APPENDIX C. STORMWATER MANAGEMENT MODEL CALIBRATION & RESULTS

Provided separately for Parksville Community Park Stormwater Management Master Plan.

technical memo

Project Name	Parksville Community Park Stormwater Management Master Plan **Date** 20/04/2021
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Regarding	Parksville Community Park Stormwater Management Model Calibration & Results

1 Introduction

EOR was retained by the City of Parksville to develop the Parksville Community Park Stormwater Management Master Plan (PCPSWMMP). As part of this work, EOR is developing a stormwater management model to assess existing drainage conditions in the park. The model will be adjusted to assess potential future land use, climate and drainage conditions in the Park as well. This memorandum provides an update on the development of the existing conditions model.

2 Background Information

The following background information on the study area was provided by the City and other agencies:

1. Storm sewer map
2. Map of areas anecdotally prone to flooding
3. GIS layers, including:
    - Storm sewer, sanitary, and watermain pipes and structures
    - Roads
    - Buildings (limited extent)
    - Zoning and land use
    - Parcels, including land ownership
4. October 2016 light detection and ranging (LiDAR), provided by Regional District of Nanaimo
5. Climate station data:

Table 1. Summary of Climate Station Data

Station Name	Location	Recording Interval	Period of Record
060B-ParksvilleMuni	Community Park N 49.3223°, W 124.3082°	5 minute	2009-present
Public Works (Ops)	1116 Herring Gull Way N 49.3036°, W 124.2694°	30 minute	2008, 2017-2019
Nanaimo A **Station ID: 1025370**	N 49.0544°, W 124.8700°	hourly	1954-2013
Station ID: 1025369		hourly	2013-2020
Park Operations	Community Park	n/a	n/a

6. Park irrigation zones
7. Monitoring data from level loggers installed in catchbasins DMH15, DMH43, DMH14, DMH3 and DMH21 (identified in CCTV report) on November 8, 2019 and removed March 10, 2020
8. Surficial soil map published by the Geological Survey of Canada (Fyles, 1963)

9. Sea water levels (2002 to 2020) at Point Atkinson BC, Station 7795 (Fisheries & Oceans Canada, 2019)

Past plans, studies and standards were also reviewed for pertinent information, including the following:

1. Plan Parksville: A Vision for Our Future – Official Community Plan (City of Parksville, 2013)
2. Parksville Community Park Shoreline Erosion Protection (Northwest Hydraulics Consultants Ltd., 2015)
3. City Storm Drainage Master Plan (Koers & Associates Engineering Ltd., 2016)
4. Community Park Master Plan 2017-2037 (Vancouver Island University & City of Parksville, 2017)
5. City of Parksville Engineering Standards and Specifications (City of Parksville, 2018)
6. Parks, Trails and Open Spaces Master Plan (City of Parksville et al., 2019)

As part of the PCPSWMMP project, EOR also coordinated compilation of the following additional data:

1. CCTV of the sewer network by Pipe-Eye Video Inspections & Services Ltd (Pipe-Eye) in January and February 2020
2. Tape-down measurements at storm sewer structures compiled by Pipe-Eye in January and February 2020
3. Utilities located by Kelly's 1st Call Locating Ltd in January and February, 2020
4. Utilities surveyed by JE Anderson & Associates Ltd (JE Anderson) in February and March, 2020
5. Topographic survey of the Park by JE Anderson in January and February 2020
6. Topographic survey of the rights-of-way adjacent to the Park on Corfield Drive and Highway 19A, by Sims Associates Land Surveying Ltd in May 2020
7. Development of mid and late-century Intensity-Duration-Frequency Curves by Dillon Consulting (Dillon Consulting, 2020)
8. Geotechnical investigation of selected sites conducted by Thurber Engineering in May 2020
9. Coastal Inundation assessment and mapping by Northwest Hydraulics Consultants Ltd. (Northwest Hydraulics Consultants, 2020a)
10. Sea water level timeseries adjusted to Parksville Bay by Northwest Hydraulics Consultants Ltd.(Northwest Hydraulics Consultants, 2020b)

In addition, EOR conducted site visits from January 29 to 31, 2020 to confirm site conditions.

3 Existing Conditions

The Community Park is located in the centre of Parksville, BC on the east side of Vancouver Island. The Park is bordered by Highway 19A to the south, Corfield Street North to the east, the Park Sands Beach Resort to the west, and Parksville Bay to the north. Land cover in the Park includes buildings, parking lots (paved and gravel), roads, trails, a skate park, beach volleyball courts, playgrounds, baseball diamonds, tennis courts, a basketball/lacrosse court, a sandcastle exhibition space, a splash pad, a tree arboretum, and other open spaces. The City operates and maintains the Park year-round, including a comprehensive irrigation system and frequent street sweeping.

Elevations in the Park range from sea level to 5 m above sea level, with a steep slope on the southern boundary rising to 11 m above sea level. Most of the 16 ha park drains to a storm sewer network which discharges to an outfall to Parksville Bay, located at the northeast corner of the site, without any water quality or quantity control. Outfall capacity is periodically limited by tides and sediment accumulation in the sewer from the outfall. A small area at the southeast corner of the Park drains to the municipal storm sewer on Corfield Street North, which discharges to the western portion of the Englishman River estuary. Other areas of the Park drain to isolated underground infiltration chambers referred to as rock pits or dry wells. The existing roads and parking lots direct runoff to the storm sewer network via curb and gutter systems. Emergency overland flow capacity to the bay is limited because the shoreline and trail system along the north boundary of the Park are elevated above inland areas of the park. Surficial soils on site are primarily salish sediments (i.e. shore, deltaic and fluvial deposits composed of gravel, sand, silt, clay and peat) with a small area of terraced fluvial deposits at the southeast corner (i.e. deltaic deposits composed of gravel and sand underlain by silt and clay) (Fyles, 1963). Additional geotechnical information was collected to characterize soils and depth to groundwater on the site (Thurber Engineering, 2020).

The City and park users have identified nuisance flooding issues along roads, in parking lots and along the walking trails. The nuisance flooding typically recedes within a day or so, however in the wet winter season it is common for some nuisance flooding areas to remain flooded for multiple days. Prolonged flooding may be causing premature deterioration of pavement. One maintenance building south of the playground has flooded, however no other structures have been flooded in the past based on the City's anecdotal records.

The Park irrigation system draws from the City's drinking water system, which was recently expanded to support on-going development in the region. The City irrigates based on precipitation recorded at the Park (City of Parksville, n.d.).

Water from the splashpad and outdoor showers is currently directed to the municipal sanitary sewer.

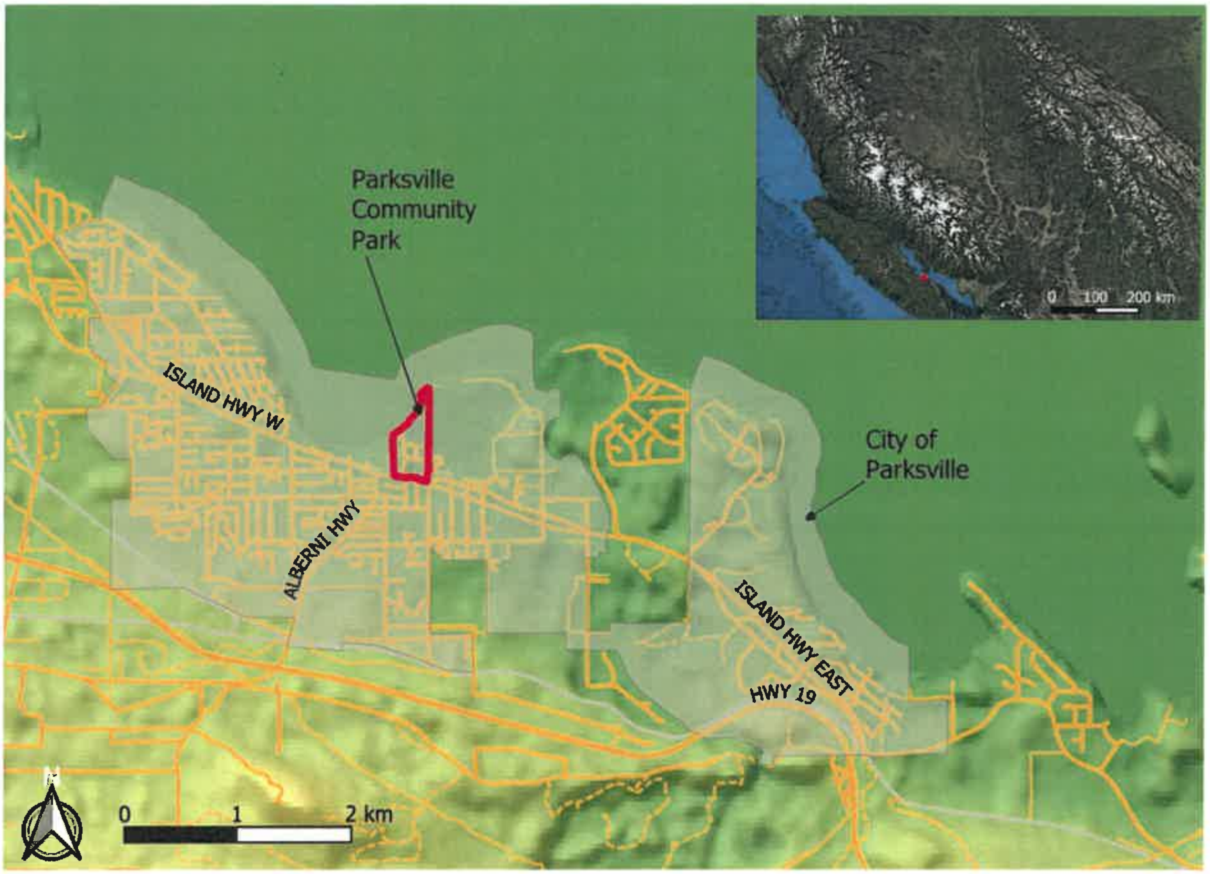

Figure 1. Location Map

4 Existing & Future Conditions Model Development

EOR created a model of the minor and major drainage system in the Park using PCSWMM 2D. The model was developed to assess the nuisance flooding issues in the park. This section of the report describes how the model was developed.

4.1 Data Processing

EOR utilized and processed the available background data in order to develop the 2D PCSWMM model of the park. Data processing included the following tasks:

- Catchbasin rims were defined using surveyed data and converted from CGVD28 to CGVD2013 to match the Lidar-derived digital elevation model (DEM).
- Storm sewer information was consolidated from Autocad, spreadsheet, and PDF formats into the PCSWMM existing conditions model. CCTV invert depths were converted to elevations based on catchbasin and manhole rim elevations. Missing CCTV inlet invert data in the circle was filled using hand sketched record drawings from the City. Other missing invert data in the Park was approximated using either the downstream or upstream pipe slopes to estimate the manhole inverts.

- Depression areas were defined using the DEM in PCSWMM. The DEM was resampled in PCSWMM to define subcatchment slopes.
- Subcatchment impervious percentage was parameterized using survey information in AutoCAD.

4.2 Climate

4.2.1 Historic Rainfall Events

The City has operated two weather stations within Parksville at the Public Works yard (30 minute intervals) and in the Park (5 minute intervals). Rainfall data from the Park station will be used for simulating historic events and calibrating the model. No winter storms with return periods greater than 5 years have been recorded at the Park station. One short-duration, high-intensity rainfall event was observed in September 2013. The extreme historic events recorded at the Park station are summarized in Table 2.

Table 2. Historic Rainfall Events at Community Park Weather Station

Date	Total Rainfall Depth (mm)	Maximum Intensity (mm/hr)	Duration	Estimated Return Period	
Oct 1-2, 2009	14.6	14	2 hours	2 year	5 minute to 2 hour
Nov 18-19, 2009	80	9.6	2 days	2 to 5 year	6 to 24 hour
Sept 2, 2013	33.2	96	30 minute	> 100 year	5 minute to 2 hour
Oct 21-22, 2014	42	4.8	2 days	2 year	6 to 12 hour
Jan 10-11, 2014	45.2	12	2 days	< 5 year	5 minute to 24 hour
Feb 15-16, 2014	44	12	2 days	< 5 year	5 minute to 24 hour
Dec 8-11, 2014	98.8 mm	12	4 days	2 year	4 day
Jan 31-Feb1, 2020	47.4 mm (80.2 mm in preceding week)	7.2	2 days	< 2 year	48 hour

4.2.2 Intensity-Duration-Frequency Curves

The City's current Engineering Standards and Specifications (City of Parksville, 2018) include the Intensity-Duration-Frequency (IDF) curves that were developed as part of the City-wide Storm Drainage Master Plan. The IDF curves were developed by factoring the Environment Canada Nanaimo City Yard climate station (ID: 10253G0) IDF to the City of Parksville South climate station (ID: 1025977) based on the correlation between the rainfall data recorded at each station over the same time period (1983 to 1992). The Nanaimo City Yard station included a 25 year period of record from 1980 to 2005 (Koers & Associates Engineering Ltd., 2016). As part of the Stormwater

Management Master Plan for the Community Park, Dillon Consulting reviewed available climate data and updated the IDF curves using the Nanaimo Airport data (1985-2017) to also include multi-day events. The updated IDF curves are provided in Table 3. Future climate changed IDF curves for expected conditions in 2100 are provided in

Table 4. Additional details, including a comparison with IDF curves contained in the City of Parksville Engineering Standards, are provided in City of Parksville – Rainfall Design & Climate Change Guidance (Dillon Consulting, 2020)and in the separate MS Excel Spreadsheets prepared by Dillon Consulting (e.g. 25th and 75th percentile IDF Curves).

Table 3. Rainfall Depth-Duration-Frequency Curves (mm) based on Nanaimo Airport (1985-2017)

Duration	Return Period					
	2 year	5 year	10 year	25 year	50 year	100 year
5-min	2.8	3.7	4.3	5	5.6	6.1
10-min	4.1	5.6	6.6	7.8	8.8	9.7
15-min	5	7.1	8.5	10.3	11.6	13
30-min	7.1	10.1	12.1	14.7	16.6	18.4
1-h	10	13.4	15.7	18.5	20.7	22.8
2-h	14.9	18.2	20.3	23.1	25.1	27.1
6-h	29.8	35.3	38.9	43.5	46.9	50.2
12-h	42	50.4	56	63	68.2	73.4
24-h	55.6	69.7	79	90.9	99.6	108.3
2-day	69.8	85.6	96.0	109.2	119.0	128.7
3-day	81.8	99.0	110.4	124.8	135.5	146.1
4-day	96.1	117.0	130.9	148.4	161.4	174.3
5-day	108.6	133.2	149.5	170.1	185.4	200.6
6-day	118.1	142.9	159.4	180.1	195.5	210.8
7-day	124.9	151.3	168.9	191.0	207.4	223.7
8-day	133.5	162.1	181.0	204.9	222.6	240.3
9-day	142.5	172.9	193.1	218.5	237.4	256.2
10-day	150.6	183.5	205.3	232.9	253.4	273.6

Source: Rainfall Design & Climate Change Guidance – Final Technical Report (Dillon Consulting, 2020)

Table 4. Mean Future (2080s) Rainfall Depth-Duration-Frequency Curves for Parksville, BC (mm)

Duration	Return Period					
	2 year	5 year	10 year	25 year	50 year	100 year
5-min	3.7	4.9	5.7	6.6	7.4	8.1
10-min	5.4	7.4	8.8	10.4	11.7	12.9
15-min	6.6	9.4	11.3	13.7	15.4	17.3
30-min	9.4	13.4	16.1	19.5	22.1	24.5
1-h	13.3	17.8	20.9	24.6	27.5	30.3
2-h	19.5	23.9	26.6	30.3	32.9	35.5
6-h	37.0	43.8	48.2	53.9	58.2	62.2
12-h	52.1	62.5	69.4	78.1	84.6	91.0
24-h	68.9	86.4	98.0	112.7	123.5	134.3
2-day	86.6	106.1	119.1	135.4	147.6	159.6
3-day	101.4	122.8	136.9	154.7	168.0	181.1
4-day	119.1	145.1	162.3	184.0	200.1	216.2
5-day	134.6	165.2	185.4	210.9	229.9	248.7
6-day	146.5	177.2	197.6	223.3	242.4	261.4
7-day	154.8	187.6	209.4	236.8	257.2	277.4
8-day	165.5	201.0	224.4	254.1	276.1	297.9
9-day	176.7	214.4	239.4	271.0	294.4	317.6
10-day	186.7	227.6	254.6	288.8	314.2	339.3

Source: Rainfall Design & Climate Change Guidance – Final Technical Report (Dillon Consulting, 2020)

4.2.3 Hyetographs

The City-wide Storm Drainage Master Plan considered the applicability of multiple synthetic hyetographs to representing the distribution (i.e. amount and intensity) of rainfall events over time in Parksville. The plan considered the Atmospheric Environment Services (AES) Canada, Soil Conservation Services (SCS), and Huff distributions. The Chicago distribution was not considered because it does not represent rainfall patterns for the BC coast. Multiple durations of the AES distribution were simulated in the City-wide XPSWMM model, which indicated that the 1-hour AES hyetograph governed all systems except for the Romney Creek catchment governed by the 6-hour duration storm (Koers & Associates Engineering Ltd., 2016).

The PCPSWMMP existing conditions model is intended to test inlet, sewer and overland capacity for the 10-year and 100-year return period events. The preliminary model will be run using the 1-hour AES BC Coast and the 24-hour SCS Type IA (Pacific Coast) distributions. These simulations will test which event governs in terms of conveying short-high intensity events and/or temporarily detaining longer-duration events.

The PCPSWMMP future conditions model will confirm the performance of recommended stormwater management upgrades to the park, along with expected park infrastructure changes to be implemented by 2040 (see Figure 3 and Figure 5). The 2100 10-year 24 hour SCS Type IA (Pacific Coast) return period rainfall event was used to confirm the inlet and sewer capacity to achieve the goal of preventing surface ponding. Overland flow capacity and flooding depth was confirmed using the 2100 10-year 24 hour SCS Type IA (Pacific Coast) return period rainfall event. The 2100 100-year AEP coastal inundation event (see Figure 6) was applied to the 2D model to confirm that the system is able to discharge the coastal inundation through a combination of overland flow and storm sewer capacity.

4.3 Hydrology

All subcatchment parameters are summarized in Table 5 and were defined as follows:

- Subcatchments were delineated for every catchbasin in the subwatershed using the DEM (see Figure 2. Existing Conditions Subcatchments) and further subdivided as deemed necessary for identifying internally drained areas, yielding a total of 42 subcatchments. Existing conditions subcatchment parameters are detailed for each subcatchment in Table 5, while future land use changes are reflected in the future conditions subcatchment parameters detailed in Table 6.
- Subcatchment slopes were calculated based on the DEM (upscaled to 10m resolution) in PCSWMM.
- Subcatchment lengths were defined based on length of gutters and/or length of overland flow.
- Subcatchment widths were automatically calculated in PCSWMM by dividing the subcatchment area by the subcatchment length.
- Green-Ampt infiltration and runoff parameters were defined based on the values for sandy loam soils summarized in Table 7, and updated with results from the geotechnical survey, as needed.
- Existing imperviousness cover was initialized based on topographic survey of buildings, pavement and gravel surfaces in the park. Future imperviousness combines the existing conditions with expected changes including planned infrastructure upgrades from the CPMP, the amphitheatre and the conceptual design of the Sandcastle Drive extension (currently in development).
- Subcatchment routing was used to direct runoff from buildings onto lawns to represent the disconnected roof drains observed through the majority of the park. All other impervious surfaces were generally modeled as draining to the catchbasin.
- Manning's roughness and depression storage were defined based on the SWMM 5 manual (see footnotes to Table 5).

Table 5. Existing Conditions Model Subcatchment Parameters

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Suction Head (mm) ***	Conductivity (mm/hr) ***	Initial Deficit (frac.) ***
S001	0.823	337.295	24.4	9.086	24.411	0.016	0.097	1.27	2.54	0	OUTLET	100	24.51	60.2	0.404
S002	0.4305	51.993	82.8	5.011	41.229	0.016	0.299	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S003	0.1045	22.817	45.799	1.862	53.428	0.017	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S004	4.4337	100	443.37	3.065	38.108	0.021	0.243	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_1	0.2638	23.12	114.1	3.46	14.603	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_2	0.7911	69.334	114.1	3.46	14.603	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S006	0.2019	21.097	95.701	5.154	63.395	0.016	0.206	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S007	0.3384	31.925	105.998	1.309	29.164	0.016	0.231	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S008	0.1163	21.379	54.399	1.292	45.65	0.016	0.221	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S009	1.0274	54.455	188.67	1.839	0	0	0.011	1.27	2.54	0	OUTLET	100	24.51	60.2	0.404
S010	0.4309	42.524	101.331	2.4	22.598	0.019	0.092	1.27	2.54	3.4	OUTLET	100	24.51	60.2	0.404
S011	0.0116	12.046	9.63	2.089	73.683	0.032	0.314	1.27	2.54	27.1	PERVIOUS	100	30.48	14.985	0.382
S012	1.6319	109.362	149.22	1.449	12.304	0.022	0.272	1.27	2.54	4.5	PERVIOUS	30	30.48	14.985	0.382
S013	0.0988	21.858	45.201	1.972	60.299	0.016	0.263	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S014	0.0191	27.681	6.9	1.629	41.74	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S015	0.6268	91.237	68.7	1.837	22.454	0.016	0.359	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S016	0.4101	50.443	81.3	1.711	16.695	0.016	0.337	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S017	0.7046	75.52	93.3	1.793	60.092	0.022	0.234	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S018	0.3648	29.396	124.099	1.768	47.103	0.016	0.291	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S019	0.0936	18.634	50.231	0.77	84.02	0.016	0.375	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S020	0.1671	34.461	48.49	0.985	66.287	0.016	0.395	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S021	0.1949	22.795	85.501	1.254	36.126	0.016	0.386	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S022	0.2041	21.076	96.84	1.981	85.566	0.016	0.24	1.27	2.54	0.57	OUTLET	100	30.48	14.985	0.382
S023	0.0537	21.566	24.9	1.296	93.351	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S024	0.0769	19.468	39.501	1.049	84.939	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S025	0.1709	31.101	54.95	2.225	59.069	0.016	0.259	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S026	0.1757	24	73.208	1.721	78.704	0.094	0.24	1.27	2.54	92	PERVIOUS	100	30.48	14.985	0.382
S027	0.1611	24	67.125	1.787	98.694	0.084	0.24	1.27	2.54	80	PERVIOUS	100	30.48	14.985	0.382
S028	0.2435	26.788	90.899	2.185	49.983	0.038	0.338	1.27	2.54	25.7	PERVIOUS	25	30.48	14.985	0.382
S029	0.0469	8.621	54.402	3.646	41.039	0.016	0.4	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S030	0.2962	28.29	104.701	1.703	16.491	0.017	0.379	1.27	2.54	0.8	OUTLET	100	30.48	14.985	0.382
S031	0.0267	17.485	15.27	2.239	42.84	0.024	0.344	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S032	0.2305	61.631	37.4	0.922	84.739	0.023	0.282	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S033	0.0348	13.073	26.62	1	85.754	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S034	0.0555	14.59	38.04	0.859	78.307	0.016	0.297	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S035	0.0835	39.952	20.9	0.66	86.266	0.016	0.33	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S036	0.1196	22.912	52.2	1.549	8.126	0.016	0.365	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S037	0.5233	36.09	144.999	1.745	9.681	0.047	0.4	0.05	0.05	32.4	PERVIOUS	100	30.48	14.985	0.382
S038	0.6488	54.067	119.999	2.804	1.785	0.016	0.4	0.05	0.05	0	OUTLET	100	30.48	14.985	0.382
S039	0.3178	103.35	30.75	17.204	37.038	0.016	0.274	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S040	2.8872	74.135	389.451	5.157	18.17	0.017	0.253	1.27	2.54	1	PERVIOUS	60	30.48	14.985	0.382
S040_2	0.1099	2.822	389.451	5.157	18.17	0.017	0.253	1.27	2.54	1	PERVIOUS	60	30.48	14.985	0.382

Notes:

* Area weighted average based on land cover values in Table 8

** Table A.5 of the SWMM 5 User Manual: Lawns (0.1 inches) and Impervious Surfaces (0.05 inches) (James et al., 2010)

*** See Table 7

Table 6. Future Conditions Model Subcatchment Parameters

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Suction Head (mm) ***	Conductivity (mm/hr) ***	Initial Deficit (frac.) ***
Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Suction Head (mm)	Conductivity (mm/hr)	Initial Deficit (frac.)
S002	0.4305	51.993	82.8	5.011	18.5	0.016	0.299	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S006	0.2209	23.082	95.702	5.154	63.395	0.016	0.206	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S010	0.4309	42.524	101.331	2.4	22.598	0.019	0.092	1.27	2.54	3.4	OUTLET	100	24.51	60.2	0.404
S013	0.0988	21.858	45.201	1.972	60.299	0.016	0.263	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S011	0.0116	12.046	9.63	2.089	73.683	0.032	0.314	1.27	2.54	27.1	PERVIOUS	100	30.48	14.985	0.382
S028	0.2435	26.788	90.899	2.185	49.983	0.038	0.338	1.27	2.54	25.7	PERVIOUS	25	30.48	14.985	0.382
S035	0.0835	39.952	20.9	0.66	86.266	0.016	0.33	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S029	0.0469	8.621	54.402	3.646	41.039	0.016	0.4	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S009	1.0274	54.455	188.67	1.839	0	0	0.011	1.27	2.54	0	OUTLET	100	24.51	60.2	0.404
S019	0.0936	18.634	50.231	0.77	84.02	0.016	0.375	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S008	0.1163	21.379	54.399	1.292	45.65	0.016	0.221	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S024	0.0769	19.468	39.501	1.049	84.939	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S023	0.0537	21.566	24.9	1.296	93.351	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S022	0.2041	21.076	96.84	1.981	85.566	0.016	0.24	1.27	2.54	0.57	OUTLET	100	30.48	14.985	0.382
S030	0.2962	28.29	104.701	1.703	16.491	0.017	0.379	1.27	2.54	0.8	OUTLET	100	30.48	14.985	0.382
S031	0.0267	17.485	15.27	2.239	42.84	0.024	0.344	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S036	0.1196	22.912	52.2	1.549	8.126	0.016	0.365	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S039	0.3178	103.35	30.75	17.204	37.038	0.016	0.274	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S033	0.0348	13.073	26.62	1	85.754	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S032	0.2305	61.631	37.4	0.922	84.739	0.023	0.282	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S012	1.6319	109.362	149.22	1.449	12.304	0.022	0.272	1.27	2.54	4.5	PERVIOUS	30	30.48	14.985	0.382
S003	0.1045	22.817	45.799	1.862	53.428	0.017	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S014	0.0191	27.681	6.9	1.629	41.74	0.016	0.24	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S015	0.6268	91.237	68.7	1.837	22.454	0.016	0.359	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S016	0.4101	50.443	81.3	1.711	16.695	0.016	0.337	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S017	0.7046	75.52	93.3	1.793	60.092	0.022	0.234	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S004	4.4337	100	443.37	3.065	38.108	0.021	0.243	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S026	0.1757	24	73.208	1.721	78.704	0.094	0.24	1.27	2.54	92	PERVIOUS	100	30.48	14.985	0.382
S027	0.1611	24	67.125	1.787	98.694	0.084	0.24	1.27	2.54	80	PERVIOUS	100	30.48	14.985	0.382
S001	0.823	337.295	24.4	9.086	24.411	0.016	0.097	1.27	2.54	0	OUTLET	100	24.51	60.2	0.404
S037	0.5233	36.09	144.999	1.745	9.681	0.047	0.4	0.05	0.05	32.4	PERVIOUS	100	30.48	14.985	0.382
S038	0.6488	54.067	119.999	2.804	15.7	0.016	0.4	0.05	0.05	0	OUTLET	100	30.48	14.985	0.382
S025	0.1709	31.101	54.95	2.225	59.069	0.016	0.259	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S040	2.8872	74.135	389.452	5.157	18.17	0.017	0.253	1.27	2.54	1	PERVIOUS	60	30.48	14.985	0.382
S040_2	0.1099	2.822	389.44	5.157	18.17	0.017	0.253	1.27	2.54	1	PERVIOUS	60	30.48	14.985	0.382
S1_4	0.0592	7.4	80	8.691	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_2	0.3012	37.65	80	7.582	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_5	0.1672	29.857	56	2.184	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_3	0.175	29.167	59.999	7.687	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_1	0.2269	36.597	62	10.242	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_6	0.1967	30.262	64.999	12.656	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1_8	0.4602	46.959	98	11.988	40	0.016	0.1	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S007_1	0.1378	13	106	1.309	71.3	0.016	0.231	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S007_2	0.2006	18.925	105.997	1.309	0	0.016	0.231	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_3	0.1346	11.797	114.097	3.46	53.5	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_4	0.1291	11.315	114.096	3.46	8	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_1	0.1685	14.768	114.098	3.46	46.2	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S005_5	0.6037	52.91	114.099	3.46	0	0.016	0.22	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S018_1	0.3572	28.783	124.101	1.768	47.103	0.016	0.291	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Suction Head (mm) ***	Conductivity (mm/hr) ***	Initial Deficit (frac.) ***
S020_1	0.1315	27.119	48.49	0.985	66.287	0.016	0.395	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S021_1	0.1711	20.011	85.503	1.254	36.126	0.016	0.386	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S034_2	0.0452	11.882	38.041	0.859	78.307	0.016	0.297	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382
S1	0.0774	11.724	66.018	1.129	56.684	0.016	0.369	1.27	2.54	0	OUTLET	100	30.48	14.985	0.382

Table 7. Green Ampt Parameters for Sandy Loam Soils

Soil Texture Code	Loamy Sand	Sand
Conductivity (mm/hr)	29.972	120.396
Suction Head	60.96	49.022
Porosity	0.437	0.437
Field Cap	0.105	0.062
Wilting	0.047	0.024
Initial Deficit	0.382	0.404

Source: (Rawls et al., 1983)

Table 8. Manning's Roughness of Overland Flow

Material	Manning's Roughness, n
Grass	0.24 **
Woods with light underbrush	0.4 *
Gravel	0.024 ***
Bare Sand	0.01 ****
Paved Surface	0.016 *

* City of Parksville Engineering Standards and Specifications (City of Parksville, 2018)

** dense grass (McCuen et al., 2002)

*** cement rubble surface (McCuen et al., 2002)

**** bare sand (Engman, 1986)

Parksville Community Park
Stormwater Management
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Figure 2: Existing Park Topography

EOR
water
ecology
community

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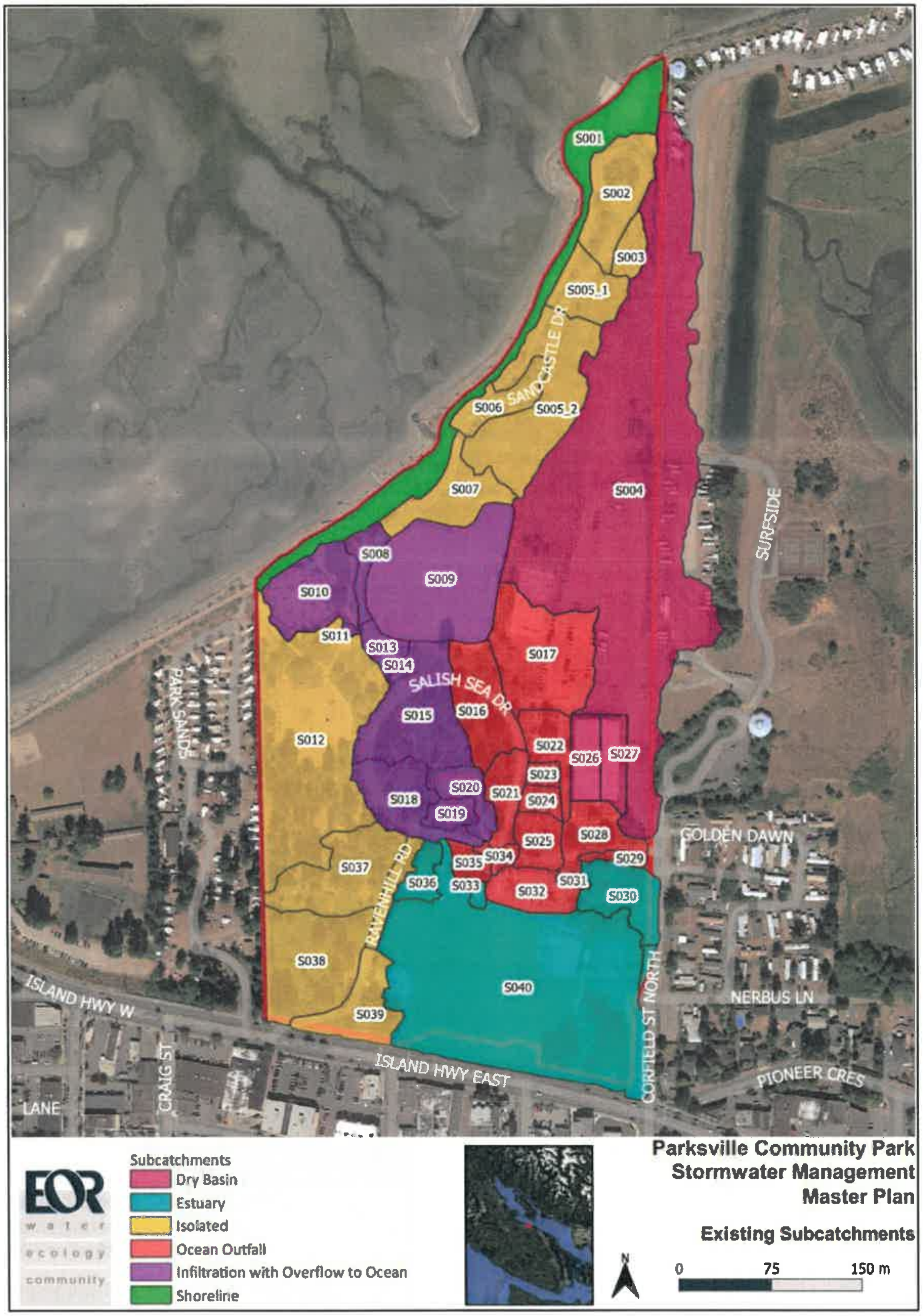

Figure 2. Existing Conditions Subcatchments

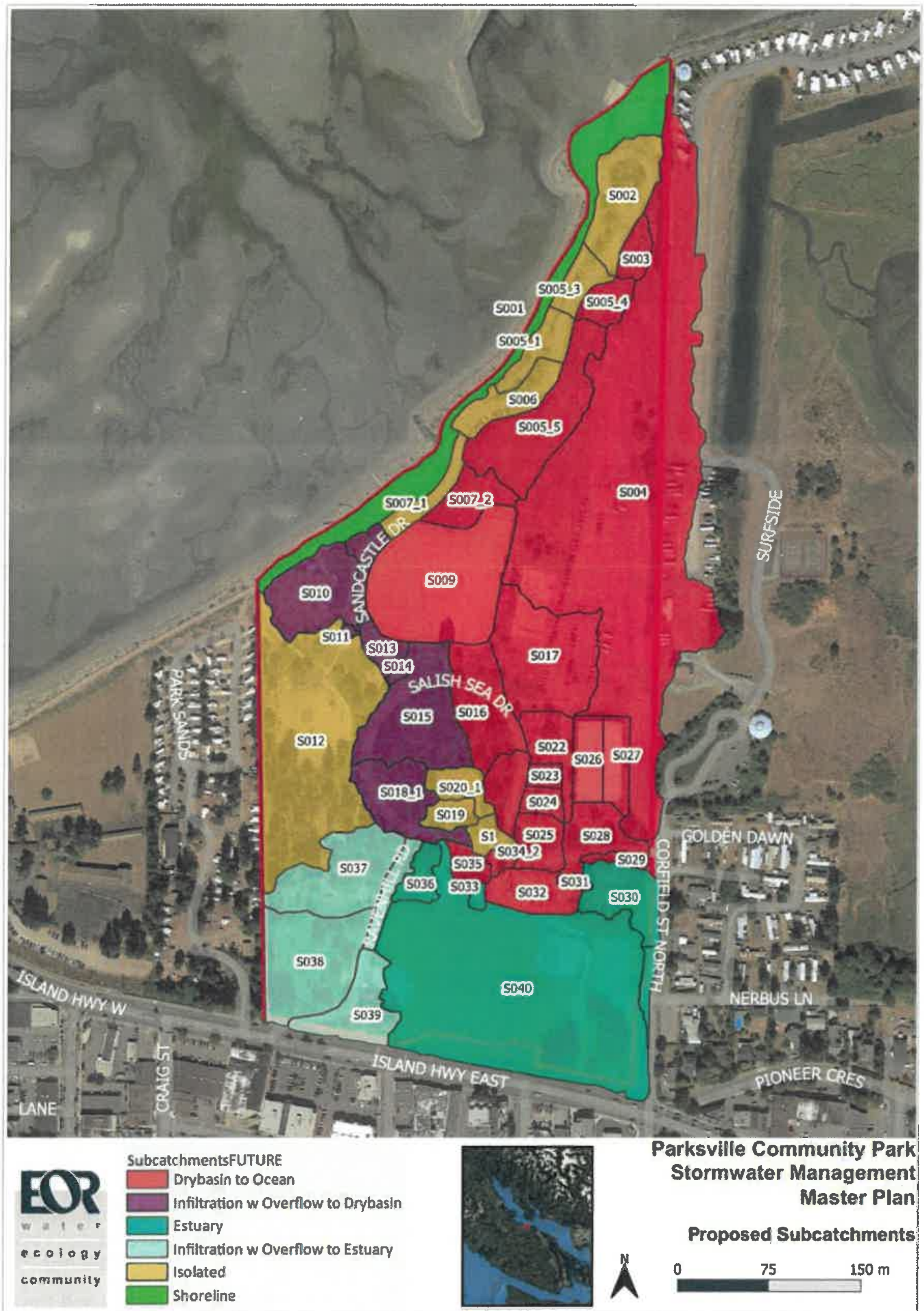

Figure 3. Future Conditions Subcatchments

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EOR water ecology community

Bare Sand		Boardwalk	
Building		Paved Parking	
Grass		Pedestrian	
Gravel		Paved Surface	
Road		Permeable Pavers	
Rubber		Gravel Pathway	
Woods			

Parksville Community Park
Stormwater Management
Master Plan

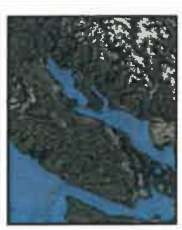
Figure 4: Existing Land Cover

N

0 75 150 m

EOR water ecology community

- Bare Sand
- Building
- Grass
- Gravel
- Paved Parking
- Permeable Pavers
- Rubber
- Woods
- Road
- Pedestrian
- Dry Basin
- Ditch
- Paved Surface
- Boardwalk
- Raingarden



Parksville Community Park Stormwater Management Master Plan

Figure 5: Proposed Land Cover

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0 75 150 m

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Date: 2021-03-25T12:11:20.891 Author: Kerri Robinson Layout: RM_INUNDATION DEPTH
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EOR water ecology community

- 10 yr AEP 2020
- 100 yr AEP 2020
- 10 yr AEP 2100
- 100 yr AEP 2100
- Park Boundary

Depth (m)

- 0.01
- 0.3
- 1
- 1.5
- 2
- 2.7

Parksville Community Park
Stormwater Management Master Plan

Figure 6. Coastal Inundation Mapping for Year 2020 and Year 2100 (Northwest Hydraulics Consultants, 2020a)

N

0 75 150 m

4.4 Existing Conditions Hydraulics

Key components of the model hydraulics included the storm sewer system and the 2D mesh representing the overland flow pathways.

Storm sewer data was primarily sourced from the CCTV survey provided in January and March 2020, and provided to the City of Parksville Engineering Department in hardcopy reports and digital video files. AutoCAD drawings were used to fill in missing storm pipe data. All rim elevations were defined using the DEM and confirmed by survey in select locations. Catchbasin invert elevations and diameter of leads were calculated by subtracting the CCTV depths from the rim elevation defied by the DEM. Ponding was enabled at every junction and a nominal amount of ponding area (1 m^2) was provided. Entry and exit losses were defined by the pipe connection at the manhole. Entry losses were estimated as 0.5 throughout. Straight-thru junctions had an exit loss coefficient of 0.5. Junctions that met at an angle of less than 90° were assigned an exit loss coefficient of 0.7, and those that met at an angle greater than 90° were assigned an exit loss coefficient of 0.9. Pipes that emptied to rock pit storage areas were given an exit loss coefficient of 1. Manning's n for storm pipes was set to 0.013 throughout.

Several catchments drain to rock pits. These structures serve as infiltration areas, however no data on their construction was available. In addition, DMH42 (the manhole located at the northwest corner of the oval drive and Beachside Drive) was noted in the record drawings as an infiltration manhole. The storage and infiltration capacity of this structure was uncertain due to limited information and were considered later for calibration. These structures were modeled as storage units with a constant area and infiltration rate (mm/hr). Infiltration at DMH42 was calibrated during the 1D model calibration, while the remaining areas were calibrated during 2D model calibration.

The 2D mesh was developed using the DEM and PCSWMM's built-in 2D modeling tools to represent overland flow pathways. A trial run was used to define the smallest possible bounding layer that encompassed all overland flow routes in the subwatershed while minimizing run times. A hexagonal mesh was generated with a 3 m resolution, sampling factor of 1, distance tolerance of 3 m, elevation tolerance of 0.3 m, roughness of 0.033, and seepage rate of 0. Edges were defined along road curbs.

The 2D mesh was connected to the storm sewer network using a standard horizontal rectangular orifice (571 mm by 305 mm) with a discharge coefficient of 0.65 to represent the openings in the City's standard catchbasin (SP582-05.02, BC Ministry of Transportation (MOT), 2005). This catchbasin inlet is common throughout the City and Community Park. Unique inlets, if found, should be incorporated during detailed design.

The water level at the outfall to Parksville Bay was defined using a time series of tidal data covering the time period September 2019-May 2020 and based on CGVD2013 (Northwest Hydraulics Consultants, 2020a). Design storm hyetographs were timed to coincide with a normal high tide cresting at 1.835m at 8:00pm on January 19th, 2020. The future conditions water level at the outfall to Parksville Bay was defined by adding 0.79m to the September 2019-May 2020 (CGVD2013) tidal time series data to reflect expected sea level rise countered by land uprising by 2100 (Northwest Hydraulics Consultants, 2020a).

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EOR
water
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community

- Drainage Manhole
- Area Drain
- Catchbasin
- Outfall
- Outlet Stub
- Rock Pit
- Infiltration Manhole
- LoggerLocations
- 0.075m Pipe
- 0.1m Pipe
- 0.15m Pipe
- 0.2m Pipe
- 0.3m Pipe
- 0.45m Pipe
- 0.75m Pipe
- Ditch
- Dry Basin
- Park Boundary

Parksville Community Park
Stormwater Management
Master Plan
Figure 7: Existing Storm Sewer Network

N

0 75 150 m

4.5 Future Conditions Hydraulics

Nerbus Lane

The connection between DMH21 (inv. 2.22m) and the estuary outfall (inv. 2.006m CGVD2013) is a very flat system (0.088% slope). In order to ensure that catchbasins are not surcharging and flooding the adjacent residential area during large storm events, the pipe connection between the Lacrosse Box (DMH21) and the outfall was sized at 450mm. Corfield Drive catchbasins will need to be redirected to the new storm line, along with existing CBs along Nerbus Lane. If catchbasins are replaced, sumps are highly recommended for maintenance purposes. Maintenance manholes should be placed along the length, to allow regular cleaning of the flat storm pipe, since sedimentation is likely.

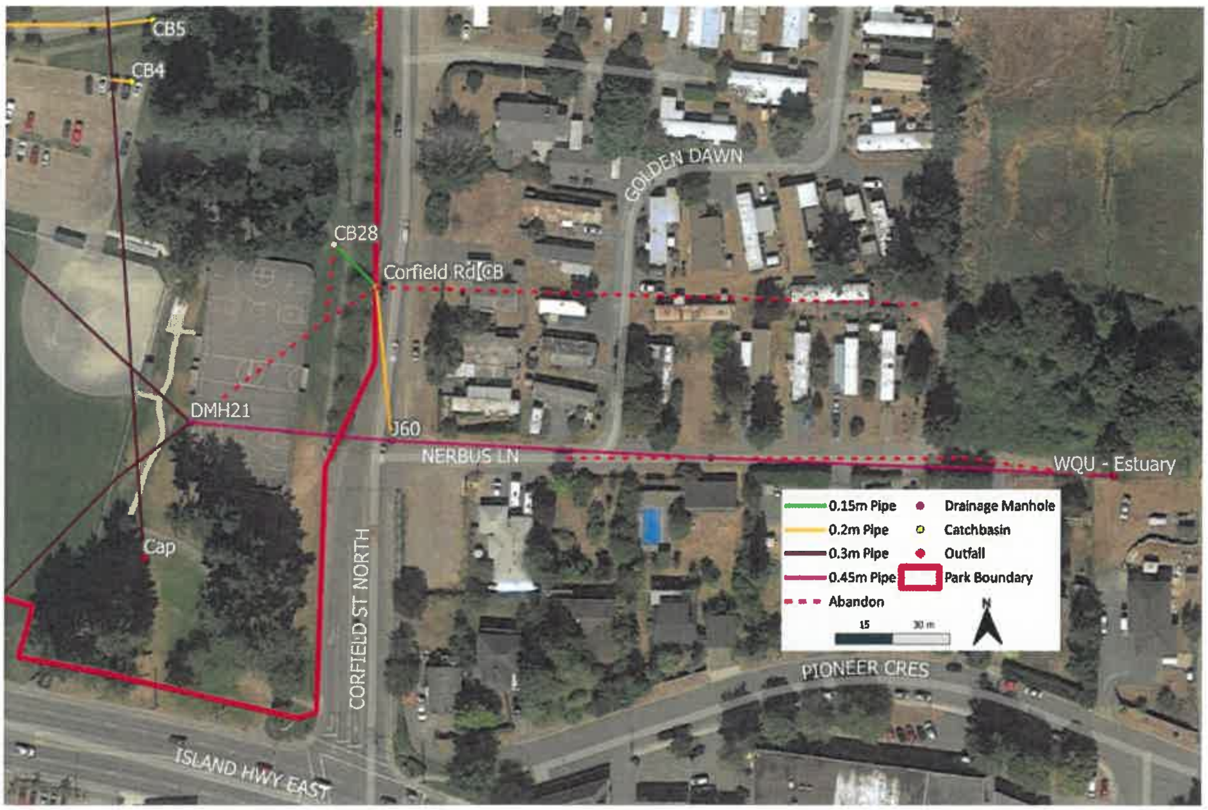

Figure 8. Nerbus Lane Realignment Concept Plan

Salish Sea Drive Bypass

In order to avoid disturbance to the valuable trees in the Arboretum, a bypass of the storm sewer sag located at the intersection of Salish Sea Drive and Sandcastle Drive is recommended. The bypass connects DMH39 in Sandcastle Drive, upstream of the sag, to CB44 downstream of the sag in Salish Sea Drive (-0.018%). The final catchbasin within the sag is diverted to the bypass and the bypass pipe size increases from 200mm to 450mm at that junction. This may be a blind connection or a new drainage manhole, as needed. The connection from Salish Sea Drive (CB44) to the main storm trunk (J8) requires upsizing to 450mm as well, to prevent ponding during intense rainfall events. While construction on the bypass and downstream storm pipe is occurring, the north section of Salish Sea

Drive road should be adjusted to ensure positive drainage to the catchbasins. System details are listed in Table 9.

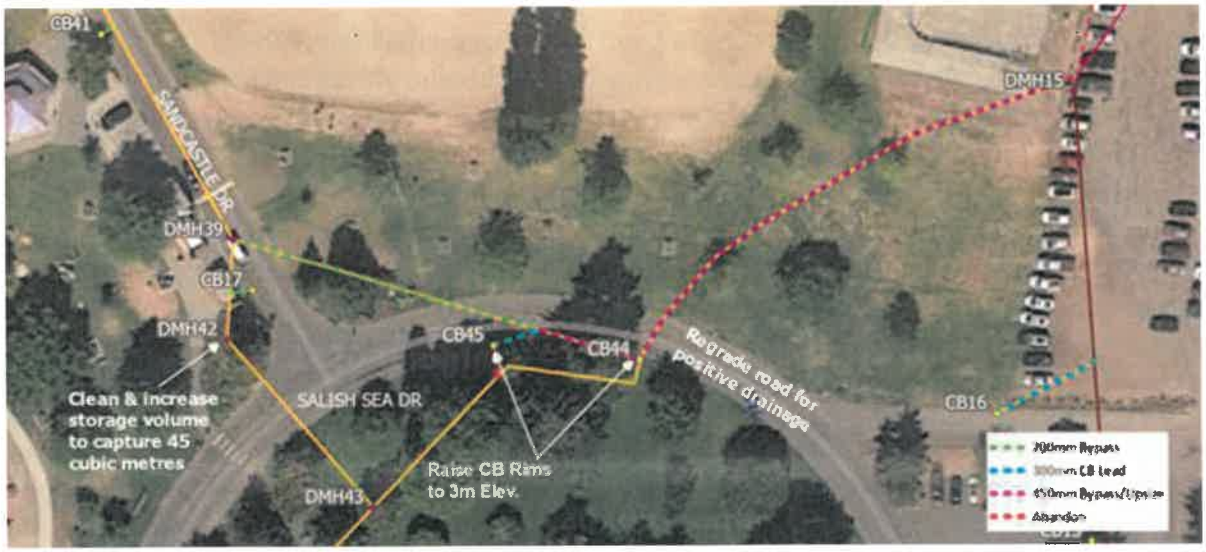

Figure 9. Salish Sea Drive Bypass Concept Plan

Dry Pond & Stub Connection

Diverting the existing storm trunk into the dry basin provides much needed water quality treatment prior to discharge into Parksville Bay. Increasing the storm trunk size to 450mm from J8 to the new dry basin inlet (J29), and again from the dry basin outlet (J59) to the ocean outfall stub (J102) meets Engineering design standards and ensures appropriate conveyance in the storm system during design events. Table 9 outlines the max velocities in these pipe segments during the 2100 10-year and 100-year 24 hour design events.

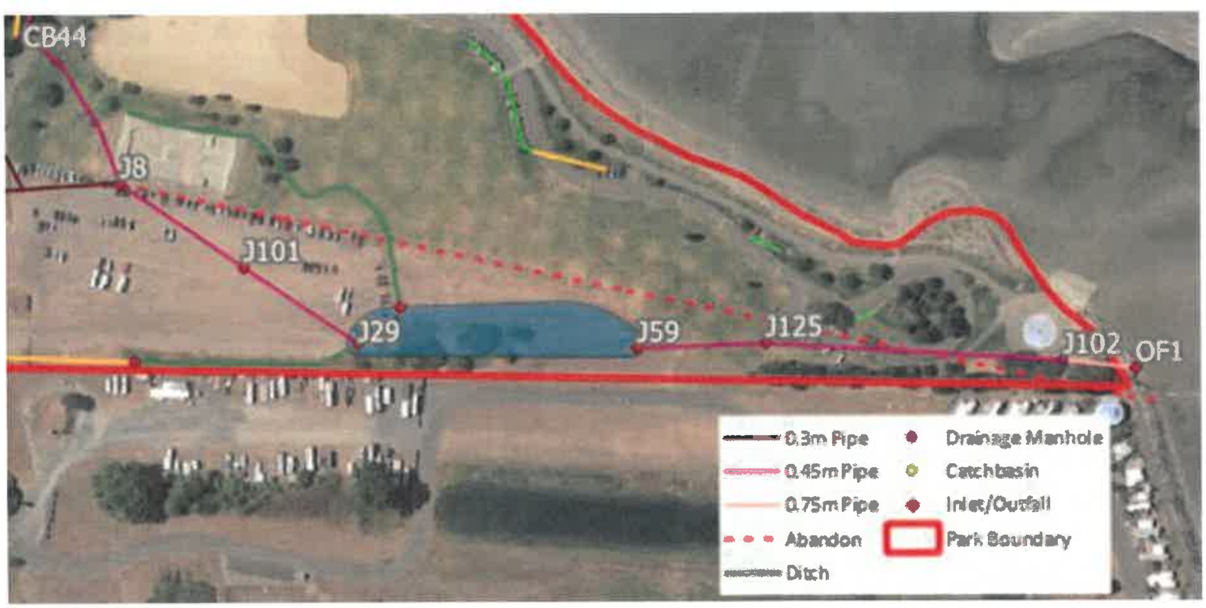

Figure 10. Dry Pond & Stub Connection Concept Plan

Table 9: Future storm sewer upgrades – Hydraulic conditions

Conduit	Inlet ID	Inlet Inv. (m)	Outlet ID	Outlet Inv. (m	2100 10-yr 24 hr design event Velocity (m/s)	2100 100-yr 24 hr design event Velocity (m/s)
Nerbus Lane	DMH21	2.22	J60	2.006	0.83	1.03
Salish Sea Dr Bypass	DMH39	1.32	CB44	2.522	0.37	0.54
Salish Sea Dr Main Trunk Connection	CB44	2.522	J8	2.128	0.64	0.74
Dry Pond Inlet	J101	1.879	J29	1.62	1.06	1.13
Dry Pond Outlet	J59	1.600	J125	1.506	0.84	0.857
Stub Connection	J102	1.14	OF1	0.966	0.27	0.344

Infiltration Galleries

Several catchments drain to rock pits, serving as infiltration areas. These structures were modeled as storage units with a constant volume and infiltration rate (mm/hr), as indicated in Table 10. New infiltration areas were modelled with zero seepage, as we lack the data to indicate a value and assuming zero is the more conservative

Table 10: Infiltration rock pits

Infiltration Rock Pit	Model ID	Volume (m^3)	Infiltration Rate (mm/hr)
Arbutus Point	SU6	60	40
Sandcastle Dr East	SU5	38.8	10
Sandcastle Dr Centre South	SU4	150	10
Sandcastle Dr West	SU3	89.2	10
Sandcastle Dr/Salish Sea Dr	SMH42	124.194	24
Tennis Court	Tennis_raingarden	35.45	0
Ravenhill Road	J20	100	0

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EOR water ecology community

- Drainage Manhole
- Area Drain
- Catchbasin
- Dry Basin Inlet/Outfall
- Dry Basin
- Infiltration Gallery
- Raingarden
- Opt-Storage/WQU
- Park Boundary
- 0.075m Pipe
- 0.1m Pipe
- 0.15m Pipe
- 0.2m Pipe
- 0.3m Pipe
- 0.45m Pipe
- 0.75m Pipe
- Abandon
- Ditch

Parksville Community Park
Stormwater Management Master Plan

Figure 8: Future Proposed Storm Sewer Network

N

0 75 150 m

5 Calibration

5.1 Method

The 1D portion of the existing conditions model was calibrated to runoff events observed during the month of January, 2020, with ground-truth data coming from level loggers installed by City staff in catchbasins DMH15, DMH43, DMH14, DMH3 and DMH21. Sea levels in Parksville Bay were represented using a time series of tidal data developed by Northwest Hydraulics Consultants. The monitoring locations are shown in Figure 7 to Figure 13.

The monitoring period did not capture any rainfalls greater than the 2-year recurrence interval, so it is difficult to know how well the calibration will serve for low-frequency events such as the 10- and 100-year storms. For similar accuracy in running higher intensity storms, future calibration to periods including such storms will be necessary.

Calibration was initially attempted using the Sensitivity - based Radio Tuning Calibration (SRTC) tool in PCSWMM to improve the goodness of fit between the computed and observed water levels. Uncertainties were set for the parameters in **Error! Reference source not found.** using the Uncertainty Estimator. The SRTC tool was then used to iteratively adjust the parameters with uncertainties. A computational grid was used to run SRTC tool scenarios in parallel and reduce the overall time required for calibration. Because of the low rainfall intensities during the observational period, the results were found to be almost completely insensitive to the calibration parameters and in the end the initial parameterization was deemed to provide an acceptable match to the observed data.

The infiltration rate at DMH42 was adjusted manually to provide a reasonable fit to the observed peak depths and drainage rate observed.

Table 11. Uncertainty of Subcatchment Parameters

Parameter	Uncertainty (%)
Flow length (m)	50
Subcatchment slope (%)	20
Depression Storage for Impervious Surfaces	100
Depression Storage for Pervious Surfaces	100
Green-Ampt Parameters: Conductivity	100
Green-Ampt Parameters: Suction Head	100

5.2 Results

The lack of large rainfall events made quantitative calibration difficult. We attempted to match the peak depths and depth hydrograph shape at each location for the largest events in the dataset; the large outlier event in the afternoon of January 18th could not be duplicated with the observed rainfall and is thought to be the result of outlet clogging, along with another smaller peak observed at DMH15 on January 26th. Outside of this event most locations saw only minimal depth (<25cm) throughout the calibration period, with the exception being DMH43, which is in a sag with an infiltration manhole. Because of the lack of large rainfalls, we could not effectively calibrate catchment runoff, as the rainfall intensity and soil moisture did not reach a point where pervious surface runoff would be expected.

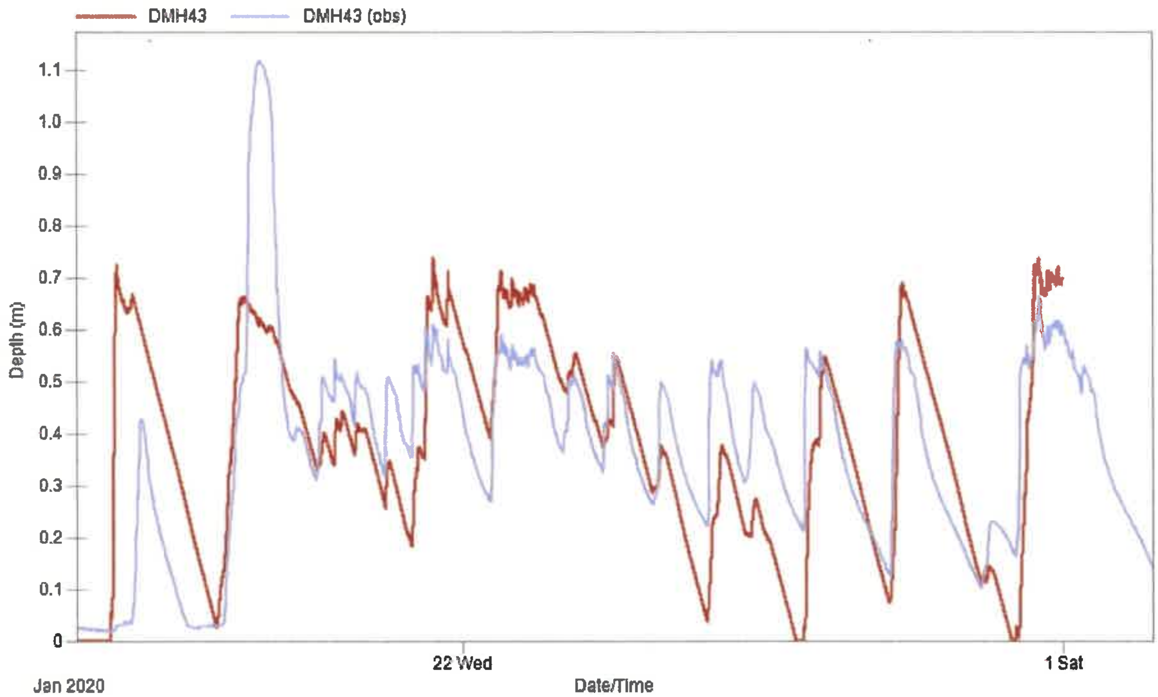

Figure 9. Waterlevel calibration at DMH43. Time period differs from other locations because the system response is significantly different.

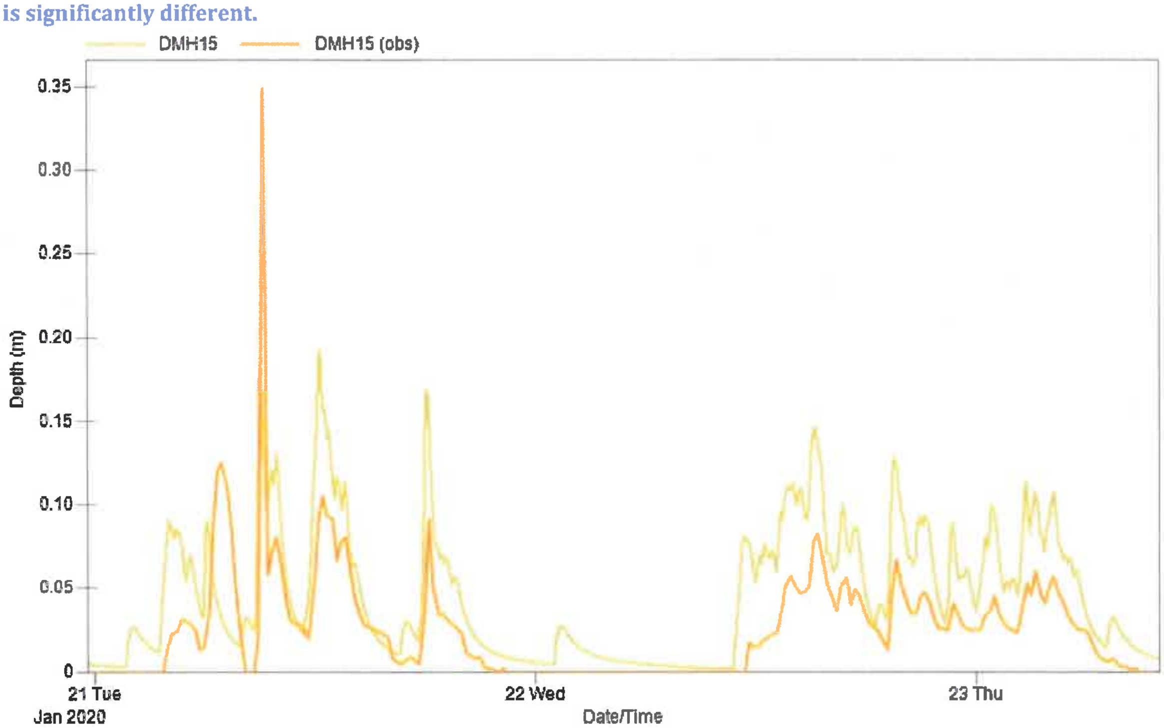

Figure 10. Waterlevel calibration at DMH43

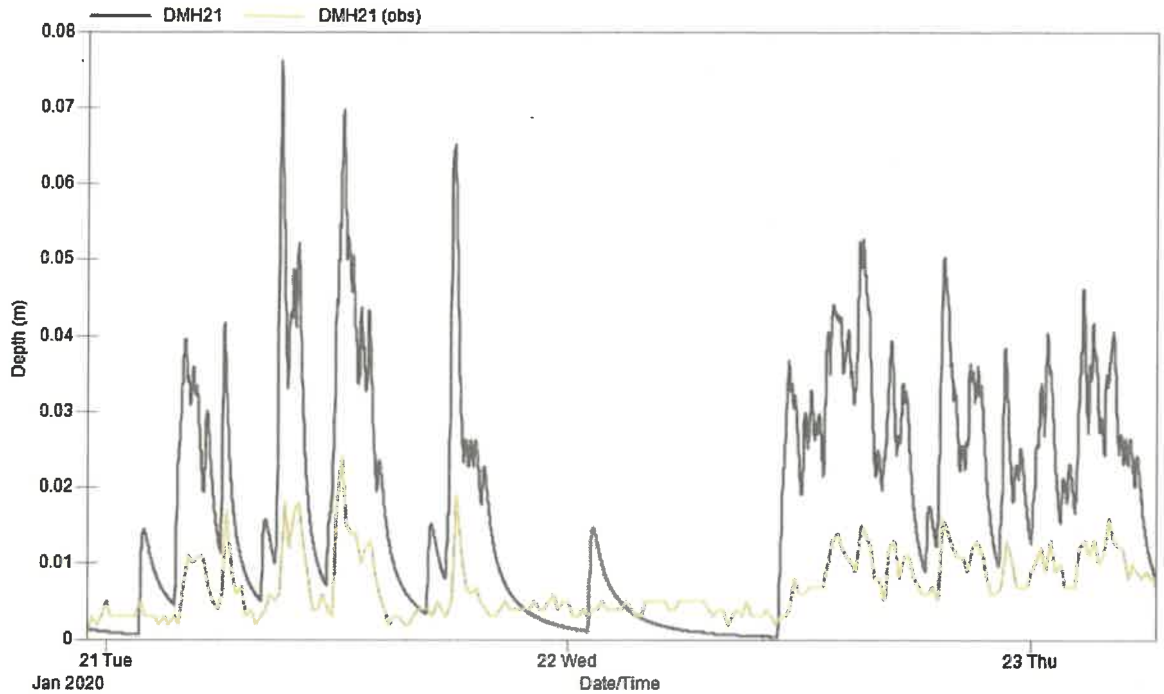

Figure 11. Waterlevel calibration at DMH21

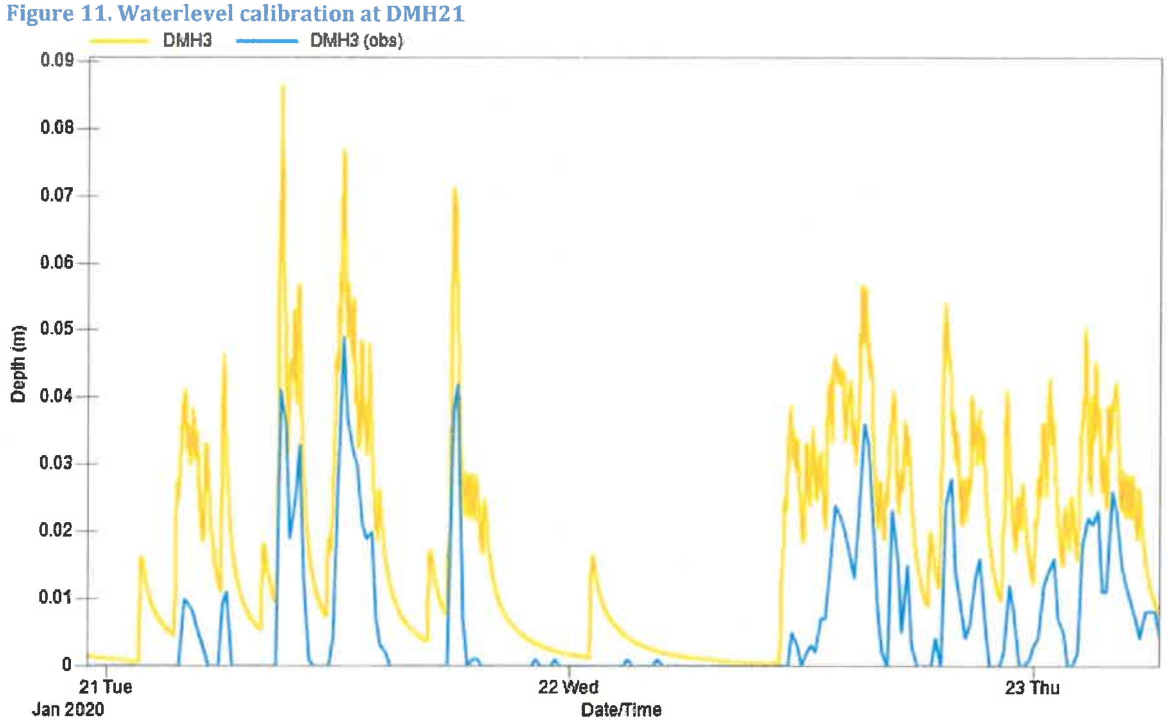

Figure 12. Waterlevel calibration at DMH 21

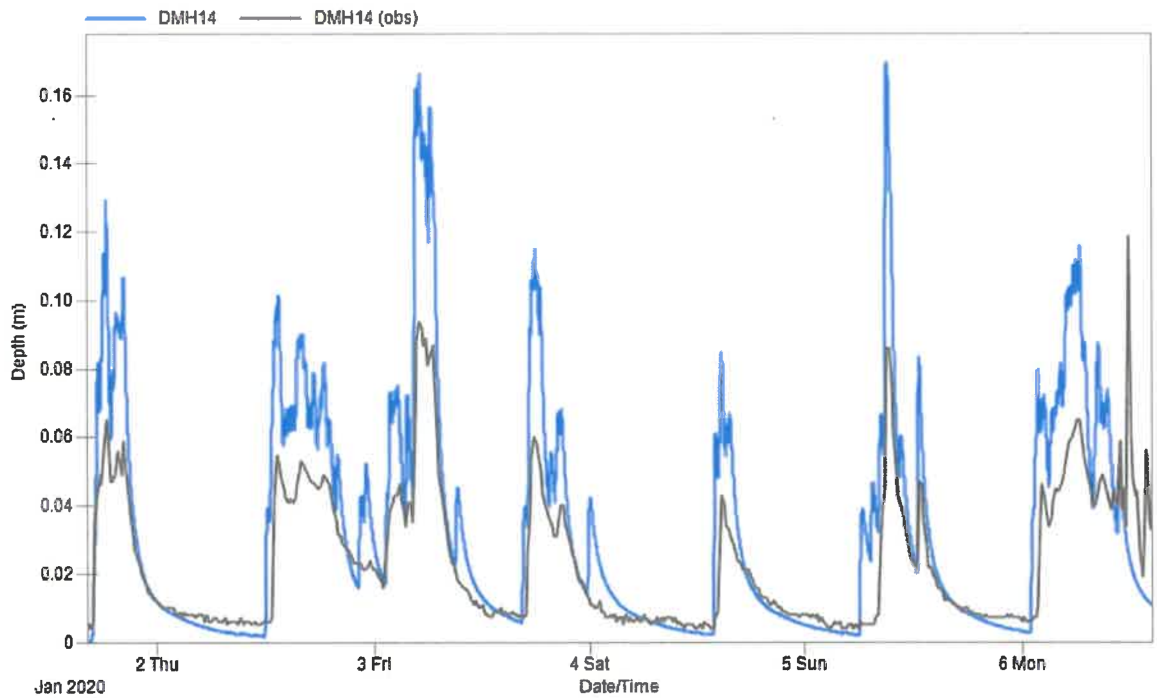

Figure 13. Waterlevel calibration at DMH 14. Displayed time period differs from other locations because of issues with monitoring data.

Because the model results showed little sensitivity to the selected calibration parameters, the decision was made to maintain the original model parameters, with the exception of suction head and conductivity which were decreased to make the model more conservative.

The 2D model was calibrated based on observed flood extents on the afternoon of January 29th, 2020. This calibration was carried out manually, and primarily consisted of adjusting infiltration parameters in areas of ponding and in the rock pits in order to reasonably match observed conditions.

5.3 Discussion

It is difficult to assess the quality of the 1D network calibration. Input parameters are set to reasonable values based on known soil parameters and site conditions, but the site generates little runoff from pervious surfaces during minor rainfalls like those that were captured during the monitoring period. Since the primary goal of the model is to assess flood risk, the lack of any large event during the monitoring period means we have not had the opportunity to test the sorts of events that result in overwhelming the storm sewer network.

The flooding on January 29th, however, demonstrated that limitations in the drainage network are not the only (or even the primary) flood source in the park. The 2D model was validated using photos and observations from a site visit on January 29, 2020. While the storm sewer network was able to cope with the water on that day, the rock pits were overwhelmed, and water backed up causing street flooding. The model does a reasonable job duplicating those results (see Photo 1 to Photo 6). The 2D model demonstrates that the primary shortcomings of the system are in infiltration, storage capacity and overland drainage.

We believe the model to be sound and well-designed, however we recommend continuing to gather observational data so improvements can be made when the opportunity arises.

Figure 14. Locations of validation photos taken January 29, 2020

Photo 1: North Parking Lot

Photo 2: Arbutus Point Parking Lot

Photo 3: Dry Pond

Photo 4: Tennis Court

Photo 5: Ball Diamond

Photo 6: Ravenhill Road

6 Design Events

6.1 Performance Objectives

The key objectives for performance of the park's stormwater management system include the following:

1. Prevent nuisance flooding during the late-century 10-year 24-hour rainfall event, considering the late-century astronomical tide as a potential constraint to sea outfall capacity.
2. Mitigate flood risk during extreme rainfall and coastal inundation events to acceptable levels of risk, such as allowing up to 0.15 m of flooding in roads and parking lots or temporarily closing areas where permanent flood mitigation is cost prohibitive.
3. Mitigate non-point source pollution impacts to receiving waters and their ecosystems by capturing and treating the first flush event (31 mm 24-hour event).
4. Offset drinking water demand to the extent feasible by using stormwater as a resource (i.e. for irrigation) in a cost-effective manner.
5. Be resilient to coastal inundation within the park, such as excessive erosion from wave action, debris, and saltwater.
6. Support future use and development of the park, considering future increases in imperviousness
7. Support other goals of the SWMMP by incorporating design elements that enhance public awareness of flood risk/climate change adaptation, enable cost-effective operation and maintenance, and connect the public with First Nations.

6.2 Sizing Criteria

- Water quality treatment provided for the first flush event (31 mm, 24-hour event). These facilities must drain within 48 hours of the event to support vegetation and provide capacity for future events.
- Storage and conveyance capacity in the system provided to prevent surface flooding during the 10-year 24-hour late century rainfall event. Discharge to the sea outfall must consider limited outlet capacity due to late-century astronomical tides and potential clogging from sediment.
- Assess vulnerability of the system and provide temporary ponding / emergency procedures for extreme rainfall and coastal inundation conditions, including:
    - Late-century 10-year rainfall during 10-year coastal inundation, which has a combined 100-year annual exceedance probability assuming the two are independent
    - Drainage of late century 10-year and 100-year coastal inundation across the park

6.3 Scenarios

Individual design storm events for the 10-year and 100-year return period events. The model was run using the 1-hour AES BC Coast and the 24-hour SCS Type IA (Pacific Coast) distributions and the IDF curves outlined in Section 4.2.

Sea level at the Parksville Bay storm sewer and 2D mesh outfalls were defined based on a time series of sea water levels developed by NHC, which were based on measured levels at Point Atkinson that were transformed to reflect conditions at the project site. The time series includes the measured astronomical tide as well as residuals from storm surge and wind/wave set-up (Northwest Hydraulics Consultants, 2020b). The simulation period was selected so that the sea level ranged from approximately the Lower Low Water Mean Tide to the Higher High Water Mean Tide (Table 12), as highlighted in Figure 15. The simulation was started at a time such that the peak sea level on the first day roughly corresponded with peak rainfall intensity.

Table 12. Summary of Tides based on Northwest Bay (Northwest Hydraulics Consultants, 2020a)

Sea State	Year 2020 Tide Elevation (m, CGVD2013)
Higher High Water Large Tide (HHWLT)	2.18
Higher High Water Mean Tide (HHWMT)	1.68
Mean Water Level (MWL)	0.18
Lower Low Water Mean Tide (LLWMT)	-1.73
Lower Low Water Large Tide (LLWLT)	-2.83

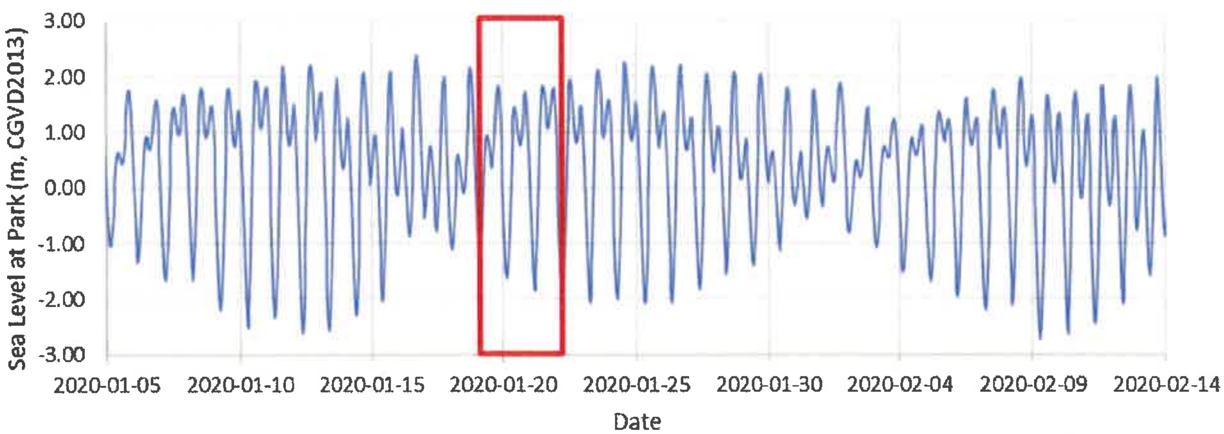

Figure 15. Sea Level at Park (Northwest Hydraulics Consultants, 2020b)

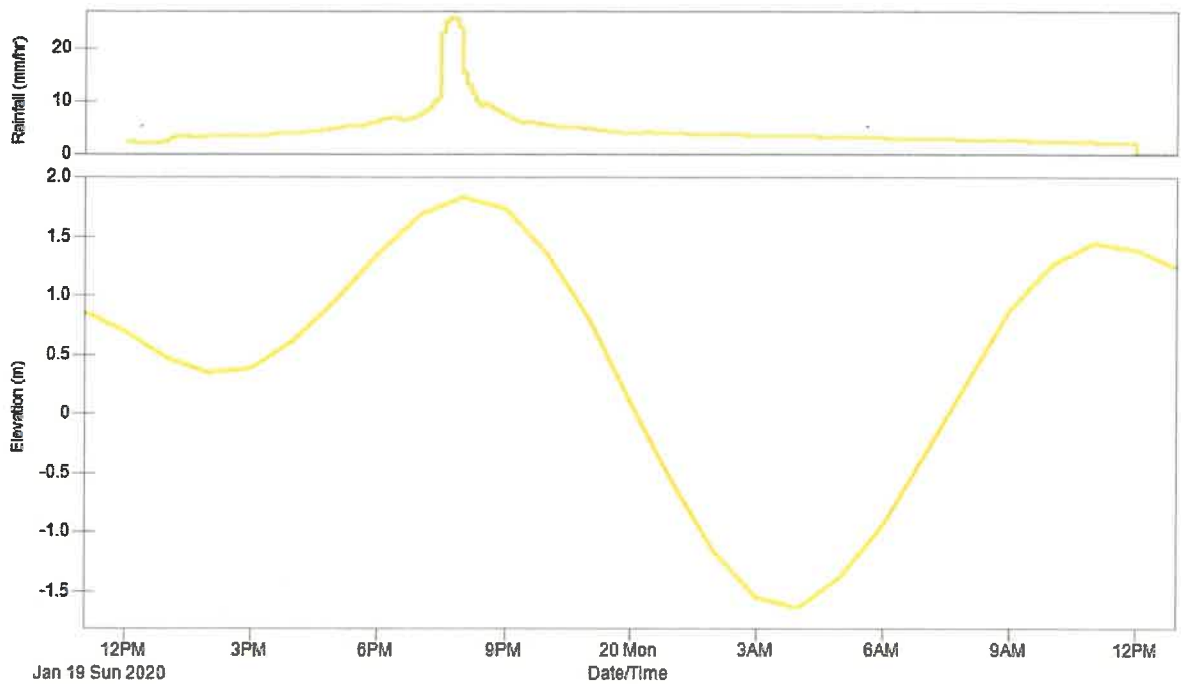

Figure 16. Sea level (yellow) and 100-yr 24-hr SCS Type 1A rainfall intensity (green)

The future conditions water level at the outfall to Parksville Bay was defined by adding 0.79m to the September 2019-May 2020 (CGVD2013) tidal time series data to reflect expected sea level rise countered by land uprising by 2100 (Northwest Hydraulics Consultants, 2020a). Design storm hyetographs were timed to coincide with a normal high tide cresting at 1.835m at 8:00pm on January 19th. Discharge of coastal inundation during the 2100 100-year AEP event was simulated by setting the inundation water depth (3.805m) as the initial 2D model condition to confirm that the stormwater management system is capable of discharging inundation to the park. Discharge time is limited by tidal influences on the outfall, located below the high tide elevation.

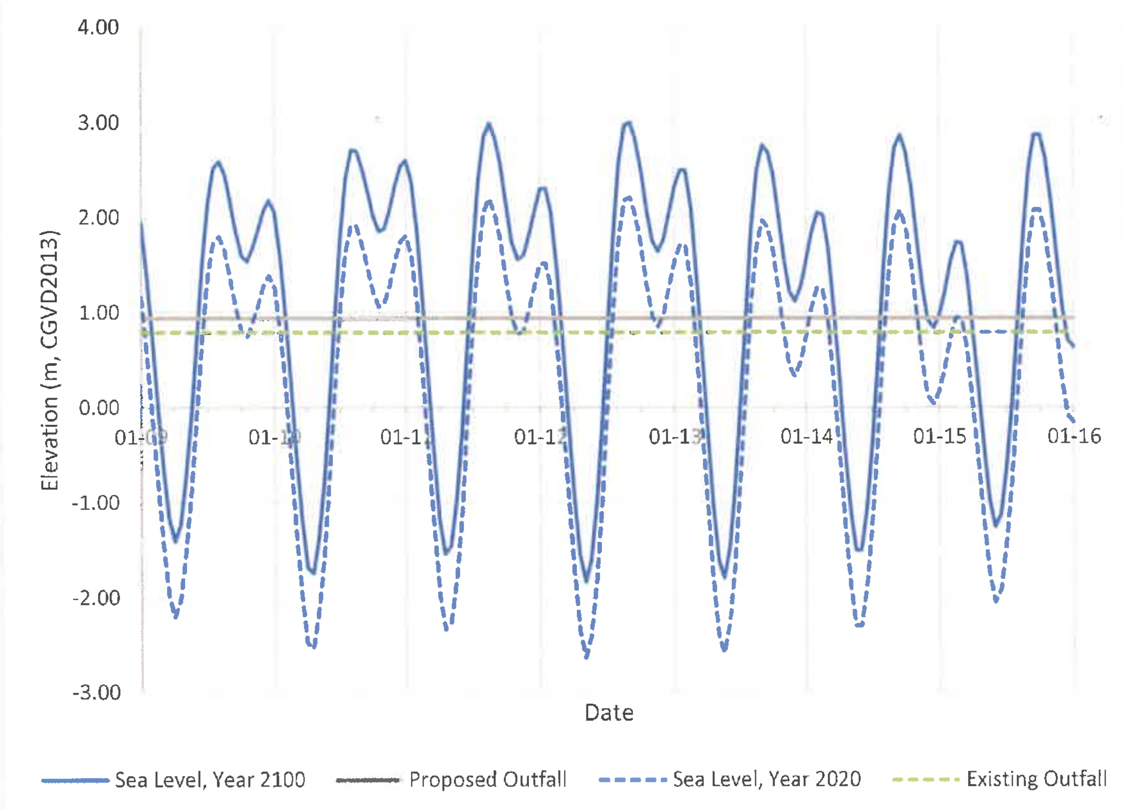

Figure 17. Tidal timeseries for current and future conditions

6.4 Existing Conditions Model Results

Key model outputs include the pipe segments with exceeded capacity, the areal extent and depth of surface flooding, the number of buildings flooded, and the number of inlets with exceeded capacity. The results are summarized in Table 13 and Figure 18 to Figure 21 The results indicate that the existing storm sewers have sufficient capacity during the 10-year event, however some locations have limited inlet capacity or lack positive grades to inlets. There is insufficient sewer and overland flow capacity to prevent flooding during the 100-year event, which should be limited to a maximum depth of 0.15 m in roads and trails.

Table 13. Peak Flow and Runoff Volume

Point of Interest	10 year-1 hr AES BC Coast		100 year-1 hr AES BC Coast		10 year-24 hr SCS Type IA		100 year-24 hr SCS Type IA	
	Peak Flow (m^3/s)	Runoff Volume (ML)	Peak Flow (m^3/s)	Runoff Volume (ML)	Peak Flow (m^3/s)	Runoff Volume (ML)	Peak Flow (m^3/s)	Runoff Volume (ML)
Outfall to Parksville Bay	0.065	0.195	0.078	0.346	0.056	1.629	0.062	2.304
Outfall to Estuary	0.006	0.015	0.016	0.036	0.014	0.233	0.03	0.374

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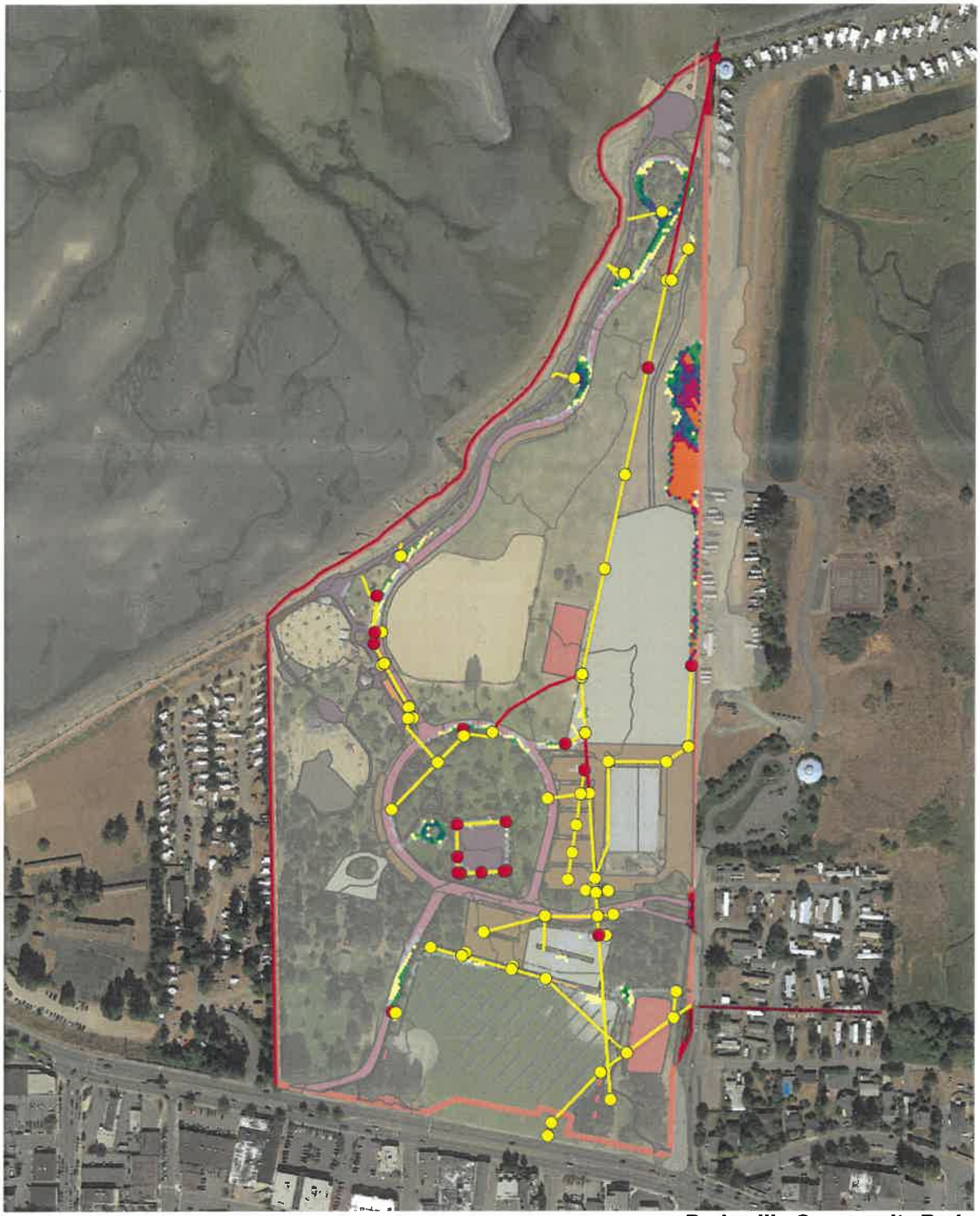


EOR water ecology community

Surface Ponding Depth
- 1-5cm
- 5-10cm
- 10-15cm
- 15-20cm
- 20-25cm
- 25-30cm
- 30-40cm
- >40cm

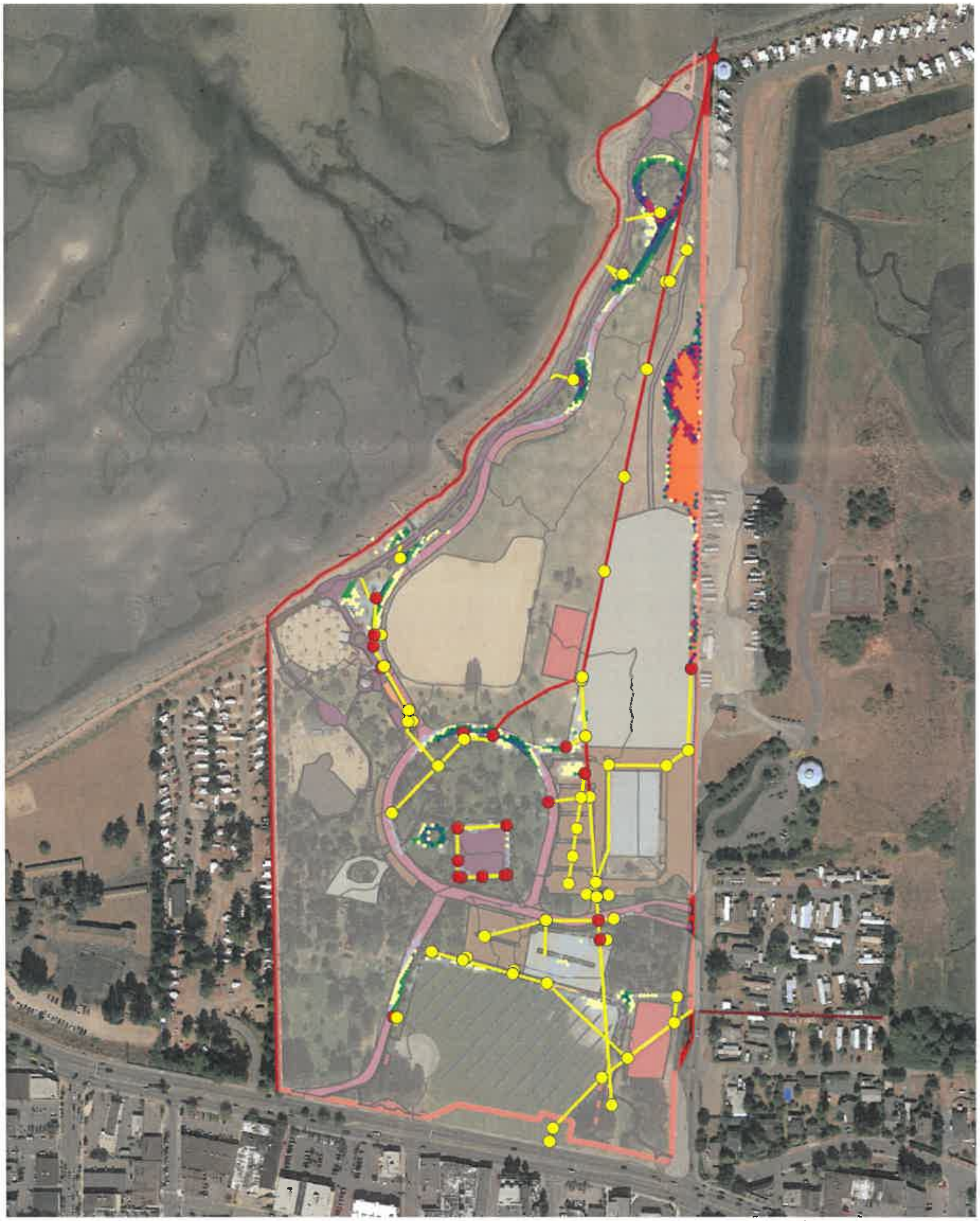
Storm Sewer Pipes
- Capacity Limited
- Capacity Not Limited

Catchbasins/Manholes
- Flooded
- Not Flooded

Parksville Community Park
Stormwater Mgmt Master Plan

Figure 18: Model Results for 2020 10-yr 24hr SCS Type 1A Rainfall Event

N
0 75 150 m



EOR water ecology community

Parksville Community Park
Stormwater Mgmt Master Plan

Figure 19: Model Results for 2020 100-yr 24hr SCS Type 1A Rainfall Event

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EOR water ecology community

Surface Ponding Depth
- 1-5cm
- 5-10cm
- 10-15cm
- 15-20cm
- 20-25cm
- 25-30cm
- 30-40cm
- >40cm

Storm Sewer Pipes
- Capacity Limited
- Capacity Not Limited

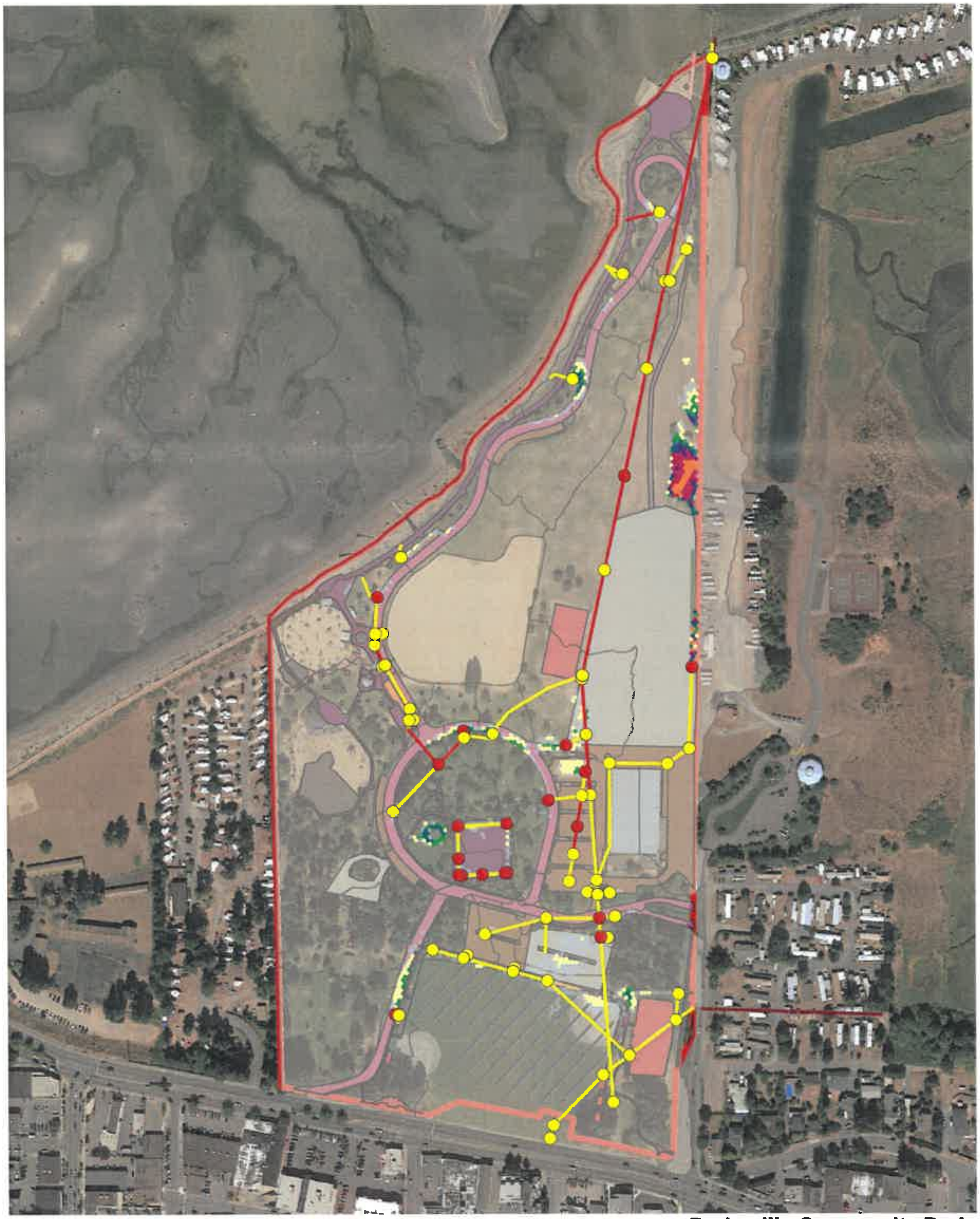
Catchbasins/Manholes
- Flooded
- Not Flooded

Parksville Community Park Stormwater Mgmt Master Plan

Figure 20: Model Results for 2020 10-yr 1hr AES Rainfall Event

N

0 75 150 m



Parksville Community Park
Stormwater Mgmt Master Plan

Figure 21: Model Results for 2020 100-yr 1hr AES Rainfall Event

EOR water ecology community

Surface Ponding Depth
- 1-5cm
- 5-10cm
- 10-15cm
- 15-20cm
- 20-25cm
- 25-30cm
- 30-40cm
- >40cm

Storm Sewer Pipes
- Capacity Limited
- Capacity Not Limited

Catchbasins/Manholes
- Flooded
- Not Flooded

N 0 75 150 m

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6.5 Future Conditions Model Results

The site discharge flow and volumes under future conditions are summarized in Table 13. Figure 22 and Figure 23 identify the pipe segments with exceeded capacity, the areal extent and depth of surface flooding, the number of buildings flooded, and the number of inlets with exceeded capacity. The results indicate that the recommended changes to the future storm sewer system will have sufficient capacity during the 10-year event to prevent nuisance ponding deeper than 6cm. Overland flow capacity and sewer conveyance following implementation of the recommended stormwater management upgrades will prevent flooding in excess of 150mm depth on roads and trails during the 100-year event. Discharge of the 2100 coastal inundation events can be managed by the stormwater management system, however additional maintenance to remove sediment and debris deposited during these events is critical to ensuring function and longevity of the stormwater management system.

Table 14. Future Condition Peak Flow and Runoff Volume

Point of Interest	2100 10 year-24 hr SCS Type IA		2100 100 year-24 hr SCS Type IA	
	Peak Flow (m^3/s)	Runoff Volume (ML)	Peak Flow (m^3/s)	Runoff Volume (ML)
Outfall to Parksville Bay	0.123	3.208	0.168	4.782
Outfall to Estuary	0.06	1.039	0.114	1.595

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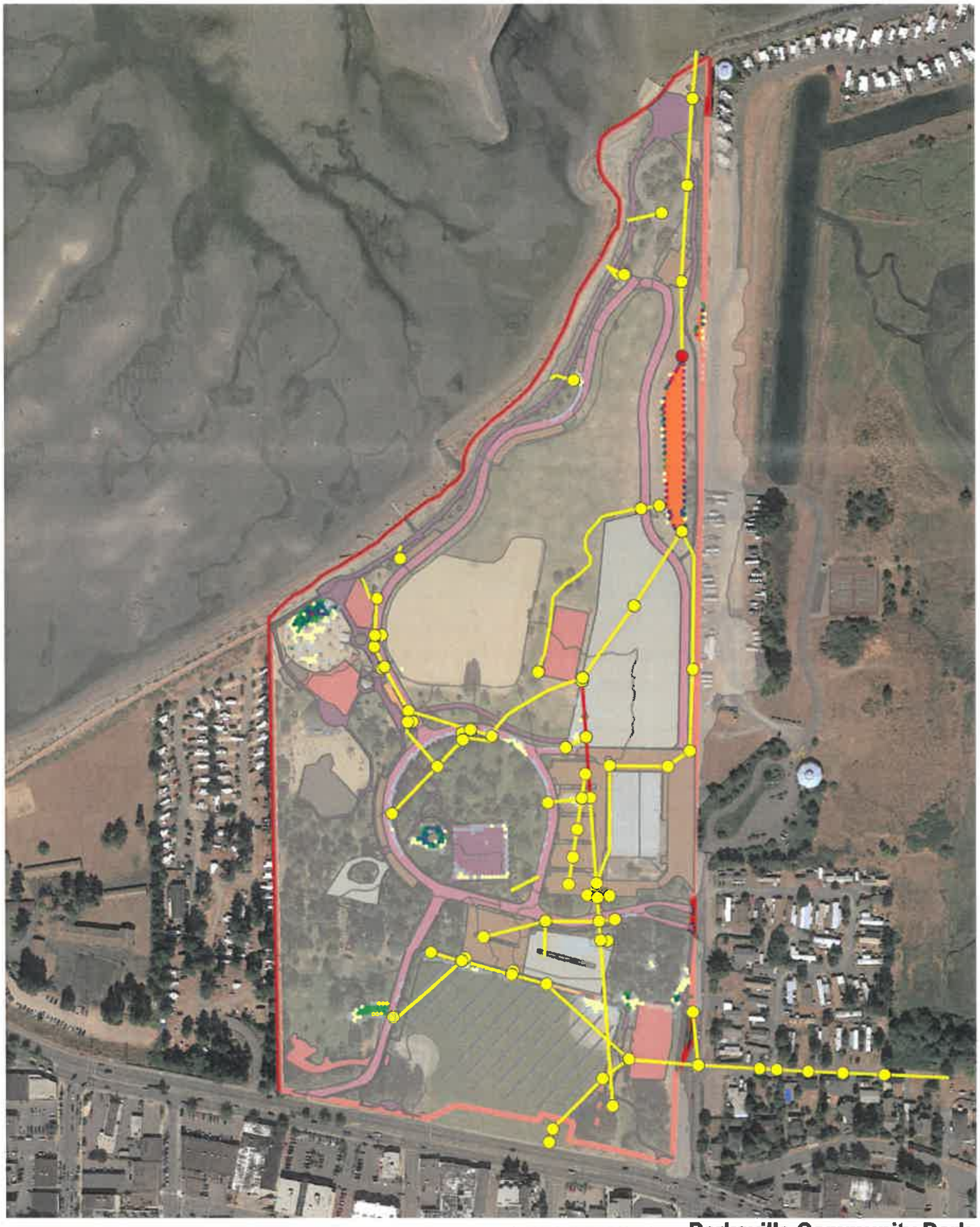


EOR water ecology community

Surface Ponding Depth
- 1-5cm
- 5-10cm
- 10-15cm
- 15-20cm
- 20-25cm
- 25-30cm
- 30-40cm
- >40cm

Storm Sewer Pipes
- Capacity Limited
- Capacity Not Limited

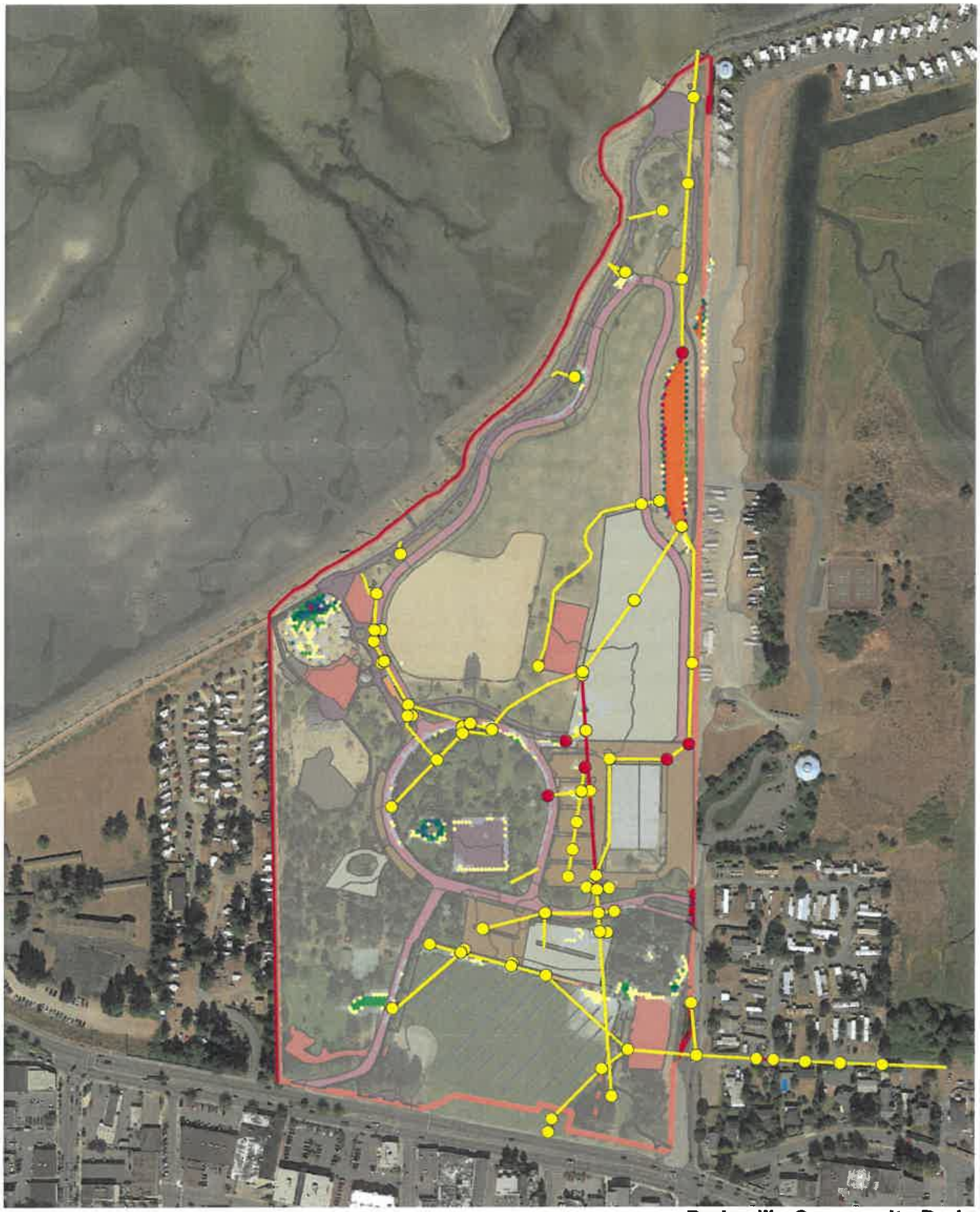
Catchbasins/Manholes
- Flooded
- Not Flooded

Parksville Community Park Stormwater Mgmt Master Plan

Figure 22: Model Results for 2100 10-yr 24hr SCS Type 1A Pacific Coast Rainfall Event

N

0 75 150 m



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EOR
water
ecology
community

Surface Ponding Depth
- 1-5cm
- 5-10cm
- 10-15cm
- 15-20cm
- 20-25cm
- 25-30cm
- 30-40cm
- >40cm

Storm Sewer Pipes
- Capacity Limited
- Capacity Not Limited

Catchbasins/Manholes
- Flooded
- Not Flooded

Parksville Community Park Stormwater Mgmt Master Plan

Figure 23: Model Results for 2100 100-yr 24hr SCS Type 1A Pacific Coast Rainfall Event

N
0 75 150 m

6.6 Discussion of Existing Conditions

Surface flooding in the Park appears to be dominated by issues with surface drainage, storage, and infiltration, rather than by a lack of capacity in the subsurface storm sewer network. The exception to this is flooding that appears to be caused by the known issue of outlet clogging, which is not directly addressed by this modeling. This appears to be true even under relatively high sea levels, meaning that at the current moment it does not appear that sea level is a driver of flooding.

The existing rock pits are not well characterized and our understanding of their functioning is poor. They were generally sized in the model to roughly match the observed ponding observed along the roadway on January 29, 2020. With only a single observation date, there are multiple model geometries that could provide a solution, so significant uncertainty remains about how these structures would function under other rainfall conditions. The model also cannot account for the variations in infiltration that appear to be driven by tidal conditions. It is clear, however, that the existing facilities are undersized relative to observed rainfall, and that they lack a robust overflow option.

Many areas of problem flooding within the Park can be fixed through improved grading and carefully sized and located storage and infiltration facilities. The model as designed will provide robust estimates of flood volume at the various problem areas. Allowing for sufficient storage to hold the 10-year 24-hour return period storm, and surface drainage to pass the 100-year 24-hour return period event, should alleviate much of the surface flooding in those areas. In the remaining areas, while the subsurface drainage network exceeds its nominal capacity during the design storms, it does not flood at the surface; this suggests that a limited amount of additional drainage could be routed through the system, particularly if the peak were attenuated and delayed through storage prior to discharge into the storm sewer.

As would be expected, the more intense rainfalls of the 1-hour AES design events stressed the subsurface network more than the 24-hour SCS events did; however this did not translate to more surface flooding, which is driven more by volume than intensity in this system. Since the ultimate issue in the Park is surface flooding, rather than pipe capacity, we have chosen to use the 24-hour SCS Type 1A storms as the design standard.

6.7 Discussion of Future Conditions

Sea level increases will decrease the timeframe available to discharge detained stormwater resulting in extended discharge periods. Without raising the outfall above the normal high-tide elevation this condition cannot be alleviated. Integrating the dry pond into the stormwater management infrastructure will alleviate roadway flooding while still allowing park users to observe the impacts of the coastal setting and climate changes on the Park in a safe manner.

Implementation of the recommended stormwater management upgrades, in terms of additional volumes, upgraded conveyance networks and site grading to facilitate positive drainage, is expected prevent surface flooding in excess of 0.15m in the park, based on model results, during the 2100 10-year 24-hour SCS Type 1A design storm. Infiltration facilities are critical elements of the stormwater management system and must be designed, constructed and maintained to ensure full function is available during storm events. Modelling shows that the proposed drainage system will manage stormwater runoff under predicted future conditions according to the design criteria.

The future proposed stormwater management system was modelled with a blocked ocean outfall for the 2100 10-year 24-hour SCS Type 1A storm event (98mm), representing an extremely conservative scenario where the only outflows are via the estuary outfall and infiltration through defined infiltration galleries. The proposed system was able to store all the water however ponding in some areas without positive surface drainage to the storm sewer system exceeded the arbitrary depth target of six centimetres. Providing proper grading to catchbasins and overland flow pathways, along with implementation of all recommended storm sewer upgrades, would alleviate nuisance ponding even in this most conservative scenario.

Under the predicted 2100 100-year 24-hour SCS Type 1A storm event (134.2mm), the system storage volume is insufficient to store the entire event onsite if the outlet is completed blocked for the duration of the storm, and this condition would result in significant flooding and overland flow from the dry basin to the estuary east of the site. A conservative scenario modelled the 2100 100-year 24-hour design event with an assumed static outfall water level of 1.68 m (CGVD2013), matching the elevation at the top of the existing outfall stub. Tidal fluctuations adjusted for the 2100 mean sea level and land uplifting will result in 14 hours per day with sea level elevations at or below 1.68m, limiting the period available for stormwater to discharge from the park. In the modelled scenario the system shows some isolated areas of flooding exceeding 0.15m in depth resulting from depressions in the landscape which can be eliminated through improved grading, as identified in the Stormwater Management Master Plan.

The model predicted that discharge of the future 100-year coastal inundation scenario draining freely (with an assumed fixed outflow elevation of 1.68m) through the future stormwater management system, would be able to clear the streets within 8 hours, and approaches fully drained in less than 24 hours. While tidal inundation cannot be practically prevented, the proposed system will be able to fully drain these waters as the tide recedes.

Respectfully submitted,

Emmons & Olivier Resources Canada Inc.

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Emmons & Olivier Resources Canada Inc.

Evan Murdock
Stormwater Modeling Specialist
Emmons & Olivier Resources Inc.

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