSP)

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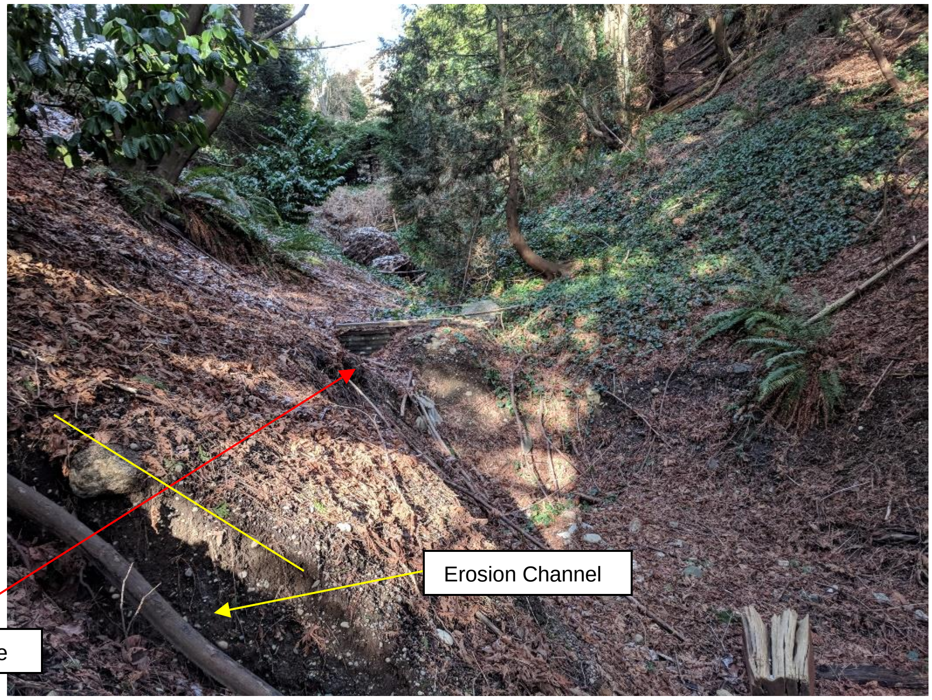

Photo 15 Everall Ravine West Branch – Erosion at bottom of Ravine

Photo 16 Everall Ravine – Creek Channel Damage in 2002

Photo 17 Everall Ravine – Storm Water Pipe Exposed in Ravine Floor

Photo 18 Everall Ravine – East Slope of Creek

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Photo 19 Everall Ravine – Rebar Reinforcement for Fence


Photo 20 Everall Ravine – Gabion and Rip Rap Armour Lined Channel above Inlet Structure

Appendix C
2018 CCTV Programs Pipe Condition Summary

FIGURE C1

Legend

Study Area

Condition / Maintenance Rating

- 3
- 4
- 5
- 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!

Drainage Master Plan Update 2019

2018 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

Appendix D
Capital Plan

Item	Location	Model ID	Exist. Diam (mm)	Proposed Diam (mm)	10-Year Peak Flow (L/s)	Length (m)	Unit Cost	Total Cost
Year 2018 (Mostly Condition Upgrades)								
0.1	George St, north of Russell Ave	5373	300	300	35	76.5	$ 500	$ 67,167
0.2	Fir St, north of Russell Ave	7089	250	250	28	94.0	$ 817	$ 76,798
0.3	Fir St, at Russell Ave	5085	250	300	16	12.0	$ 878	$ 10,536
0.4	Finlay St, north of Russell Ave	5306	250	250	61	123.0	$ 817	$ 100,499
0.5	Stevens St, south of Russell Ave	5582	250	250	8	86.5	$ 817	$ 70,671
0.6	Kent St, north of Goggs Ave	5593	250	TPR	6	35.0		$ 500
0.7	Kent St, north of Thrift Ave	5689	250	TPR	20	89.5		$ 500
0.8	Merklin St, north of Roper Ave	5090	200	TPR	28	122.5		$ 500
0.9	Intersection of Roper Ave & Merklin St	5898	150	250	32	13.7	$ 817	$ 11,177
0.10	Cliff Ave, west of Balsam Ave	5131	200	250	78	90.6	$ 817	$ 74,012
0.11	Dolphin St, south of Semiahmoo Ave	6329	200	200	44	44.5	$ 770	$ 34,288
0.12	Pacific Ave, at Balsam St	6476	375	375	121	18.3	$ 975	$ 17,843
0.13	Cliff Ave, east of Finlay St	7224	250	250	78	125.8	$ 817	$ 102,738
0.14	Cliff Ave, west of Lee St	5134	200	375	132	41.2	$ 975	$ 40,199
0.15	Lane south of Royal St, west of Balsam St	6559	200	200	19	44.9	$ 770	$ 34,588
0.16	Pacific Ave, Stevens St to Stayte Rd	5205	450	EPR	150	92.7		$ 500
0.17	Habgood St, south of Pacific Ave	6619	450	EPR	408	56.0		$ 500
0.18	Kent St, south of Russell Ave	7099	200	200	32	82.5	$ 770	$ 63,525
0.19	Stevens St, north of Russell Ave	7102	200	200	14	86.0	$ 770	$ 66,220
0.20	Lee St, north of Roper Ave	5169.1	200	200	7	163.0	$ 770	$ 125,518
0.21	Roper Ave, west of Parker St	5913	200	200	13	39.2	$ 770	$ 30,153
0.22	Marine Dr, east of Stevens St	7205	250	250	43	67.4	$ 817	$ 55,033
0.23	Thrift Ave, west of Stayte Rd	5733	300	525	725	47.4	$ 1,234	$ 58,504
0.24	Habgood PS Relocation/Construction[1]							$ 5,405,000
0.25	Oxford PS Upgrade[2]							$ 550,000
							Sub-total	**$ 6,996,968**
							Engineering - 10%	**$ 699,697**
							Contingency - 25%	**$ 1,749,242**
							Total	**$ 9,445,907**

[1]Cost from prelim design report, with $0.5M allowance for additional pump station capacity and $0.85M removed from outfall (removed cost included as Item 2.22 in table)

[2]Cost from 2012 capital plan, with 10% inflation

Item	Location	Model ID	Exist. Diam (mm)	Proposed Diam (mm)	10-Year Peak Flow (L/s)	Length (m)	Unit Cost	Total Cost
Capacity Upgrades (2019)								
1.1	Roper Ave, Foster St to south	Roper-Diversion		375	246	55.5	$ 975	$ 54,152
1.2	Roper Ave, Winter St to Foster St	5062p	250	375	122	11.0	$ 975	$ 10,754
1.3		5889	250	375	183	45.0	$ 975	$ 43,885
1.4		5888	250	375	198	44.0	$ 975	$ 42,929
1.5	Foster St, south of Roper Ave to Beuna Vista Ave	5029	150	375	311	49.0	$ 975	$ 47,775
1.6		6076	250	450	387	44.0	$ 1,059	$ 46,596
1.7		6077	250	375	410	58.0	$ 975	$ 56,550
1.8	Buena Vista Ave, from Foster St to Everall St	5017	250	375	492	94.3	$ 975	$ 91,923
1.9		5019	300	375	528	13.8	$ 975	$ 13,416
1.10		6114	250	375	645	86.7	$ 975	$ 84,494
1.11		6118	300	450	759	122.6	$ 1,059	$ 129,812
1.12		5020	375	450	776	95.6	$ 1,059	$ 101,230

1.13	Buena Vista Ave, East of Oxford St	6116	750	900	2538	61.2	$ 1,972	$ 120,627
1.14	Oxford St, north of Buena Vista Ave to south of Marine Drive	6100	150	375	295	14.9	$ 975	$ 14,537
1.15		6206	900	1050	2820	56.2	$ 2,275	$ 127,923
1.16		6224	800	1050	2837	13.9	$ 2,275	$ 31,577
1.17		6239	800	1050	2842	8.0	$ 2,275	$ 18,280
1.18		6239_3	800	1050	2845	4.1	$ 2,275	$ 9,298
1.19		6239_2	800	1050	2845	7.6	$ 2,275	$ 17,245
1.20		6243	800	1050	2858	20.1	$ 2,275	$ 45,728
1.21		5016p	800	1050	2858	3.7	$ 2,275	$ 8,304
1.22		6292_1	900	1050	2858	3.6	$ 2,275	$ 8,190
1.23	Outfall @Oxford St	6292_2	900	1050	2868	38.5	$ 2,275	$ 87,497
1.24	Russell Ave, Fir St to Best St	5100	300	375	170	46.0	$ 975	$ 44,821
1.25		5101	300	375	206	47.1	$ 975	$ 45,923
1.26		5102	375	450	245	13.2	$ 1,059	$ 13,936
1.27	Best St, Russell Ave to Thrift Ave	5634	375	525	299	138.5	$ 1,234	$ 170,909
1.28		5632	450	525	373	58.0	$ 1,234	$ 71,572
1.29	Thrift Ave, George St to east of Best St	5087	300	450	178	11.1	$ 1,059	$ 11,723
1.30		5690	300	525	215	43.9	$ 1,234	$ 54,185
1.31		5697	300	450	247	39.8	$ 1,059	$ 42,191
1.32		5698	375	450	267	12.3	$ 1,059	$ 13,005
1.33		5700	375	450	295	38.2	$ 1,059	$ 40,475
1.34		5701	375	450	320	44.5	$ 1,059	$ 47,094
1.35		5703	450	525	383	118.9	$ 1,234	$ 146,772
1.36		5704	600	675	783	132.9	$ 1,400	$ 186,004
1.37		5710	600	675	808	138.5	$ 1,400	$ 193,872
1.38	Fir St, Buena Vista Ave to Pacific Ave	5096p	300	525	148	6.0	$ 1,234	$ 7,404
1.39		6170	300	525	535	29.1	$ 1,234	$ 35,922
1.40		6169	300	525	544	20.0	$ 1,234	$ 24,643
1.41		6237	300	525	550	45.5	$ 1,234	$ 56,147
							Sub-total	**$ 2,419,316**
							Engineering - 10%	**$ 241,932**
							Contingency - 25%	**$ 604,829**
							Total	**$ 3,266,077**
Capacity Upgrades (2020)								
2.1	Marine Dr, west of Cypress St to Balsam St	6639	200	300	105	44.2	$ 878	$ 38,834
2.2		6657	250	300	155	44.2	$ 878	$ 38,843
2.3		5149	250	375	179	43.2	$ 975	$ 42,130
2.4		5150P	250	450	194	44.5	$ 1,059	$ 47,168
2.5	Balsam St, Marine Dr to south	6690	250	375	186	20.0	$ 975	$ 19,490
2.6	Marine Dr, Finlay St to west of Ash St	7168	200	300	29	81.3	$ 878	$ 71,355
2.7		6731	200	300	46	74.0	$ 878	$ 64,928
2.8		5155	200	375	74	61.6	$ 975	$ 60,099
2.9	Marine Dr/west of Ash St	5156	200	450	117	19.4	$ 1,059	$ 20,545
2.10	Lane north of Victoria Ave, west of Ash St	5153	100	200	19	98.7	$ 770	$ 75,976
2.11	Columbia Ave, Stayte Rd to Finlay St	Columbia-D1		1050	4404	93.2	$ 2,275	$ 211,962
2.12		Columbia-D2		1050	4492	109.0	$ 2,275	$ 247,975
2.13		Columbia-D3		1350	4893	97.5	$ 2,844	$ 277,290
2.14		Columbia-D4		1350	5008	65.8	$ 2,844	$ 187,021
2.15		Columbia-D5		1350	5007	45.4	$ 2,844	$ 129,004
2.16		Columbia-D6		1500	5058	89.8	$ 3,146	$ 282,354
2.17		Columbia-D7		1500	5139	103.5	$ 3,146	$ 325,705
2.18		Columbia-D8		1500	5162	97.4	$ 3,146	$ 306,515

2.19	Columbia Ave	Connections to Diversion Trunk		Varies				$ 25,000
2.20	Maple St, south of Columbia	7489	1050	1500	5632	87.9	$ 3,146	$ 276,408
2.21		7488	1050	1500	5634	103.5	$ 3,146	$ 325,548
2.22	Outfall	7485	1200	1350	6312	54.8	$ 2,844	$ 155,766
							Sub-total	**$ 3,229,914**
							Engineering - 10%	**$ 322,991**
							Contingency - 25%	**$ 807,479**
							Total	**$ 4,360,384**
Capacity Upgrades (2021)								
3.1	Buena Vista Ave, east of Best St to Fir St	5115	200	250	62	89.5	$ 817	$ 73,154
3.2		6129	250	450	202	44.3	$ 1,059	$ 46,893
3.3		6128	300	450	229	39.4	$ 1,059	$ 41,703
3.4		5127	250	450	247	24.5	$ 1,059	$ 25,914
3.5		5097p	300	525	391	94.3	$ 1,234	$ 116,354
3.6	Fir St, Buena Vista Ave to north	6102	150	200	52	63.0	$ 770	$ 48,510
3.7	Pacific Ave, Johnston Rd to First St	5078	450	525	274	18.5	$ 1,234	$ 22,878
3.8		6197	375	450	302	103.2	$ 1,059	$ 109,331
3.9		6232	375	450	332	43.8	$ 1,059	$ 46,363
3.10	Pacific Ave, Fir St to Dolphin St	6281	600	675	926	85.7	$ 1,400	$ 119,938
3.11		6303	600	675	985	62.0	$ 1,400	$ 86,730
3.12		5130	600	750	1030	135.7	$ 1,585	$ 215,037
3.13		6384	600	675	1084	17.8	$ 1,400	$ 24,976
3.14	Dolphin St, Pacific Ave to Royal Ave	6444	600	675	1092	84.4	$ 1,400	$ 118,160
3.15	Royal Ave, Centre St to Dolphin St	6389	300	375	74	150.0	$ 975	$ 146,221
3.16	Royal Ave, Dolphin St to Cypress St	5144	600	675	1187	120.2	$ 1,400	$ 168,252
3.17		6488	600	675	1187	6.7	$ 1,400	$ 9,394
3.18		6499	600	600	1187	15.3	$ 1,319	$ 20,233
3.19	Royal Ave, Fir St to west	6324	200	300	79	46.5	$ 878	$ 40,845
3.20		6332	200	300	66	11.4	$ 878	$ 10,009
3.21	Columbia Ave, Fir St to Johnston Rd	6423	250	375	82	3.4	$ 975	$ 3,286
3.22		6420	250	375	203	29.6	$ 975	$ 28,850
3.23		6403	250	375	220	36.2	$ 975	$ 35,285
3.24	Stayte Rd, Buena Vista Ave to south	BuenaVista-South		1050	4033	34.4	$ 2,275	$ 78,192
3.25	Stayte Rd, south of Buena Vista to Pacific Ave	7776	300	1050	4033	125.0	$ 2,275	$ 284,330
3.26		7775	300	1050	4033	37.7	$ 2,275	$ 85,813
3.27		7773	300	1050	4086	89.4	$ 2,275	$ 203,317
3.28		7771	300	1050	4086	64.1	$ 2,275	$ 145,919
3.29		7770	300	1050	4086	41.9	$ 2,275	$ 95,277
3.30		7769	300	825	4086	14.5	$ 1,650	$ 23,925
							Sub-total	**$ 2,475,088**
							Engineering - 10%	**$ 247,509**
							Contingency - 25%	**$ 618,772**
							Total	**$ 3,341,369**
Capacity Upgrades (2022)								
4.1	Stayte Rd, North Bluff Rd to south	5180	200	450	238	36.6	$ 1,059	$ 38,749
4.2	Stayte Road, Russell Ave to Thrift Ave	5510	600	750	1186	74.5	$ 1,585	$ 118,083
4.3		5613	600	750	1203	37.2	$ 1,585	$ 59,010
4.4		5686.1	600	750	1215	80.1	$ 1,585	$ 126,943

4.5	Stayte Road & Thrift Ave	5184P	600	750	1215	9.3	$ 1,585	$ 14,788
4.6		5185	200	825	2103	15.8	$ 1,650	$ 26,087
4.7	Stayte Road, Thrift Ave to Buena Vista Ave	5909	200	1050	2868	187.3	$ 2,275	$ 426,153
4.8		5943	375	1050	2868	8.3	$ 2,275	$ 18,837
4.9		7504	375	1050	3192	5.3	$ 2,275	$ 11,989
4.10		5193	375	1050	3227	191.0	$ 2,275	$ 434,525
4.11	North Bluff Rd, west of Kent St to east of Keil St	5243	250	300	158	90.7	$ 878	$ 79,599
4.12		5244	200	300	168	61.2	$ 878	$ 53,751
4.13		5245P	250	300	180	77.6	$ 878	$ 68,124
4.14	North Bluff Rd, west of Stevens St to Stayte Rd	5249	300	375	213	65.1	$ 975	$ 63,502
4.15		5250	300	375	227	97.8	$ 975	$ 95,365
4.16		5251	300	375	238	15.4	$ 975	$ 15,054
							Sub-total	**$ 1,650,557**
							Engineering - 10%	**$ 165,056**
							Contingency - 25%	**$ 412,639**
							Total	**$ 2,228,253**
Year 2023								
5.1	Martin St, Marine Lane to Marine Dr	5069p	300	375	294	21.9	$ 975	$ 21,382
5.2	Marine Dr, Martin St to west	5070	375	525	1045	30.8	$ 1,234	$ 37,958
5.3	Marine Dr, Martin St to west of Foster St	6484	300	375	125	12.3	$ 975	$ 12,032
5.4		5068	375	525	699	3.2	$ 1,234	$ 3,924
5.5		6454	375	525	715	81.4	$ 1,234	$ 100,435
5.6		6453	375	525	732	78.0	$ 1,234	$ 96,240
5.7	Marine Dr, east of Fir St	6495	300	375	125	35.5	$ 975	$ 34,652
5.8	Vidal St, Victoria Ave to Marine Dr	6346	300	600	207	55.0	$ 1,319	$ 72,545
5.9	Marine Dr, Vidal St to east of Elm St	6351	600	600	206	4.9	$ 1,319	$ 6,516
5.10		5073p	200	600	248	79.0	$ 1,319	$ 104,227
5.11		6320	300	600	269	15.4	$ 1,319	$ 20,326
5.12	North of outfall east of Elm St	6353	200	750	334	22.6	$ 1,585	$ 35,789
5.13	Marine Dr, Bay St to east	6081	250	375	58	88.3	$ 975	$ 86,122
5.14		6093	150	250	14	84.8	$ 817	$ 69,298
5.15	Outfall, east of Bay St	5003p	600	825	1221	23.6	$ 1,650	$ 38,907
5.16		6096	600	825	1235	8.0	$ 1,650	$ 13,200
5.17	Anderson St, W Beach Ave to Gordon Ave	6015	300	300	82	12.0	$ 878	
5.18		6014	200	300	108	45.7	$ 878	$ 40,098
5.19	North Bluff Road, north west corner of City	6774	250	375	30	122.2	$ 975	$ 119,165
5.20		6773	250	300	46	101.7	$ 878	$ 89,257
5.21	North Bluff Road, east of Cory Rd to Nichol Rd	6779	250	300	87	94.4	$ 878	$ 82,857
							Sub-total	**$ 1,084,929**
							Engineering - 10%	**$ 108,493**
							Contingency - 25%	**$ 271,232**
							Total	**$ 1,464,654**
Year 2024-2029								
1	Thrift Ave (local diversion)	Thrift-diversion		375	540	14.0	$ 975	$ 13,650
2	Thrift Ave, west of Stayte Rd	5738	300	525	738	148.8	$ 1,234	$ 183,607
3	Pacific Ave (local diversion)	Pacific-diversion		250	189	15.3	$ 817	$ 12,533
4	Habgood St, Cliff Ave to South	6414	300	375	250	54.5	$ 975	$ 53,138
5	Habgood St, north of Pacific	6519	300	375	262	22.5	$ 975	$ 21,977
6	Stevens St, Cliff Ave to south	6373	200	300	60	12.0	$ 878	$ 10,536
7		5203	200	300	111	39.7	$ 878	$ 34,848
8	Stevens St, north of Pacific Ave	7229	200	300	111	42.5	$ 878	$ 37,350

9	Propsect Cr to Buena Vista Ave	6053	200	300	68	18.3	$ 878	$ 16,076
10		6055	200	300	68	32.9	$ 878	$ 28,869
11		6056	200	300	68	47.8	$ 878	$ 41,977
12		6085	250	300	68	38.1	$ 878	$ 33,425
13		6139	250	300	68	22.0	$ 878	$ 19,316
14		7135	200	300	68	49.9	$ 878	$ 43,795
15		6152	200	300	68	4.0	$ 878	$ 3,512
16	Finlay St (@Balsam)	5133	150	250	40	43.5	$ 817	$ 35,548
17	Parker Street	5171.1	200	300	44	67.0	$ 878	$ 58,826
18	Parker Place	5161	200	250	44	118.5	$ 817	$ 96,839
19	Russell Ave, Kent to Maple St	5504	100	200	19	283.9	$ 770	$ 218,611
20	Thrift Ave, Martin St to Vidal St	5675.1	600	675	859	12.0	$ 1,400	$ 16,842
21		5674	600	675	863	93.8	$ 1,400	$ 131,250
22		5672.1	600	675	961	7.0	$ 1,400	$ 9,856
23		5666	600	750	1157	100.8	$ 1,585	$ 159,784
24	Blackwood St, Thrift Ave to north	5010	250	375	167	127.3	$ 975	$ 124,118
25	Oxford St, Russell Ave to Thrift Ave	5552	300	450	363	54.0	$ 1,059	$ 57,186
26		5598.1	300	450	409	42.7	$ 1,059	$ 45,198
27		5640	375	450	431	53.8	$ 1,059	$ 56,995
28		5655	375	450	434	19.2	$ 1,059	$ 20,290
29		5670	375	450	434	13.0	$ 1,059	$ 13,767
30	Oxenham Ave, west of Oxford St	5792	375	450	632	71.6	$ 1,059	$ 75,835
31	Vine Ave, to Russell Ave	5323	200	250	59	60.9	$ 817	$ 49,763
32		5055	200	250	59	81.5	$ 817	$ 66,586
33	Sunset Dr, Brearley St to Archibald Rd	7560	150	300	41	11.6	$ 878	$ 10,141
34		7315	200	300	42	109.5	$ 878	$ 96,159
35	Phoenix St, Vine Ave to south	6983	250	375	87	73.0	$ 975	$ 71,175
36	Lancaster St, Coldicutt Ave to Blackburn Ave	6817	250	375	119	12.0	$ 975	$ 11,700
37		6824	250	300	119	43.4	$ 878	$ 38,140
38	Malabar Ave, west of Lancaster St to Cory Rd	6891	200	300	34	101.8	$ 878	$ 89,416
39		6887	375	450	93	14.7	$ 1,059	$ 15,588
40		6885	375	450	281	88.9	$ 1,059	$ 94,177
41	Chestnut St, Coldicutte Ave to Blackburn Ave	7002	200	300	70	99.8	$ 878	$ 87,581
42	Marine Dr, west of Kerfood Rd	6969	250	300	161	40.9	$ 878	$ 35,910
43		6971	250	300	194	58.7	$ 878	$ 51,565
44	Phoenix St, near Park Ave	6858	375	450	148	44.0	$ 1,059	$ 46,543
45	Saturna Dr, east of Archibald Rd	5280	100	250	6	25.9	$ 817	$ 21,128
46	Magdalen Ave, Archibald Rd to Brearley St	5037	250	375	124	14.8	$ 975	$ 14,459
47		6920	250	375	152	90.3	$ 975	$ 88,082
48		7666	250	375	152	5.5	$ 975	$ 5,363
49	Magdalen Ave, east of Kerfood Rd	7670	300	450	272	18.9	$ 1,059	$ 19,983
50	Thrift Ave, west of Oxford St	5646	375	375	216	6.2	$ 975	$ 6,045
51	Finlay St, Oxenham Ave to south	7171	300	375	65	13.5	$ 975	$ 13,172
52		5939	300	375	111	44.0	$ 975	$ 42,900
53	Parker Pl (north of Russell Ave)	5161	200	250	44	118.5	$ 817	$ 96,839
54	Balsam St, Cliff Ave to Semiahmoo Ave	6354	200	300	99	40.4	$ 878	$ 35,480
55		6396	200	300	114	40.8	$ 878	$ 35,858
56		7303	200	300	114	11.7	$ 878	$ 10,299
57	Ash St, at Royal Ave	7631	375	450	402	21.7	$ 1,059	$ 22,980

Sub-total	$	2,852,583
Engineering - 10%	$	285,258
Contingency - 25%	$	713,146
Total	$	3,850,987

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