Appendix J

Drainage Report

Report

Preliminary Drainage Assessment

H362376-PM-230-S0-0001

2021-08-03	B	Client Review	K. Perera / D. Lavigilante	M. Riddoch	I. Mckenna	
DATE	REV.	STATUS	PREPARED BY	CHECKED BY	APPROVED BY	APPROVED BY
				Discipline Lead	Functional Manager	Not Required

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1. Introduction and Objective

Hatch was retained by Canadian Pacific Railway (CP) to provide design engineering services for the proposed track expansion project called East L Yard Expansion – L30/31, including the design of associated stormwater management assets and infrastructure.

This report documents preliminary drainage assessment for the proposed development. The objectives of this report include:

- To determine the impact of the proposed track expansion based on hydrology and hydraulic conditions.
- To design an adequate drainage system to accommodate minor storm runoff and an overland route to accommodate larger storm events.

The scope of this assessment does not include assessing hydrologic or hydraulic conditions for storm sewer or overland route system outside of CP's ROW.

2. Project Overview

This project expands the South Shore East L Yard between Mi. 125.34 and 126.93 in Vancouver, BC and the proposed works include:

- New 1,585m extension to the L30 track;
- New 1,795m extension to the L31 track;
- New 500m extension to the K10 track;
- K01, K02, K09 and K10 track shifts east of Commissioner St. overpass;
- Five new or relocated crossovers; and
- One modified at-grade crossing for the Columbia Containers Lead Track.

The intent is to construct two new yard tracks namely L30 and L31 between the South Shore East L Yard existing tracks and Commissioner Street. The existing rail corridor will be widened to the north, towards Commissioner Street, to accommodate for the horizontal alignments of the new tracks. The Commissioner Street road realignment works, as well as the associated road and utility infrastructure upgrades, have been undertaken by the Vancouver Fraser Port Authority (VFPA) to clear space for the new tracks. The extent of the new rail corridor where the proposed tracks are to be constructed is approximately 8.5 ha and bounded by the south limit of the proposed Commissioner Street to the north and privately-owned properties to the south. The site location is illustrated in Figure 2-1.

Figure 2-1: Site Location

3. Hydrotechnical Methodology

The following engineering guidelines, track standards, design manuals, references and codes were used in preparing this preliminary drainage assessment:

- CP Engineering Guidelines for Private Siding Design and Construction (2017)
- CP Industrial Track Standards
- "Manual of Recommended Practice" of the American Railway Engineering Association (AREMA)
- BC Ministry of Transportation (MoT) Hydrotechnical Engineering Design and Deliverable Guideline (2013)
- MMCD Design Guidelines 2014, by the Master Municipal Construction Documents Association
- Engineering Design Manual 2019 by the City of Vancouver.

The above documents do not provide specific guidance on the hydrotechnical engineering design process for rail yards. The above-mentioned references, previous similar studies and current industry trends informed the hydrotechnical methodology adopted for this study. The preliminary drainage assessment excludes assessment of water quality and climate change impacts. The main tasks for this hydrotechnical methodology include:

- Development of a hydrological model for the project based on a hydrograph method using PCSWMM modelling software; This includes the following processes:
  - Incorporating LiDAR and other survey data (where appropriate) to create a Digital Terrain Model
  - Schematics of the VFPA and CoV drainage networks
  - Development of the existing and post-development runoff hydrographs for the hydraulic model.
- Compilation of runoff hydrographs for the full range of design storms distribution, AES (City of Vancouver), Chicago and SCS Type II and durations 1, 3, 6,12 and 24 hours for 2-, 5- and 100-year storm events under the pre and post development conditions
- Assessment and identification of the peak design storm event which will provide highest impacts for the existing and post development conditions
- 1D hydraulic modelling for the existing and post development storm sewer systems to determine pipe sizes, surcharge and freeboard for the peak design storm event while also accounting for the effect of climate change.

- Identification and assessment of infrastructure assets and properties at risk of surcharge and freeboard
- Preliminary mitigation assessments.

4. Existing Conditions

The rail corridor is mainly comprised of rail, road and utility infrastructure. The project area is more than 80% impervious as defined in modeling software; the impermeable surfaces comprise of rail tracks, gravel and road pavement. Note that the ballast is considered as a free draining material while the subballast located underneath, is made of compacted aggregate material and is hence considered as impervious. Asphalt is more impervious than subballast and is defined as “zero-impervious” in modeling scenarios. Less than 20% of the area is defined as pervious area, comprising of grassed and vegetated surfaces mainly located south side of the existing track. General site conditions are shown in Figure 4-1.

The existing VFPA storm sewer network under Commissioner St is the main stormwater conveyance in and around the project area that ultimately drains to the Burrard inlet. A Metro Vancouver combined sewer runs under Commissioner St and parallel to the VFPA storm sewer, however no catch basins or culverts were recorded to connect to this sewer. The rail yard was originally constructed with a ditch on the south side of the CP main track but over time, erosion and track lifts have led to infilling of this ditch from the slope above, however the south track still drains adequately. No significant watercourse or functioning drainage culverts were located within the rail corridor. A culvert headwall was located east of Slocan on the south side of the CP main track, however the culvert could not be found.

The existing top of subballast is above the southside curb of Commissioner Street for more than 90% of the project length. Areas where the subballast is below the roadway are located at the east end of the project. The majority of the rail corridor slopes 1.0% towards Commissioner Street with a longitudinal slope from west to east varying between 0.5% to 2.0%. The existing rail tracks are either stepped-down towards Commissioner St, or occasionally at the same elevations to accommodate turnouts and crossovers.

The existing soil stratification identified in the geotechnical report states the top 2m of fill through the rail corridor is granular. Geotechnical recommendations suggest this material is free draining. A site visit during a December rain event showed negligible ponding except over flat, compacted subballast.

The surface water runoff from the existing tracks flows into a swale or a road gutter. There are catch basins within the rail corridor, however, these are mostly ineffective as the existing ground does not properly grade towards the basins as shown in Figure 4-1.

Figure 4-1

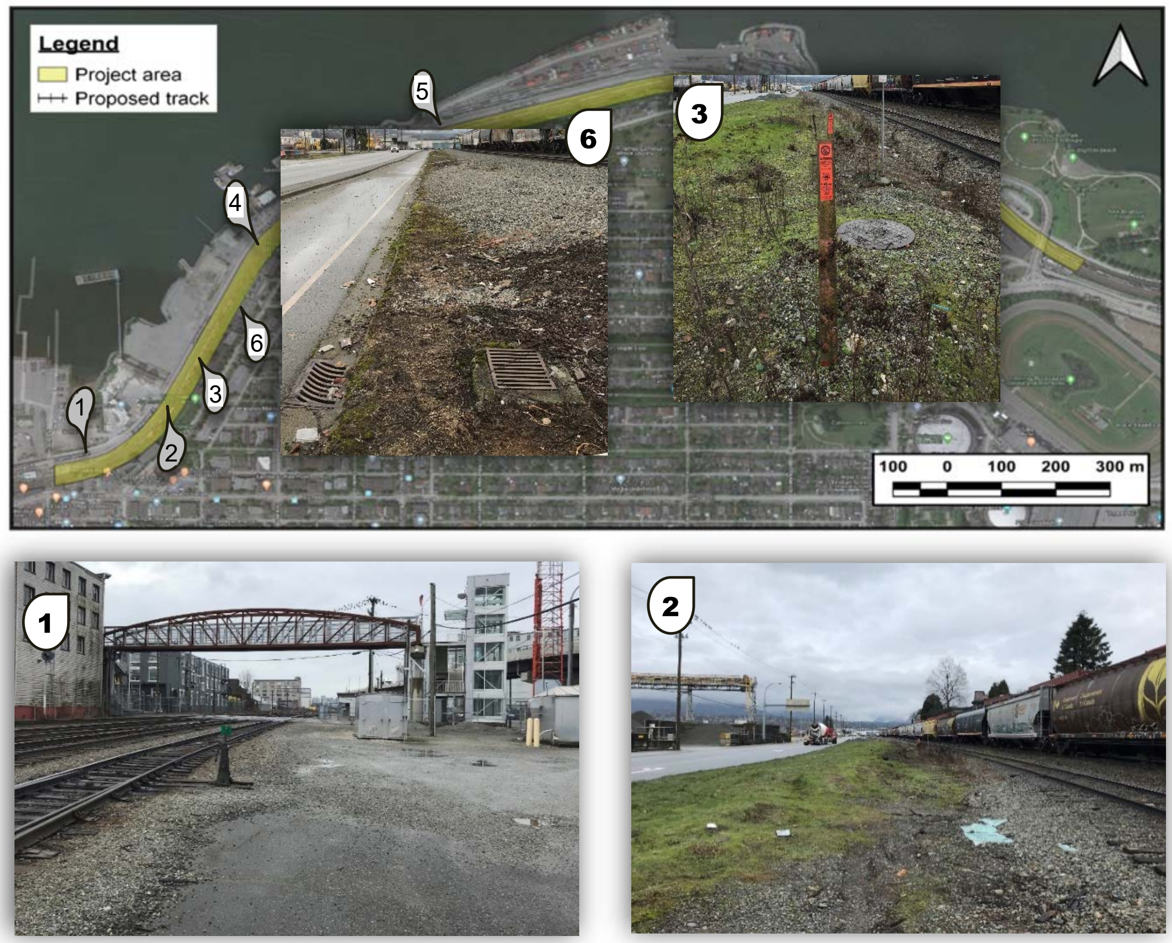

Legend
Project area
Proposed track
1
2
3
4
5
6
100 0 100 200 300 m

Figure 4-1: General Site Appearance

Any surcharging within the yard is currently being handled with standard track maintenance.

Figure 4-2 illustrates key outlets near the project discharging directly into the Burrard inlet. These outlets are located within the VFPA property and under the Federal Port jurisdiction.

Figure 4-2: Key outlets near the project discharge directly into the Burrard inlet

4.1 Existing Conditions Limitations

The scope and focus of this report are on the impact of the track expansion works and that analysis is limited to the catchments within the CP ROW. Limited information is available on:

- The size and characteristics of external catchments discharging to the key outlets shown on Figure 4-2;
- Surrounding storm sewer network; and
- Downstream outlets.

The above data gaps limit the understanding of the design informing the existing system downstream. With this in mind, the storm sewer hydraulics is limited to the connecting leads captured from the survey information (see Section 5.3.1).

5. Hydrological Modelling

5.1 Introduction

The purpose of creating a hydrological model for the East L Yard track expansion project is to generate sub-catchment hydrographs to be used as inflows in the pre and post development hydraulic model and inform on the impacts of development. The hydrological approach for this project is based on engineering guidelines, track standards, design manuals, references and codes mentioned in Section 3. Where there are conflicts between different design guidelines, such as method of runoff analysis, IDF curves and rainfall hyetographs etc., Hatch adopted the most conservative approach. The hydrological inputs, methods and results are provided in the following sections.

5.2 Catchment Definition

As set out in Section 4, the project area does not have functioning cross culverts or watercourses. The catchment for this study is defined by property lines of the rail corridor for proposed tracks. The northern project boundary is made up of the property line that bisects the future rail corridor and the realigned Commissioner Street. The south property line serves as the south limit to East and West, Mi. 125.34 and 126.93 respectively.

It has been found that land parcels beyond the southside of rail corridor slope toward the rail corridor. These areas were not considered as part of the study catchment by assuming that such land parcels were connected to road storm sewer system and do no contribute flows to the rail corridor.

The catchment boundary for the study area was edge-matched to rail corridor. The Digital Elevation Model (Section 5.3.1) was used to create contour data and define slopes and topographical features. The catchment and sub-catchment are shown in Appendix A.

5.3 Subcatchment and Stormwater System Data

5.3.1 Topographic Data and Digital Terrain Model

The Topographic Survey for proposed track alignments was supplied by Underhill Geomatics Ltd. The area to be included beyond the survey limits was downloaded from City of Vancouver open data portal, which provides Digital Elevation Model (DEM) generated from LiDAR data collected in 2013 with resolution of cells of approximately 1 meter.

A comprehensive fine scale Digital Terrain Model (DTM) was compiled using above two data sources. The DEM was constructed as a 1m rectangular grid of elevations that were sampled from this DTM. This defined the topography of the study area and the existing overland flow paths.

5.3.2 *Property Line Data for Rail Corridor*

Property lines were downloaded from the City of Vancouver open data portal and 2019 high resolution aerial photos were supplied by VFPA. It was proposed to extend the rail corridor beyond the existing properly lines to facilitate L30 and L31 track. This extends the catchments up to the future property line that is assumed to be at least 2.750 m from the proposed horizontal alignments (centreline) of L30 and L31 track or to the next nearest road gutter as shown in Appendix A. The data was used to ensure edge-matching of catchment defined for this study.

5.3.3 *Storm Sewer and Utility System Data*

Storm sewer and other utility system data was downloaded from City of Vancouver open data portal. The storm sewer and other utility system data available in the vicinity of Commissioner Street road realignment works were extracted from AECOM's design drawings. Utility survey was performed to determine existing pipe invert elevations and sizes.

5.3.4 *Model Schematisation and Subcatchments*

As there are a large number of catch basins and storm sewer pipes of varying sizes within the catchments, it is essential to ensure that the surface runoff is evenly distributed across the catchment. The advantage of this approach is that each catch basin or pit has a separate inflow.

The PCSWMM methodology does not have limitations regarding the minimum area for subcatchment and its delineation using the DTM developed in Section 5.3.1, whilst considering hydrological characteristics (local drainage patterns, contour information and significant infrastructure) and spacing of the existing catch basins, a total of 36 sub catchments were used to define the drainage patterns and characteristics of the project.

5.3.5 *Percent Imperviousness*

According to the PCSWMM methodology, each subcatchment was divided into pervious and impervious subareas. Then, the impervious area of each subcatchment was further divided into two subareas: a portion with depression storage and remaining without depression storage, which is typically defined as zero impervious.

Pervious, impervious and zero impervious subareas for each subcatchment were calculated using the high-resolution aerial photography along with photos taken during the initial site visit. The pervious area consists of grass cover to the south of existing track and remains to be same between existing and post development conditions. The impervious area consists of mostly rail tracks (categorized as graveled surface) and road pavement and concrete surface. The zero impervious area for this study was assumed to be road pavement and concrete surface plus 25% (the default value for imperious area) of graveled surface. A summary of the subareas for each subcatchment under the pre and post development conditions is provided in Table 5-1.

Table 5-1: Summary of Subcatchment Characteristics

Catchment	Area (m2)	Predevelopment			Post development	
		Previous (m^2)	Impervious Area (m^2)		Previous (m^2)	Impervious Area Graveled surface (m^2)
			Graveled surface	Road & Concrete		
S00	4250	1275	2975	0	1275	2975
S01	5010	546	4464	0	546	4464
S02	3160	329	2691	140	329	2831
S03	1910	302	1528	80	302	1608
S04	3520	649	2502	369	649	2871
S05	2050	464	1252	334	464	1586
S06	1910	360	1213	337	360	1550
S07	1040	225	645	170	225	815
S08	1590	377	991	222	377	1213
S09	1630	372	1053	205	372	1258
S10	1690	328	1103	259	328	1362
S11	1840	382	1192	266	382	1458
S12	1500	331	922	247	331	1169
S13	1820	434	1080	306	434	1386
S14	1990	521	1147	322	521	1469
S15	2270	530	1357	383	530	1740
S16	2810	690	1750	370	690	2120
S17	2620	647	1887	86	647	1973
S18	1590	370	1220	0	370	1220
S19	1630	430	1123	77	430	1200
S20	2220	577	1344	299	577	1643
S21	2330	679	1326	325	679	1651
S22	2240	624	1299	317	624	1616
S23	2230	636	1276	318	636	1594
S24	2360	679	1357	324	679	1681
S25	2290	641	1325	324	641	1649
S26	4200	1081	2592	527	1081	3119
S27	3390	790	1887	713	790	2600
S28	1400	448	778	174	448	952
S29	3020	921	1781	318	921	2099
S30	2110	555	1506	49	555	1555
S31	1900	505	1395	0	505	1395
S32	3510	588	2922	0	588	2922

Catchment	Area (m2)	Predevelopment			Post development	
		Previous (m^2)	Impervious Area (m^2)		Previous (m^2)	Impervious Area Graveled surface (m^2)
			Graveled surface	Road & Concrete		
S33	4240	648	3539	53	648	3592
S34	2710	235	2475	0	235	2475
S35	4000	0	4000	0	0	4000

5.4 Soil Conditions

The factual geotechnical report for Commissioner Street road alignment works suggests that the existing soils in the area are a combination of sands and silts with traces of gravel at various depths.

5.5 Intensity-Frequency-Duration (IFD) Curve

Intensity-Frequency-Duration (IFD) data for East L Yard was obtained from two different sources namely, the Atmospheric Environment Service (AES) as part of Environment Canada and the BGC Engineering report to the Policy and Planning Department, Metro Vancouver.

The AES 2014 IFD curves for Vancouver Harbour (ID – 1108446 and Lat: 49° 8’N and Long 123° 7’W) were compared against the corresponding IFD curve for zone 5 of BGC Engineering Report. The rainfall depth (mm) between two IFD curves were compared for storm durations 1, 3, 6,12 and 24 hours and corresponding 5, 10, 25 and 100 years. Table 5-2 shows the comparisons rainfall depth (mm) between two IDF curves.

Table 5-2: Rainfall Depth (mm) between Two IFD Curves

Duration	Rainfall Depth (mm)							
	5 years		10 years		25 years		100 years	
	AES	BGC	AES	BGC	AES	BGC	AES	BGC
1 hour	14.1	16	16.4	18.6	19.3	22.0	23.6	26.9
3 hours	28.2	28.2	32.4	33.0	37.8	38.7	41.7	47.4
6 hours	38.4	40.8	42.6	47.4	48	55.8	55.7	67.8
12 hours	57.6	58.8	63.6	68.4	73.2	79.2	85.7	97.2
24 hours	84.0	84.0	96.0	98.4	112.8	115.2	135.2	139.2

It has been observed that AES 2014 IFD curves provided similar or slightly less rainfall depth for all storms. However, AES 2014 IFD curves for Vancouver Harbour reflect the local rainfall patterns more than the BCG IFD curves. Therefore, AES 2014 IFD curves were used to simulate the pre and post development conditions respectively. The AES 2014 IFD curves for Vancouver Harbour is presented in Figure 5-1 and rainfall depth and intensity are tabulated in Table 5-3 and Table 5-4 respectively.

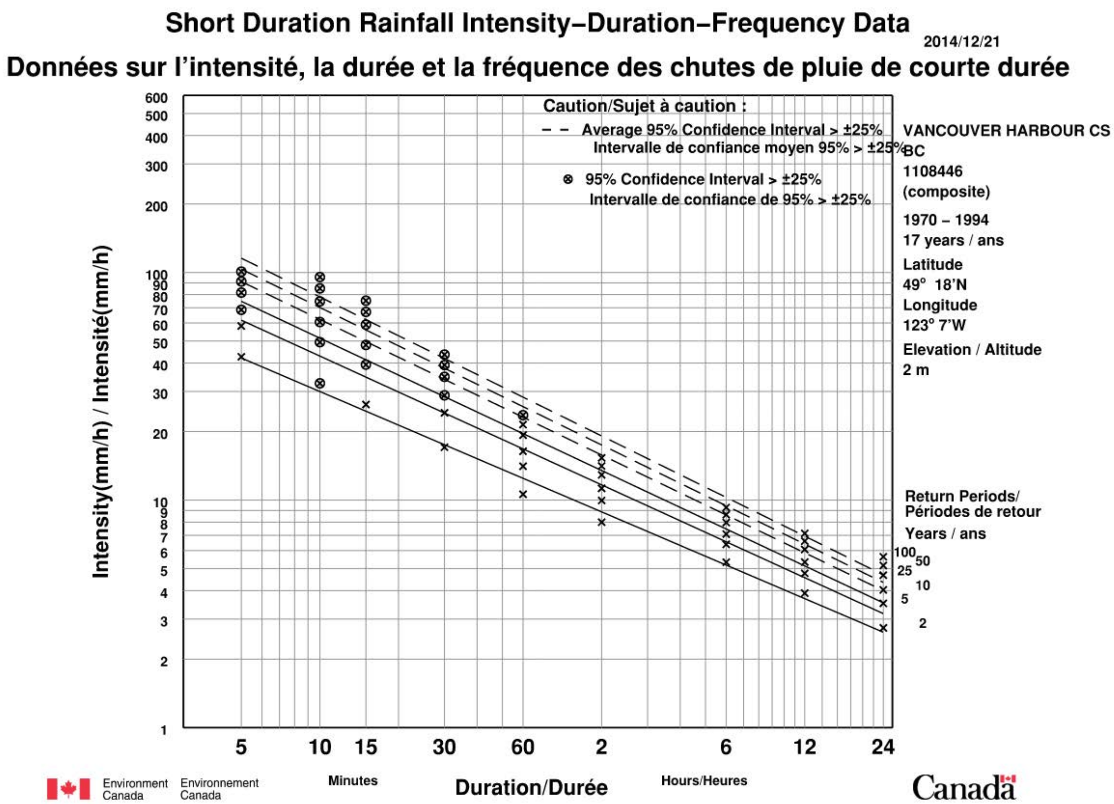


Figure 5-1: AES 2014 IFD curves for Vancouver Harbour (ID-1108446)

Table 5-3: Return Period Rainfall Amounts (mm) - Vancouver Harbour (ID-1108446)

Duration	2 years	5 years	10 years	25 years	50 years	100 years
5 min	3.6	4.9	5.7	6.8	7.6	8.4
10 min	5.4	8.2	10.1	12.4	14.2	15.9
15 min	6.6	9.8	12	14.7	16.8	18.8
30 min	8.5	12.1	14.4	17.4	19.6	21.8
1 hr	10.6	14.1	16.4	19.3	21.5	23.6
2 hr	16	19.9	22.5	25.8	28.2	30.7
6 hr	32	38.3	42.5	47.8	51.8	55.7
12 hr	46.8	57.2	64.1	72.8	79.2	85.7
24 hr	65.8	84.4	96.7	112.3	123.8	135.2

Table 5-4: Return Period Rainfall Intensity (mm/hr) - Vancouver Harbour (ID-1108446)

Duration	2 years	5 years	10 years	25 years	50 years	100 years
5 min	42.6	58.2	68.6	81.6	91.3	100.9
10 min	32.6	49.4	60.6	74.7	85.1	95.5
15 min	26.3	39.4	48	59	67.1	75.2
30 min	17.1	24.2	28.9	34.8	39.2	43.6
1 hr	10.6	14.1	16.4	19.3	21.5	23.6
2 hr	8	10	11.3	12.9	14.1	15.3
6 hr	5.3	6.4	7.1	8	8.6	9.3
12 hr	3.9	4.8	5.3	6.1	6.6	7.1
24 hr	2.7	3.5	4	4.7	5.2	5.6

5.6 Hydrological Model Establishment

PCSWMM modelling software was developed based on the Storm Water Management Model (SWMM) version 5.1 (Huber, Wayne Charles. (1985)). It can be used for modelling fully urban catchments and then generate surface runoff hydrographs from storm rainfall hyetographs. PCSWMM is an industry standard model that has been widely used in previous studies.

5.6.1 PCSWMM Model Approach

PCSWMM is a distributed model and allows the division of the study area into any number of irregularly shaped subcatchment areas to best capture the effect that spatial variability in topography, drainage pathways, land cover and soil characteristics have on runoff generation. PCSWMM then converts precipitation excess into surface runoff or overland flow on subcatchment basis, while considering infiltration, evaporation and initial abstraction.

The Green-Ampt infiltration model is used to model infiltration of rainfall into the upper soil zone. The initial abstraction is represented as the pervious and impervious depression storage. This method allows the infiltration rate to be considered separately to the catchment storage in the hydrological model. This allows more accurate representation of losses as opposed to the rational equation.

5.6.2 Subcatchments

The project area was divided into 36 sub-catchments based on Section 5.3.4 and using the 1m resolution DEM constructed in Section 5.3.1. The subcatchment area as well as percentage of pervious and impervious areas continued to be same between the pre and post condition. The zero impervious areas were changed between the pre and post development conditions and were adjusted according to Section 5.3.5. The hydrological characteristics for each subcatchment are shown in Table 5-5.

Table 5-5: Hydrological Characteristics for Subcatchments

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	Zero Imperv (%)		Subarea Routing	Percent Routed (%)
						Pre- Dev	Post-Dev		
S00	0.425	42.5	100	3	70	25.00	25.00	Outlet	100
S01	0.501	40.949	122.348	2.896	89.10	25.00	25.00	Outlet	100
S02	0.316	36.826	85.808	5.778	89.59	29.43	25.00	Outlet	100
S03	0.191	38.496	49.615	5.368	84.19	29.19	25.00	Outlet	100
S04	0.352	45.591	77.208	7.083	81.56	35.48	25.00	Outlet	100
S05	0.205	36.798	55.709	6.002	77.37	41.29	25.00	Outlet	100
S06	0.191	36.498	52.331	8.030	81.15	42.64	25.00	Outlet	100
S07	0.104	31.927	32.574	10.235	78.37	41.35	25.00	Outlet	100
S08	0.159	37.940	41.908	9.753	76.29	38.96	25.00	Outlet	100
S09	0.163	35.652	45.72	8.066	77.18	37.58	25.00	Outlet	100
S10	0.169	34.519	48.958	7.901	80.59	40.33	25.00	Outlet	100
S11	0.184	37.297	49.334	6.533	79.24	39.46	25.00	Outlet	100
S12	0.150	34.163	43.907	4.202	77.93	41.47	25.00	Outlet	100
S15	0.227	40.151	56.536	7.798	76.65	41.87	25.00	Outlet	100
S16	0.281	41.024	68.496	4.105	75.44	38.17	25.00	Outlet	100
S20	0.222	33.951	65.389	2.619	74.01	38.47	25.00	Outlet	100
S21	0.233	35.701	65.264	3.434	70.86	38.95	25.00	Outlet	100
S22	0.224	34.812	64.345	1.844	72.14	39.15	25.00	Outlet	100
S23	0.223	35.819	62.258	1.392	71.48	39.26	25.00	Outlet	100
S24	0.236	37.788	62.454	2.526	71.23	38.73	25.00	Outlet	100
S25	0.229	32.601	70.244	3.765	72.01	39.15	25.00	Outlet	100
S26	0.42	49.62	84.644	2.43	74.26	37.55	25.00	Outlet	100
S27	0.339	41.17	82.342	4.344	76.7	46.03	25.00	Outlet	100
S28	0.14	45.244	30.943	7.187	68	37.43	25.00	Outlet	100
S29	0.302	42.321	71.36	4.307	69.5	35.53	25.00	Outlet	100
S30	0.211	51.14	41.259	2.536	73.7	27.32	25.00	Outlet	100
S31	0.19	40.111	47.369	2.973	73.42	25	25.00	Outlet	100
S32	0.351	36.243	96.845	2.68	83.25	25	25.00	Outlet	100
S33	0.424	34.145	124.176	3.456	84.72	26.25	25.00	Outlet	100
S34	0.271	50.788	53.359	1.905	91.33	25	25.00	Outlet	100
S35	0.4	45.125	88.642	1.358	100	25	25.00	Outlet	100
S17	0.262	37.774	69.359	1.157	75.31	28.28	25.00	Outlet	100
S18	0.159	30.734	51.735	2.183	76.73	25	25.00	Outlet	100

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	Zero Imperv (%)		Subarea Routing	Percent Routed (%)
						Pre- Dev	Post- Dev		
S19	0.163	31.83	51.209	2.277	73.62	29.72	25.00	Outlet	100
S14	0.199	19.383	102.667	7.735	70.45	41.18	25.00	Outlet	100
S13	0.182	17.727	102.667	7.735	79.84	41.81	25.00	Outlet	100

5.6.3 PCSWMM Parameters

The PCSWMM parameters used in the hydrological modelling and the recommended soil parameters for Loamy Sand are shown in Table 5-6. These parameters were mostly extracted from PCSWMM documentation and EPA SWMM reference Manual Volume 1 - Hydrology.

Table 5-6: SWMM and Soil Parameters

Parameter	Value
Manning's n for Overland Flow - Pervious Area (Short Grass)	0.15
Manning's n for Overland Flow - Pervious Area (Gravelled Surface)	0.02
Depression Storage for Impervious Surfaces (mm)	2.00
Depression Storage for Pervious Surfaces (mm)	3.00
Groundwater	NA
Erosion	NA
Hydraulic Conductivity - Loamy Sand (mm/hr)	29.97
Suction - Loamy Sand	60.96
Porosity - Loamy Sand (Fraction)	0.437
Field Capacity- Loamy Sand (Fraction)	0.105
Wilting Point- Loamy Sand (Fraction)	0.047
Initial Deficit- Loamy Sand (Fraction)	0.39

5.7 PCSWMM Results

Using the parameters described above, the PCSWMM model was run for a full range of design storm events and runoff hydrographs have been extracted for use in the hydraulic model. A summary of PCSWMM outputs are provided in this section.

5.7.1 Rainfall Hyetographs

Two types of rainfall hyetograph were generated using design storms distribution namely AES (City of Vancouver) and Chicago based on the AES 2014 IDF curves for Vancouver Harbor. Figure 5-2 shows two rainfall hyetographs, correspond to a storm event of 24-hour duration and 100 year.

In running both hyetographs it was observed that the Chicago design storms distribution produced higher rainfall intensity for any given storm duration and year compared to the AES distribution. In turn, it generates the highest peak runoff for each sub catchment. When undergoing a hydraulic analysis against the limited connection information available, the flows produced by the AES method it did not correlate with existing pipe sizes, as the existing connecting pipes would need to be significantly larger. When undergoing a similar analysis using the runoff produced by the Chicago Storm method, the runoff flowing to the existing connections was more in line with existing conditions. The pipe sizes are undersized by today's standards but not by a significant margin.

Therefore, Chicago was selected for the purpose of the hydrological modelling.

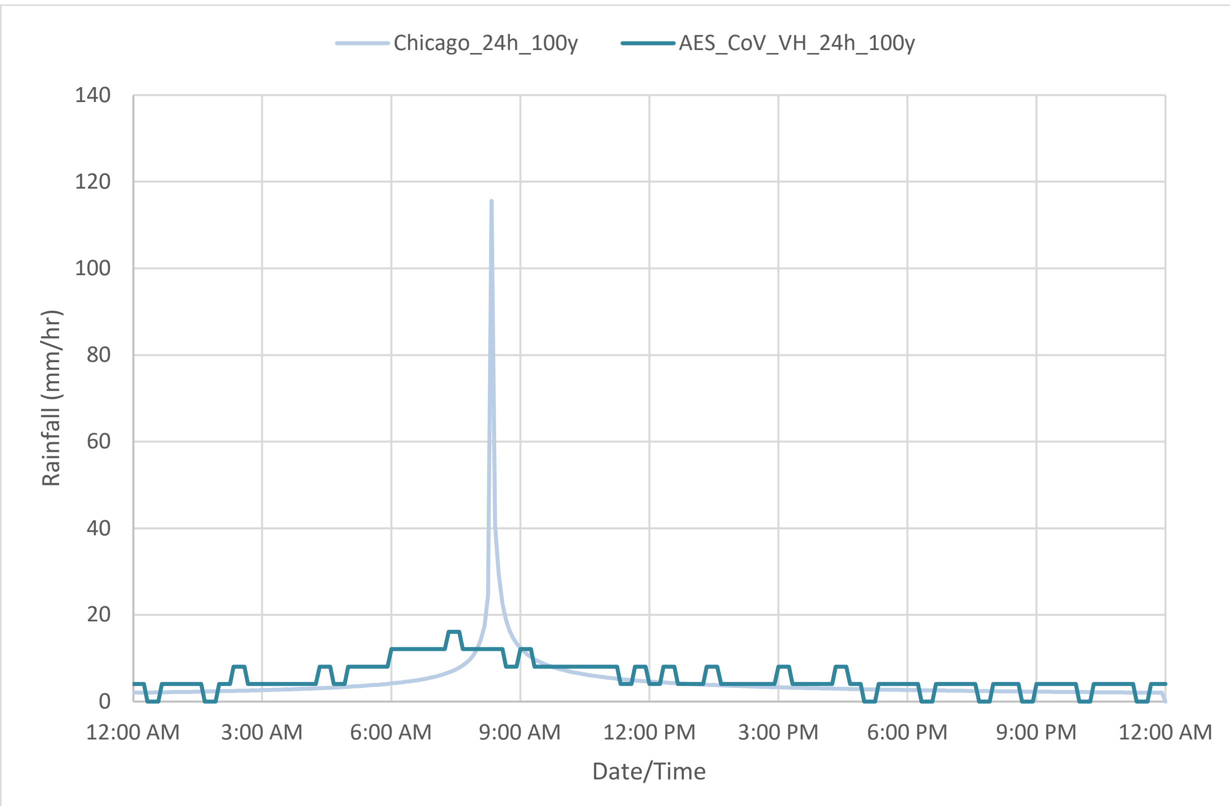


Figure 5-2: Rainfall Hyetographs for Three Storm Distribution

5.7.2 Pre and Post Development Runoff

The predevelopment PCSWMM model was used to simulate a full range of Chicago design storms. The storm duration and year varies between 1 to 24 hours and 5 to 100, respectively. Figure 5-3 shows runoff hydrographs corresponding to 6, 12 and 24 hour duration and 5 year storm events.

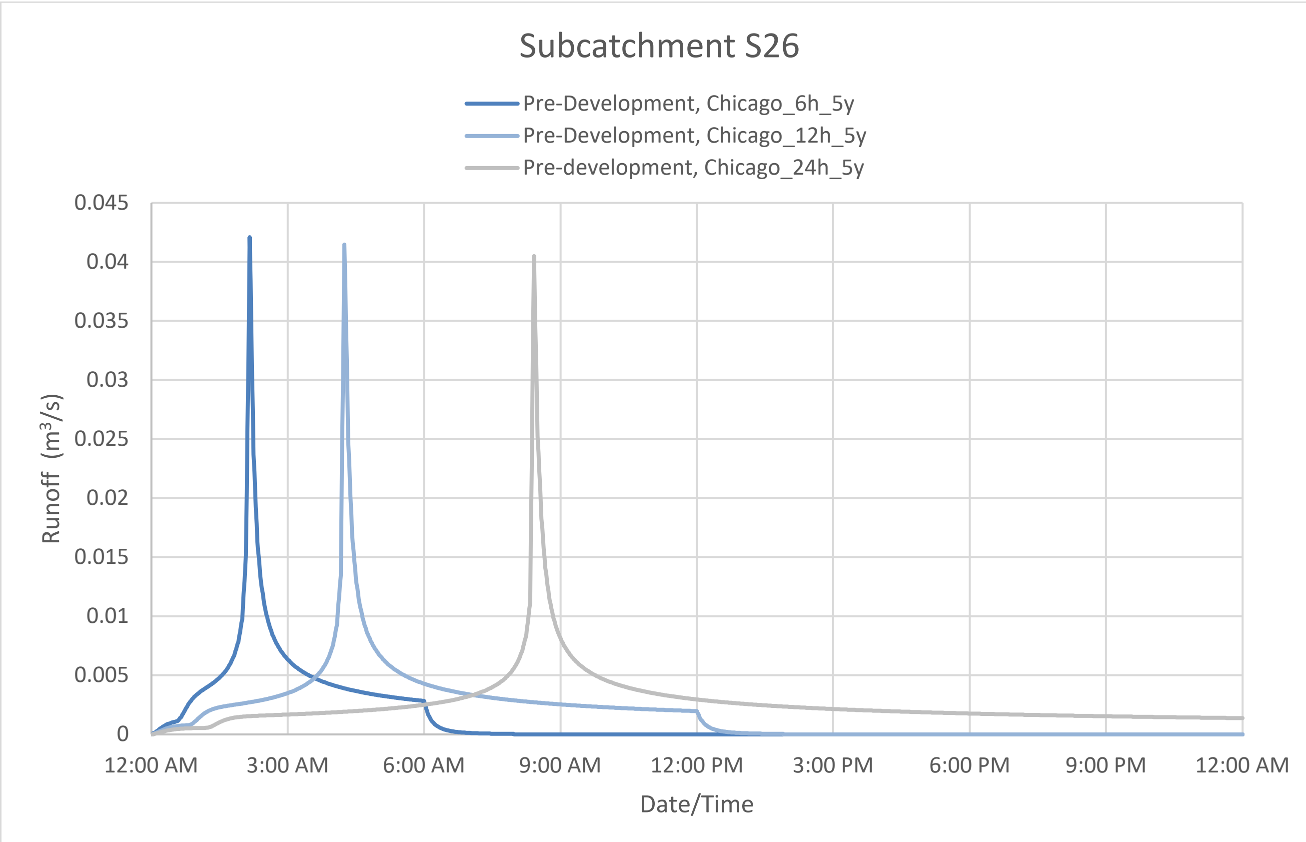


Figure 5-3: Runoff Hydrograph for S26 for 6-, 12- and 24-hour Duration, 5 year Storm Event

It was observed that there is a small difference in the peak runoff for each sub catchment (0.0015m^3/s between the 6-hour and 24-hour storm duration). However, the 24-hour storm event yielded the maximum total runoff. Given the relatively small difference in peak runoff between the three storm durations, the post development PCSWMM model was only used to simulate 24-hour duration storm events related to 2, 5 and 100 year. The peak runoff and runoff coefficient for each subcatchment associated with the pre and post development are shown in Table 5-7. The pre and post development runoff hydrographs for S26 related 24-hour duration and 5-year storm event are shown in Figure 5-4:.

Table 5-7: Pre and Post Development Peak Runoff And Runoff Coefficient For Subcatchments

Sub-catchment	Pre-development						Post-development						Diff. btw Pre & Post		
	2 yr Peak Runoff (m³/s)	Runoff Coeff.	5 yr Peak Runoff (m³/s)	Runoff Coeff.	100 yr Peak Runoff (m³/s)	Runoff Coeff.	2 yr Peak Runoff (m³/s)	Runoff Coeff.	5 yr Peak Runoff (m³/s)	Runoff Coeff.	100 yr Peak Runoff (m³/s)	Runoff Coeff.	2 yr Peak Runoff (m³/s)	5 yr Peak Runoff (m³/s)	100 yr Peak Runoff (m³/s)
S00	0.03	0.679	0.04	0.683	0.08	0.69	0.03	0.679	0.04	0.683	0.08	0.69	0	0	0
S01	0.03	0.862	0.05	0.868	0.1	0.876	0.03	0.862	0.05	0.868	0.11	0.876	0	0	0
S02	0.03	0.872	0.04	0.877	0.08	0.884	0.03	0.871	0.04	0.876	0.08	0.883	0	0	0
S03	0.02	0.821	0.02	0.825	0.05	0.831	0.02	0.82	0.02	0.824	0.05	0.831	0	0	0
S04	0.03	0.796	0.04	0.8	0.08	0.806	0.03	0.793	0.04	0.798	0.09	0.804	0	0	0
S05	0.02	0.757	0.02	0.761	0.05	0.765	0.02	0.753	0.02	0.757	0.05	0.763	0	0	0
S06	0.02	0.795	0.02	0.798	0.05	0.803	0.02	0.791	0.02	0.795	0.05	0.801	0	0	0
S07	0.01	0.768	0.01	0.771	0.02	0.775	0.01	0.764	0.01	0.768	0.03	0.773	0	0	0
S08	0.01	0.747	0.02	0.75	0.04	0.754	0.01	0.744	0.02	0.747	0.04	0.753	0	0	0
S09	0.01	0.755	0.02	0.759	0.04	0.763	0.01	0.752	0.02	0.756	0.04	0.761	0	0	0
S10	0.01	0.789	0.02	0.792	0.04	0.797	0.01	0.785	0.02	0.789	0.04	0.795	0	0	0
S11	0.02	0.776	0.02	0.779	0.04	0.784	0.02	0.772	0.02	0.776	0.05	0.782	0	0	0
S12	0.01	0.763	0.02	0.766	0.03	0.771	0.01	0.759	0.02	0.763	0.04	0.769	0	0	0
S13	0.01	0.78	0.02	0.784	0.04	0.789	0.01	0.776	0.02	0.78	0.04	0.787	0	0	0
S14	0.01	0.689	0.02	0.692	0.04	0.697	0.01	0.685	0.02	0.689	0.04	0.695	0	0	0
S15	0.02	0.751	0.03	0.754	0.05	0.758	0.02	0.747	0.03	0.751	0.05	0.756	0	0	0
S16	0.02	0.737	0.03	0.74	0.06	0.746	0.02	0.734	0.03	0.738	0.06	0.744	0	0	0
S17	0.02	0.731	0.02	0.735	0.05	0.741	0.02	0.73	0.02	0.735	0.05	0.741	0	0	0
S18	0.01	0.746	0.02	0.75	0.03	0.756	0.01	0.746	0.02	0.75	0.04	0.757	0	0	0
S19	0.01	0.717	0.02	0.721	0.03	0.726	0.01	0.716	0.02	0.72	0.04	0.726	0	0	0
S20	0.02	0.723	0.02	0.726	0.04	0.731	0.02	0.719	0.02	0.723	0.05	0.73	0	0	0
S21	0.02	0.692	0.02	0.696	0.05	0.7	0.02	0.689	0.02	0.693	0.05	0.699	0	0	0

Sub-catchment	Pre-development						Post-development						Diff. btw Pre & Post		
	2 yr Peak Runoff (m^3/s)	Runoff Coeff.	5 yr Peak Runoff (m^3/s)	Runoff Coeff.	100 yr Peak Runoff (m^3/s)	Runoff Coeff.	2 yr Peak Runoff (m^3/s)	Runoff Coeff.	5 yr Peak Runoff (m^3/s)	Runoff Coeff.	100 yr Peak Runoff (m^3/s)	Runoff Coeff.	2 yr Peak Runoff (m^3/s)	5 yr Peak Runoff (m^3/s)	100 yr Peak Runoff (m^3/s)
S22	0.02	0.704	0.02	0.707	0.04	0.713	0.02	0.701	0.02	0.705	0.05	0.711	0	0	0
S23	0.01	0.697	0.02	0.701	0.04	0.706	0.01	0.694	0.02	0.698	0.05	0.704	0	0	0
S24	0.02	0.696	0.02	0.699	0.05	0.704	0.02	0.693	0.02	0.696	0.05	0.702	0	0	0
S25	0.02	0.704	0.02	0.707	0.05	0.712	0.02	0.7	0.02	0.704	0.05	0.71	0	0	0
S26	0.03	0.724	0.04	0.727	0.08	0.733	0.03	0.721	0.04	0.725	0.09	0.731	0	0	0
S27	0.02	0.751	0.04	0.754	0.07	0.759	0.02	0.745	0.04	0.75	0.08	0.756	0	0	0
S28	0.01	0.666	0.02	0.668	0.03	0.672	0.01	0.663	0.02	0.666	0.03	0.671	0	0	0
S29	0.02	0.678	0.03	0.682	0.06	0.687	0.02	0.676	0.03	0.68	0.06	0.685	0	0	0
S30	0.02	0.718	0.02	0.722	0.04	0.727	0.02	0.718	0.02	0.721	0.05	0.727	0	0	0
S31	0.01	0.715	0.02	0.719	0.04	0.724	0.01	0.715	0.02	0.719	0.04	0.724	0	0	0
S32	0.02	0.807	0.04	0.812	0.07	0.819	0.02	0.807	0.04	0.812	0.08	0.819	0	0	0
S33	0.03	0.821	0.04	0.826	0.09	0.833	0.03	0.821	0.04	0.826	0.09	0.833	0	0	0
S34	0.02	0.887	0.03	0.892	0.07	0.9	0.02	0.887	0.03	0.892	0.07	0.9	0	0	0
S35	0.03	0.967	0.04	0.973	0.09	0.982	0.03	0.967	0.04	0.973	0.09	0.982	0	0	0

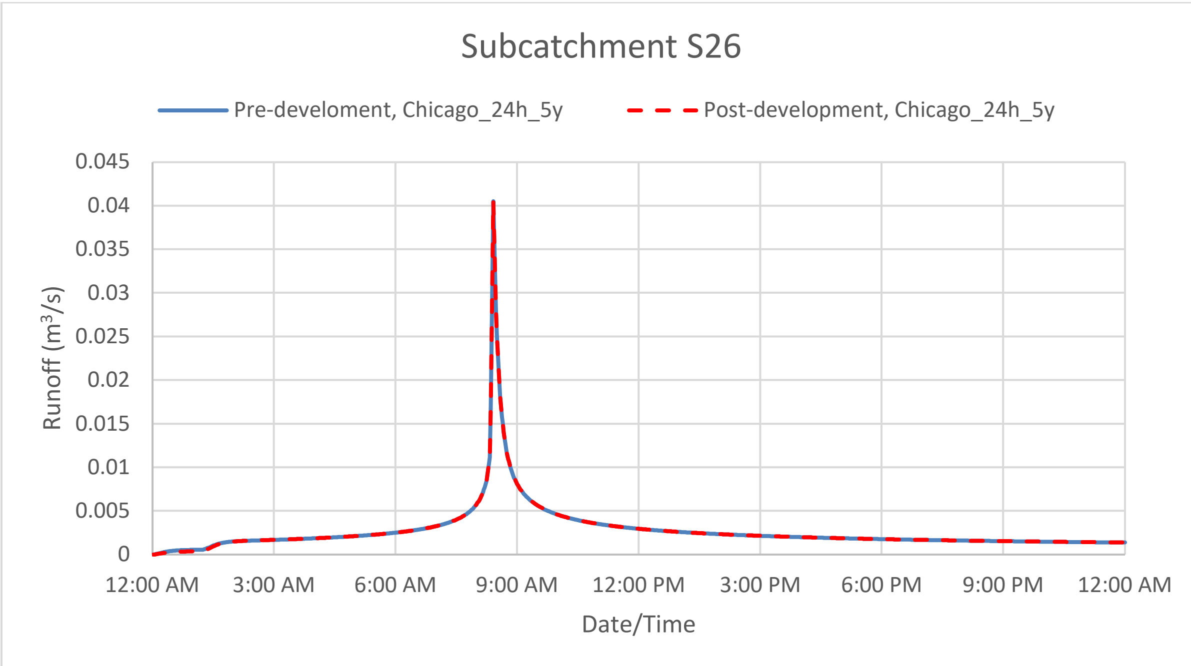


Figure 5-4: Pre and Post Development Runoff Hydrographs for S26 (24-hour, 5-year storm event)

There was a negligible decrease in runoff coefficient from existing to the post development conditions. There is a change in the zero imperious area as a result of Commissioner street existing road pavement being converted to ballast surface to facilitate the horizontal alignment of L30 and L31 tracks. However, the percentage imperviousness continued to be same between two conditions, even though there was a catchment wide 10% reduction in zero imperious area. Consequently, the peak runoff for each sub catchment remained relatively unchanged between the pre and post development conditions.

A sensitivity analysis was conducted to investigate the sensitivity of zero impervious area to the peak runoff. It was found that the percentage zero impervious area was sensitive to the shape of surface runoff hydrograph, but peak runoff remained close to zero.

It was observed that post development stormwater runoff from the rail corridor for 24 hour 5- and 100-year storm events do not discharge additional runoff to the existing storm sewer system and peak runoff remained similar to the existing conditions.

6. Hydraulic Modelling

This section discusses the hydraulic model development and analysis of the existing storm sewer system, the new drainage design, surcharging and freeboard conditions.

6.1 Existing pipe network

The VFPA storm sewer network is the main stormwater conveyance north of the project area and it ultimately drains to the Burrard inlet. As part of the early works and the Commissioner St relocation, a new drainage network was designed by AECOM. This new drainage network will be installed between Renfrew Street and Slocan Street. Information from both the VFPA storm sewer network and the AECOM road drainage design were compiled as part of the existing pipe network surrounding the project area. Additional surveys were conducted to ascertain the position, invert elevation and connecting pipes to the existing manholes, catch basins and clean outs. There is minimal information on the discharging outlets draining into the Burrard Inlet.

Various assumptions were made whilst modelling the existing storm sewer network to accommodate for the pits which could not be picked up or found as part of the feature survey, those include:

- Existing Manhole and Catch Basin
  Size – Assumed to be 1500ømm for manholes and 750x750mm for catch basins. This is assumed for all existing structures.
- Invert elevation - Assigned invert if captured by survey data. Otherwise, an invert elevation was assumed based on the flow direction.
- Existing Storm Sewer
  Size - Assigned if pipe size was captured by the survey, otherwise assumed based on the given adjacent upstream and downstream sizes.

6.2 Proposed subsurface network

The proposed subsurface network was designed to capture runoff from the new proposed tracks to the existing storm sewer network or to the adjacent road corridor. The pipe size and invert levels of the proposed pipes were governed by the existing storm sewer network (tie-in to existing manholes and downstream pipe size), the elevation of the proposed track sub ballast and the horizontal space available within the rail corridor. A minimum grade of 0.22% was also considered for the proposed pipes design. Catch basins were proposed at 90m intervals and clean outs in-between every 30m.

All proposed pipes are 300mm diameter to allow for shallower slopes without accumulating excessive sediment. These pipes are then connected to the existing storm sewer network and existing catch basin leads

The Table 6-1 below illustrates the proposed drainage strategy and the existing connection points considered.

Table 6-1: Proposed Drainage Strategy

Chainage[1]	Design level of Rail corridor relative to downstream Road corridor	Proposed Drainage Strategy	Connection Points	INV	Approx. Design RIM	Downstream pipe dia. (mm)	Remarks
0+000 to 0+225	Above	Surface runoff directed to the road corridor as sheet flow					
0+225 to 0+447	Below	Subsurface drainage	MH88012 CBE2910	6.17 6.3	8.04 7.375	200 150	• Downstream pipe connected to existing manhole to be upsized to a 300mm pipe. • Existing pipe connected to CBE2910 to be upsized to a 300mm pipe.
0+447 to 0+587	Above	Surface runoff directed to the road corridor as sheet flow					
0+587 to 0+750	Below	Subsurface drainage	MH50404	2.31	6.28	300	• Existing manhole may need to be replaced. Conditions to be confirmed in the field.
0+750 to 0+921	Above	Surface runoff directed to the road corridor as sheet flow					
0+921 to 1+367	Below	Subsurface drainage	CB50080 CB50020 CB80321 CB85783	4.64 4.38 4.16 4.53	5.54 5.408 5.04 3.37	150 150 150 150	• Existing catch basins to be removed. Existing lead to be adjusted and tied into proposed manhole/catch basin • Existing downstream outlets have unverified capacity to accommodate 2 or 5 yr model flow. • Upsize connecting downstream pipes to 300mm pipes.
1+367 to 1+625	Above	No subsurface drainage under the proposed Columbia containers lead track. Sheet flow to road corridor					
1+625 to 1+725	Below	Subsurface drainage	MH80550	4.13	3.2	200	• Relocate existing manhole to be removed. Existing lead to be adjusted and tied into proposed manhole. • Unverified pit inv and connecting existing pipe dia. for MH80550. Downstream pipe assumed to be 150mm dia. • Existing outlets have unverified capacity to accommodate 5yr model flow. • Upsize downstream pipe to 300mm pipes.

Chainage[1]	Design level of Rail corridor relative to downstream Road corridor	Proposed Drainage Strategy	Connection Points	INV	Approx. Design RIM	Downstream pipe dia. (mm)	Remarks
1+725 to 1+850	Above	Surface runoff directed to the road corridor as sheet flow					
1+850 to 1+950	Below	Subsurface drainage	CB80799 CB80813	3.16 3.12	4.02 4.14	150 150	• Existing CB80799 to be relocated. Existing catch basin & leads to be removed. • Existing CB80813 to be relocated. Existing catch basin & leads to be removed. • Unverified outlet capacity. • Upsize downstream pipe to 300mm pipes.
1+950 to 2+075	Above	Surface runoff directed to the road corridor as sheet flow					
2+075 to 2+290	Below	Subsurface drainage	CB80972 MH81052 SE lead	3.2 2.7*4	4.08 4.15	150 150	• Existing catch basin and manhole to be relocated. Existing leads to be adjusted and tied into proposed structures. • Surcharge within the proposed pipe network. • Upsize downstream pipe to 300mm pipe.
7+251 to 7+425[2]	Below	Subsurface drainage	CB81095	4.36	5.48	150	• Existing CB81095 to be replaced with a 900mm dia. manhole (MH701) • Unverified outlet capacity. • Existing outlets do not have capacity to accommodate 5 yr model flow. • Upsize downstream pipe to 300mm pipes.

Note:

1. Chainage 0+000 to 2+248 is based on the Proposed L30 track alignment.
2. Chainage 7+000 to 7+425 is based on the Proposed L31 track alignment.
3. Unable to locate MH81052 on site. Invert level was assumed to be 2.7m based on the downstream pipe elevation and the slope of other surrounding existing pipes
4. See the 90% design drawing set for the connection points.

6.2.1 *Capacity of proposed drainage network*

The proposed drainage network was modelled by developing a post-development PCSWMM model. The model is based on a 2-yr and 5-yr flood event to assess the capacity of the proposed drainage network and the impact on the immediate existing drainage network. Given the limited available information about the existing downstream storm network, the analysis currently terminates at the connection points and subsequent downstream pipes where survey information is available.

Table 6-2 below summarizes the sections of the proposed pipe network which are experiencing surcharge during a 2-yr and 5-yr flood event. Surcharging in PCSWMM occurs when flows exceed the capacity of pipes and it backs up into a junction (manhole, catch basin or clean out). The modelling shows that while there are flows backing up into the manhole and catch basin structure In All the cases, the surcharge lasts for less than six minutes. There is only one location where flows rise above the top of the manhole structure in the 5-year storm event for less than two minutes of ponding at a depth of less than 5mm between sta. 7+251 and 7+425 at the connecting manhole (MH701)

Table 6-2: Sections of the proposed pipe network surcharged during a 2yr and 5yr event

Chainage	Proposed pipe network dia. (mm)	Surcharge in the 2yr flow model	Surcharge in the 5yr flow model	Existing downstream outlet
0+225 to 0+447	300	x	x	West: Unverified downstream outlet East: Unverified tie in point, flow drains to relocated Commissioner St drainage network
0+587 to 0+750	300	x	x	Unverified downstream outlet
0+921 to 1+367	300	✓	✓	Surcharge occurs within the connecting existing leads. Unverified downstream outlet
1+625 to 1+725	300	x	x	Unverified downstream outlet
1+850 to 1+950	300	x	x	Unverified downstream outlet
2+075 to 2+290	300	x	✓	Unverified downstream outlet
7+251 to 7+425	300	✓	✓	Surcharge occurs within the downstream pipes, cleanout and catch basins of the proposed network.

Note:

1. Refer to the attached drawing set submitted by Hatch for pipe details, flow direction and invert levels.
2. The same connecting structures under surcharge during the 2yr and 5yr storm events between 0+921 and the tie in point at 1+367

6.2.2 Climate Change

The effect of climate change was taken into consideration in evaluating the 5-year storm flood events and designing the proposed pipe network. The MMCD Design Guideline does not recommend a range of factors nor does it discern a factor by geographic areas within Vancouver. However, the guideline suggests that an increase factor of 15% is applied to rainfall intensities in Greater Vancouver.

The 5-year storm model was run with a 15% increase in rain intensities, The locations of surcharge within the proposed subdrain network are similar to what was identified in Section 6.2.1.

The proposed pipe network within the rail corridor, however, has enough capacity to accommodate for the 5-year storm model with the additional 15% climate change factor.

6.2.3 Relocation of pits

As shown in Table 6-1 a few existing structures (manholes, cleanouts and catch basins) will be relocated as they either conflict with the proposed road barrier (sta. 0+760 to sta. 1+097) or will be replaced by new proposed connection points further north aligned with the proposed pipe network. See the Issued for 90% design drawing set H362376-RW-100-S0-4001 to H362376-RW-100-S0-4008 for location of structures to be relocated.

6.3 Surcharging

The surcharge within the proposed pipe network is caused by limitations at the connection point leads; the size and capacity of the existing storm sewer network. The majority of the existing connection point leads are 150mm or 200mm in diameter compared to the proposed 300mm diameter pipes. The connection to smaller pipes causes pipes to back up near the connection points. This backup dissipates upstream along the pipe networks.

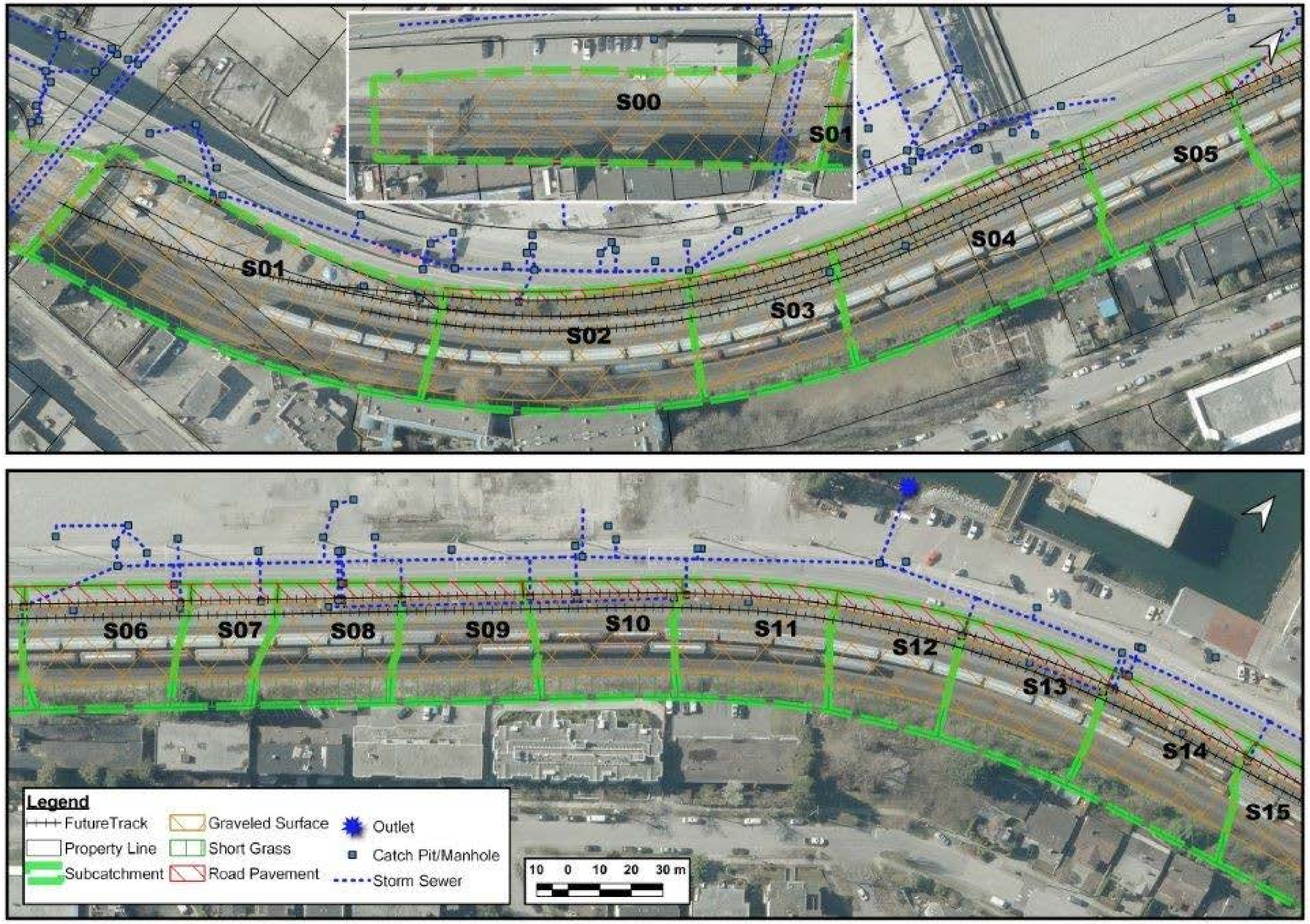
If the existing storm sewer under Commissioner St is upgraded, the existing connections to the subdrain pipes should be upsized to match the 300mm dia pipes. Such an upsize was not done as part of this project to avoid increasing flows to the existing storm sewer from the rail corridor.

7. Summary and Recommendation

This project involves the expansion of the South Shore East L Yard between Mi. 125.34 and 126.93 in Vancouver, BC. Prior to the beginning of the project, the Commissioner Street (currently north of the yard) will be relocated further north to accommodate for the new L30 to L32 tracks. A new drainage network was designed by AECOM as part of the relocation of the road corridor. Hatch has completed a preliminary drainage assessment and has determined:

- The track expansion works will result in a negligible increase in impervious surface (gravel). As a result, the peak runoff for each sub catchment remained relatively unchanged between the pre and post development conditions
- The post development minor storms: 2-year and 5-year storm events were considered as part of this analysis. It was found that that the proposed development could accommodate 2-year storms and could connect to the existing stormwater network. The limitation with the connecting stormwater and outlets to Burrand Inlet is that existing information is limited and the capacity of the existing downstream network cannot be confirmed as part of this phase of work. The Hatch design team recommend a capacity analysis of the downstream system be conducted to improve drainage and surcharge occurrences along Commissioner Street.
- The proposed pipe network can accommodate 5-yr storms with an increase 15% factor to account for climate change. Surcharge, however, occurs at the downstream proposed connecting structures for short period of time.
- Given the negligible change in the peak 100-yr runoff flow between the pre and post development conditions, the proposed extension of the CP rail yard will have no additional adverse impact on the Commissioner Street and the Port premises.
- The proposed L30 and L31 tracks will have sufficient drainage such that no excess maintenance is anticipated.

Appendix A
Subcatchment Layout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S14
S15
Legend
FutureTrack
Property Line
Subcatchment
Graveled Surface
Short Grass
Road Pavement
Outlet
Catch Pit/Manhole
Storm Sewer
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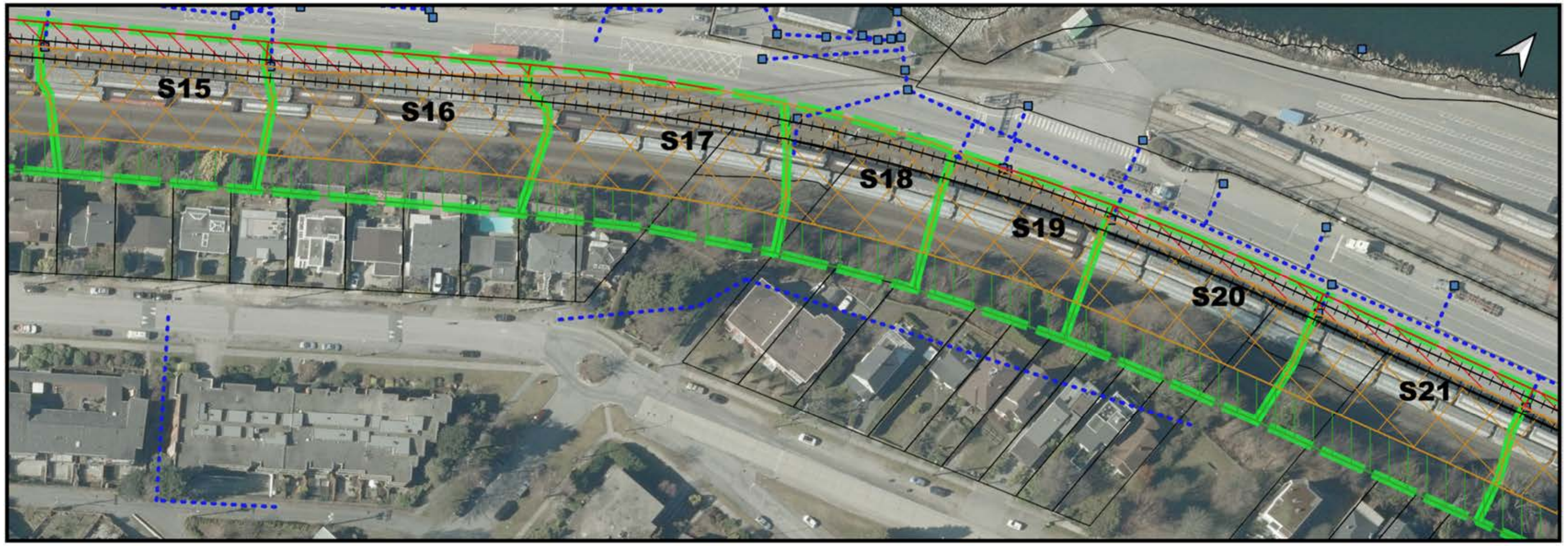
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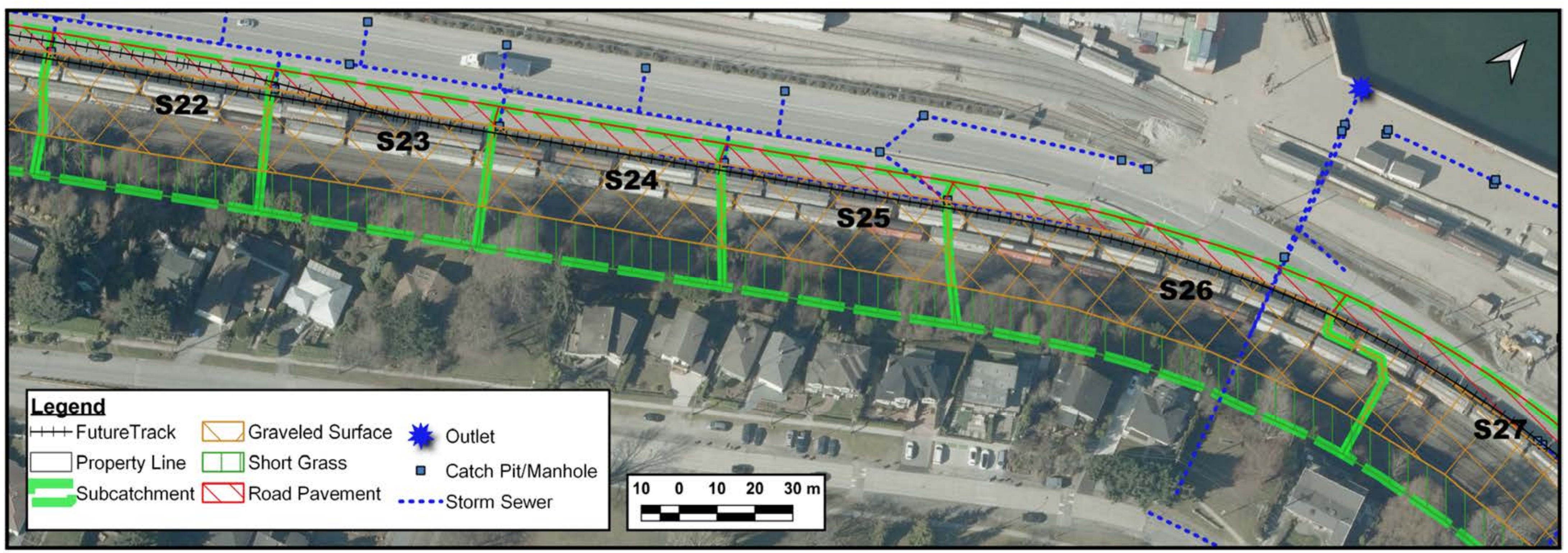
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Property Line
Subcatchment
Graveled Surface
Short Grass
Road Pavement
Outlet
Catch Pit/Manhole
Storm Sewer
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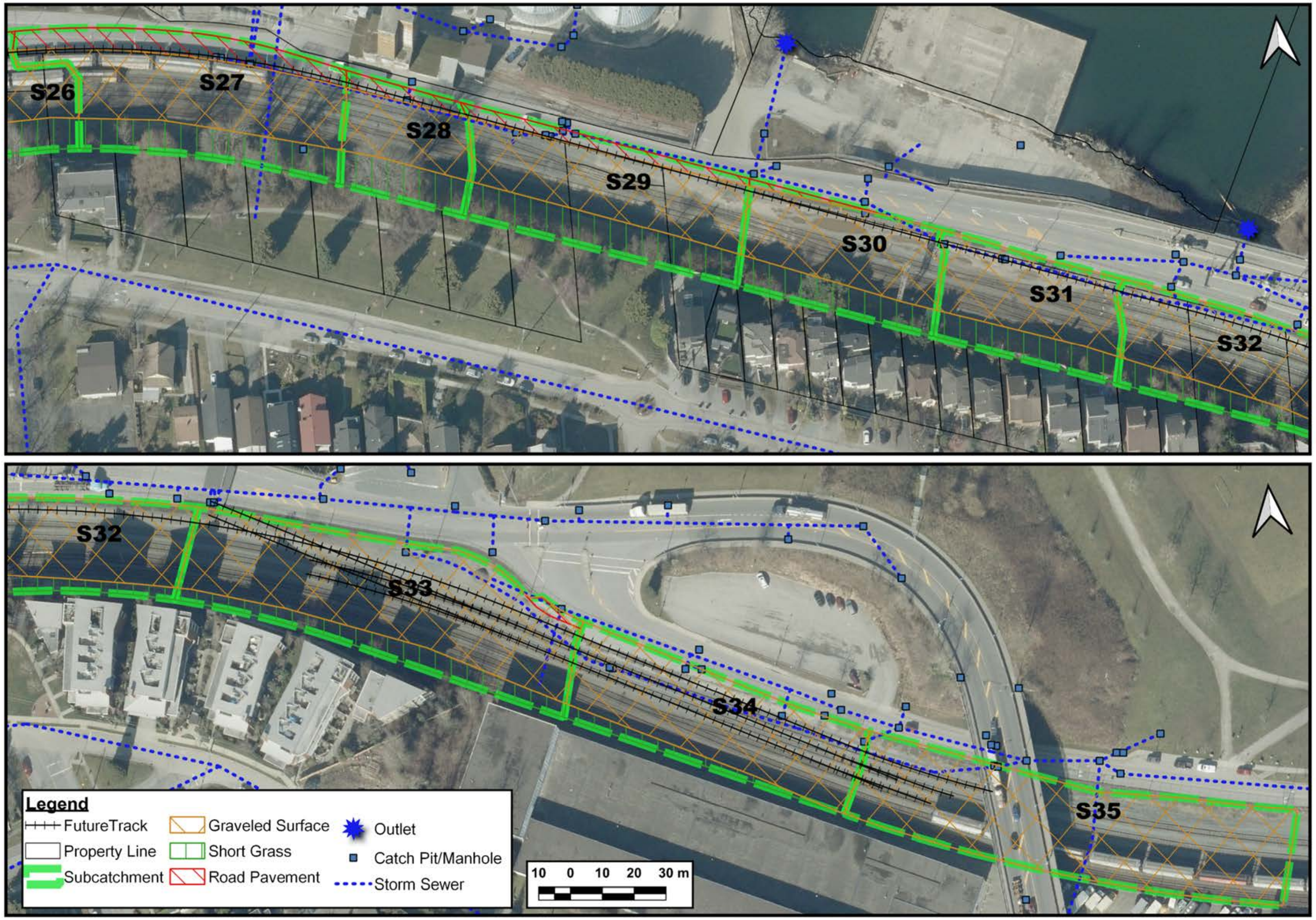
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