Seattle Public Utilities

Protecting Seattle's Waterways

South Park Hydraulic Modeling Report

September 2014

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Specifically, the project team recognizes the following personnel for their efforts:

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List of Abbreviations

Term	Definition
cfs	cubic foot/feet per second
ft	foot/feet
ft^3	cubic foot/feet
GIS	geographic information system
h	hour(s)
HGL	hydraulic grade line
in.	inch(es)
ID	identification number
MH	maintenance hole
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
RG	rain gauge
Sea-Tac	Seattle-Tacoma International Airport
SPU	Seattle Public Utilities
SR	State Route
SWMM5	EPA-SWMM5 v22

Executive Summary

This South Park Hydraulic Modeling Report describes modeling studies conducted to update the capacity requirements for the South Park Flood Control Pump Station. Since the original study (Brown and Caldwell, 2007), the tributary basin has been enlarged, and it has been decided to analyze the system under the impact of a 2-foot increase in tidal elevations due to climate change. In addition, changes in technology allow a more complete analysis of the service level of the pump station. Seattle Public Utilities' (SPU) service level is to manage stormwater within the City right-of-way up to and including a 25-year recurrence interval. In order to achieve this, the depth of surcharge above the crown of the 7th Avenue S storm drain at S Holden Street cannot exceed 3 feet more than once in 25 years. This is equivalent to a depth above the invert of the 72-inch drain of 9 feet. Based on the assumed elevation at the connecting maintenance hole (MH) D071-102 of -2.19 feet North American Vertical Datum 1988 (NAVD88), this corresponds to a water surface elevation of 6.81 feet.

The original South Park Drainage Area hydraulic model was created in 2002 as part of the South Park Drainage Study (R.W. Beck, 2002). It has been modified and updated by several consultants. This study began with the most recent model version developed for the S Portland Street Drainage Improvement project (Kennedy/Jenks, 2013), which included conversion of the model last created by Brown and Caldwell from the XPSWMM model platform to the PCSWMM platform using the EPA-SWMM5 engine. Brown and Caldwell made modifications to that model to perform the analyses reported here.

This study indicated that for the existing basin (238 acres), a flood control pump station including four 18-cubic-foot per second (cfs) pumps (72 cfs installed capacity with one pump considered as an emergency standby) would meet the identified service level.

Analyses were conducted with a future enlarged basin (278 acres) with increased imperviousness; a 4-foot tide level increase; and correction of flooding in S Chicago Street, S Holden Street, and S Austin Street by enlarging and re-grading the drains in the model. This analysis suggested that the flood control pump station would need to be enlarged to four 25 cfs pumps (100 cfs total installed capacity), to meet the service level assuming detention control of outflow from the large subcatchment in the southwest corner of the basin (subcatchment B-Inlet#1 in the model; see Figure 3-6). This 45-acre basin is expected to increase in imperviousness from the current 20 percent to 95 percent in the future. With the future imperviousness and no detention in this subcatchment, the outflow from this basin causes MH flooding in several locations downstream. If, instead of detention, SPU chose to increase the conveyance capacity of affected drains, the modeling indicates that four 30 cfs pumps (total installed capacity of 120 cfs) would be required to meet the service level.

SECTION 1

Introduction

This report describes the development and results of a hydraulic model for the Seattle Public Utilities (SPU) South Park Drainage Area (also described as the 7th Avenue S drainage basin). The South Park Drainage Area model was developed to define the current flooding potential and to size the South Park Flood Control Pump Station to mitigate flooding.

This study is the continuation of work conducted in 2007–08 for design of the flood control pump station.

1.1 Project Background

In 2007, Brown and Caldwell was commissioned to design a flood control pump station to protect the South Park Drainage Area. As a part of that work, Brown and Caldwell updated an existing XPSWMM model for the area. The original model was developed by R.W. Beck and Associates (R.W. Beck, 2002). The original model was subsequently updated by Herrera Environmental Consultants to update the Marra Farm area, and then by Davido Consulting Group to add improvements in the S Director Street area.

Brown and Caldwell received and integrated the three models listed above for use in the 2007 flood control pump station design. This work included configuring the flood control pump station with appropriate controls, reconfiguring the tide valve at the discharge, running the model for a 25-year design storm with a corresponding tide level, and running selected storm events from the SPU synthetic 158-year rainfall record produced by MGS Consultants. The goal of this effort was to define the flood control pump station capacity necessary to hold the depth of flow in the 7th Avenue S storm drain at the S Holden Street drain connection (assumed to be MH D071-102) to 9 feet above the invert, with exceedance of that depth at a frequency of no more than once in 25 years on average (service level). Based on the invert elevation at MH D071-102 in the model, the service level of 9 feet above the invert translates to a water surface elevation of 6.81 feet. The results of that modeling are contained in the Brown and Caldwell report (Brown and Caldwell, 2007).

Recently, Aqualyze, Inc. converted the 2007 version of the model from the XPSWMM platform to PCSWMM and added the S Portland Street storm drain as designed to the last Brown and Caldwell model (Kennedy/Jenks, 2013).

Brown and Caldwell received the Aqualyze model for use in the re-analysis of the project. Modifications to the model as well as modifications to the analysis technique are described below.

1.2 Project Objectives

The objective of this work was to update sizing of the flood control pump station for the South Park Drainage Area to accommodate recent changes in the basin definition and tidal level assumptions.

1.3 Study Area

The South Park Drainage Area is generally bounded by S Othello Street on the north, S 96th Street on the south, 8th Avenue S to the east, and State Route (SR) 509 on the west. A large subcatchment is included to the west of SR 509 at the south end of the area. W Marginal Way S bisects the area into upper and lower basins.

The South Park Drainage Area currently covers about 238 acres with a potential future addition of 40 acres. Figure 1-1 shows the area. Details of the drainage network and subcatchment boundaries are shown in Section 3.

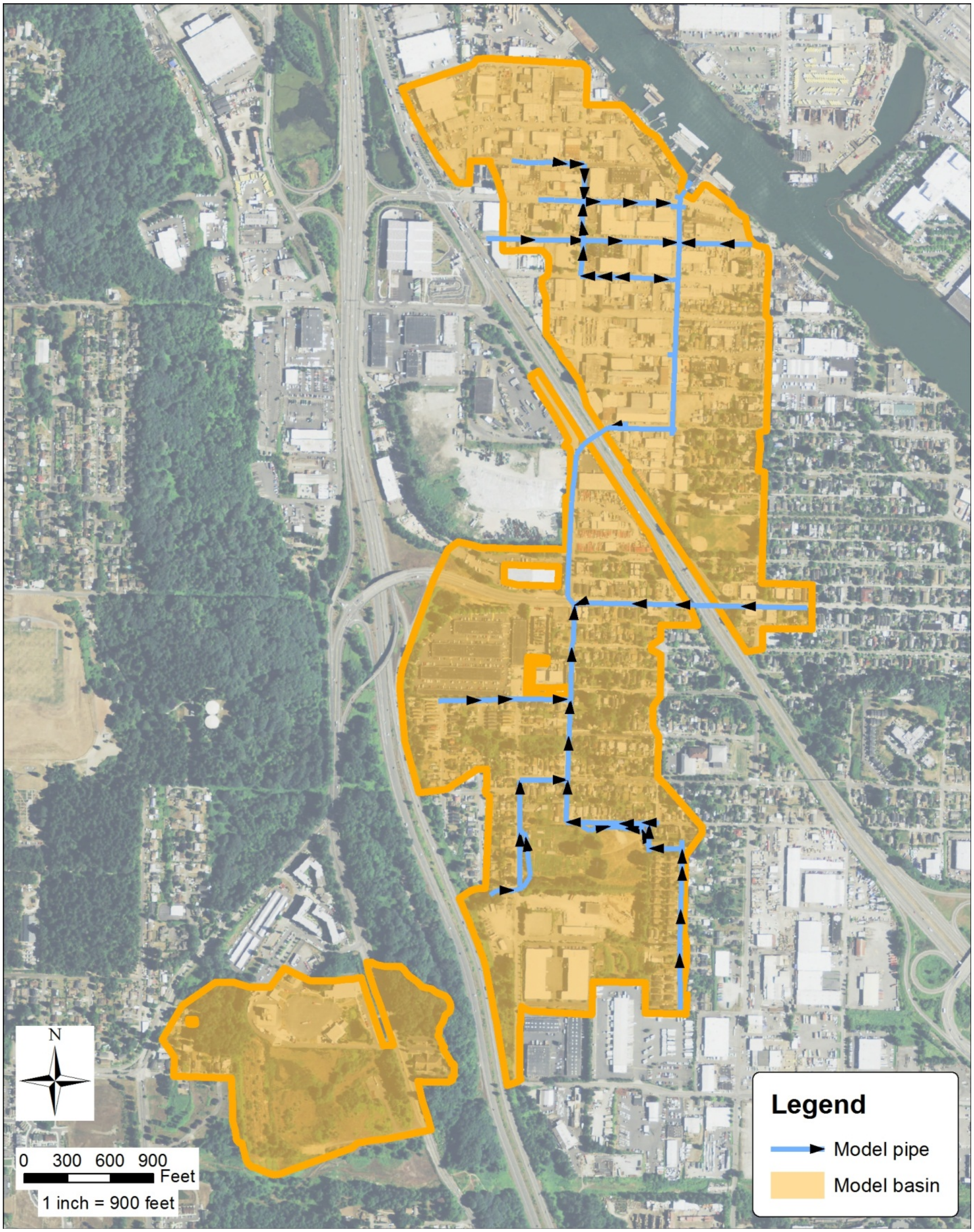

Figure 1-1. South Park Drainage Area (background imagery from USDA-FSA-APFO NAIP MrSID Mosaic-USDA/FSA – Aerial Photography Field Office, 2013)

SECTION 2

Basin Characterization

This section describes the stormwater conveyance system, climate, land use, tidal condition, and soils in the South Park Drainage Area.

2.1 Stormwater Conveyance System

The South Park hydraulic model contains approximately 17,000 feet of storm drains and about 700 feet of trapezoidal channels. Storm drains range from 8 to 72 inches in diameter. Table 2-1 provides a summary of the modeled system.

Table 2-1. Summary of Drain Pipe in the South Park Drainage Area

Diameter (in.)	Length of pipe (ft) [a]	Percent of total
8	80	< 1
12	5,203	29
15–18	4,935	28
20–21	1,173	7
24	662	4
30	699	4
42	364	2
54	291	2
72	3,581	20
Trapezoidal	718	4
Total	**17,705**	**100**

The 72-inch-diameter portion of the storm system drains to a discharge to the Duwamish River near 7th Avenue S and S Holden Street. The invert elevations of the storm drain in this area are below -2 feet and the ground surface elevations range from about 10 to 12 feet. All elevations are on the North American Vertical Datum 1988 (NAVD88) datum. A tide valve has been installed near S Riverside Boulevard to prevent back-flow from high tides into the basin.

The model contains storm drains in S Chicago Street, S Holden Street, S Austin Street, and 5th Avenue S that are not in the current SPU geographic information system (GIS) data. Similarly, the model contains drains and trapezoidal channels in the Marra Farm and S Director Street areas that are not in the GIS. Other differences between the model and GIS data exist.

2.2 Climate

Seattle typically has moderate, dry summers and mild, wet winters. Regional climate data are reported at Seattle-Tacoma International Airport (Sea-Tac). Average annual precipitation is 37.1 inches.

The Seattle area experiences three distinctive categories of storm types (MGS, 2003), as described below:

1. Short-duration storms are primarily warm season events that produce high intensities over isolated areas; they are often the controlling storm types for sizing conveyance structures in urbanized areas. Such high-intensity storms produce rapid high runoff that can fill the storm drain system in minutes.

2. Intermediate-duration storms occur throughout the year but are most common in the fall and early winter seasons. These storms often contain moderate to high intensities for a period of several hours and precipitation commonly occurs over 6 to 18 hours. Such events can result in high levels in the storm drain depending on how they align with the tidal cycle.

3. Long-duration storms are associated with continental-scale water systems originating over the Pacific Ocean and precipitation occurs over very large areas. Long-duration storms are primarily late fall and winter season events, characterized by low to moderate intensities and durations of 24 hours or more. As these events may span one or more tidal cycles, their impact on levels in the storm drain system will depend on the general tidal levels (particularly low tides, which allow gravity discharge) and the occurrence of peak intensities aligned with high tides.

In addition to the regional climate data reported at Sea-Tac, the City of Seattle operates a network of rain gauges (RGs) across the city. The closest gauges to the South Park Drainage Area are RG17 to the west and south and RG16 to the east and north. The locations of these rain gauges and others in SPU's network are shown in Figure 2-1. The Thiessen polygon boundary between RG16 and RG17 aligns approximately with W Marginal Way S, the dividing line between the upper and lower basins. For analyses using existing rainfall (1978–2014), the RG17 data are used for the upper basins and RG16 data are used for the lower basin.

SPU also utilizes a 158-year synthetic rainfall series for analysis of service level. This series, developed by MGS Environmental Consultants, consists of hourly data from rain gauges at Sea-Tac; Salem, Oregon; and Vancouver, British Columbia, disaggregated to 5-minute time steps, and contains storms meeting a full range of expected frequency of occurrence over a full range of event durations. This time series, together with a matched tidal level time and evapo-transpiration series, was used for confirmation of the service level. The sources and time spans of the data in this series are provided in Table 2-2.

Table 2-2. Summary of 158-year Rainfall Series

Rain gauge	Start date	End date
Sea-Tac	10/1/1939	9/30/1999
Salem, Oregon	10/1/1939	9/30/1999
Vancouver, British Columbia	10/1/1960	9/30/1997

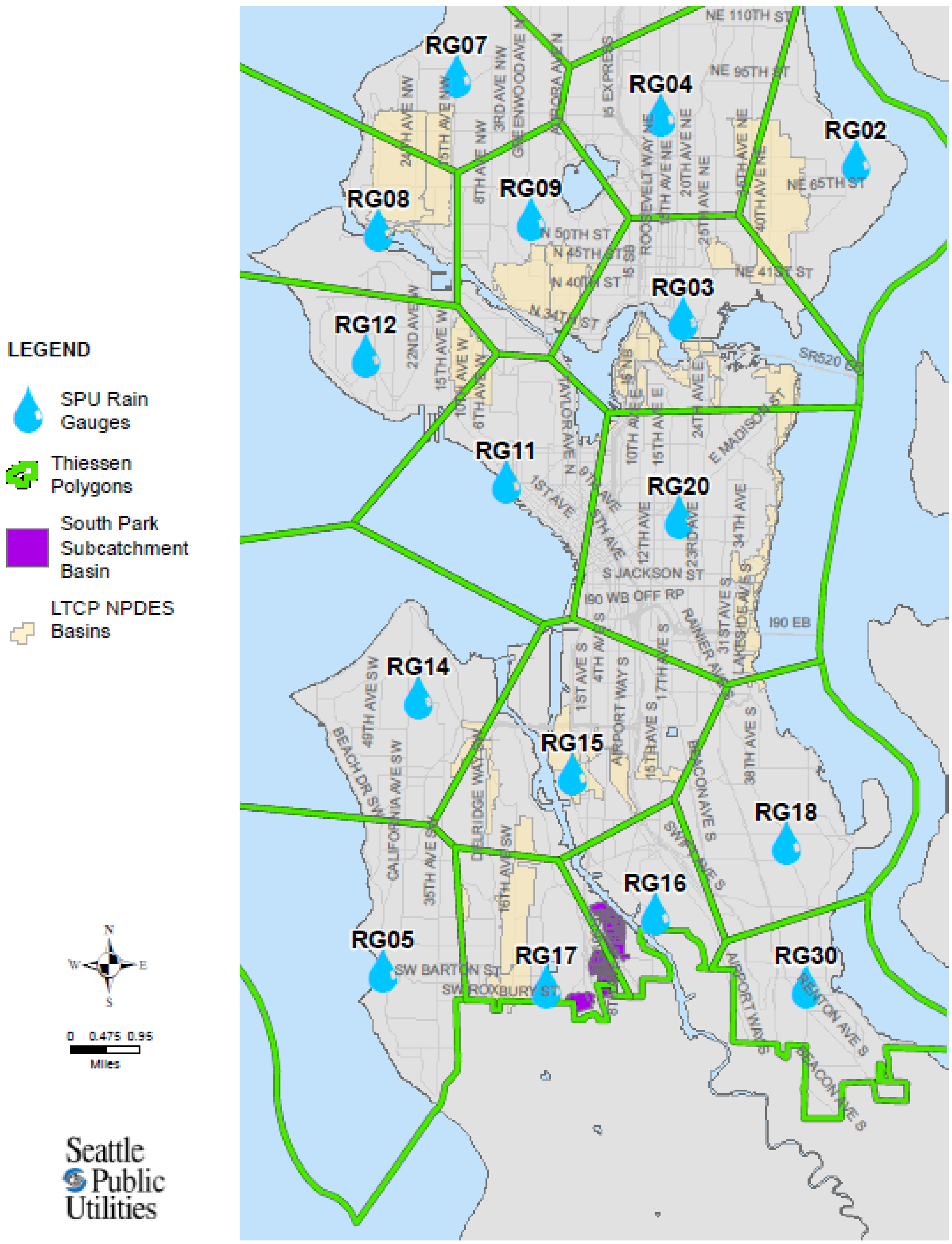

Figure 2-1. SPU rain gauge network, Thiessen polygons, and South Park Drainage Area

2.3 Land Use

Land use in the basin consists of light to heavy industrial with some moderate-density residential in the lower basin. The lower basin is expected to continue in mostly industrial use. The upper basin land use currently consists of some undeveloped land, moderate-density residential, and some light to heavy industrial uses.

The undeveloped and lightly developed area in the upper basin includes the Marra Farm, some areas near the south border of the basin, and a 45-acre tract west of SR 509 (subcatchment B-Inlet#1). With the exception of the Marra Farm area, these are expected to become highly developed in the future.

The existing model area of 238 acres contains about 145 acres of impervious area or 61 percent impervious. This assumes conversion of some currently undeveloped land to industrial use. The future basin is assumed to add about 40 acres of total area where storm drains could potentially be added. The future basin is also assumed to increase in imperviousness by further development, and conversion of the residential area to high-density residential or industrial use. The general level of imperviousness is also assumed to increase throughout the basin. Taken together, the future basin is assumed to have about 219 acres of impervious surface or 79 percent imperviousness. The subcatchment characteristics included in the model are contained in Appendix A.

2.4 Tidal Conditions

Tidal elevations currently range from about -7 feet North American Vertical Datum of 1988 (NAVD) to about +12 feet. High tides generally range from 10 to 12 feet with a maximum of about 12.2 feet. The National Oceanic and Atmospheric Administration's (NOAA) hourly tide elevations for Seattle are shown in Figure 2-2.

To account for climate change uncertainties, the existing basin analysis was conducted assuming an increase in the tide level of 2 feet. For analysis of the future basin, the tide was assumed to rise by 4 feet above the existing level.

2.5 Soils

The original South Park model was constructed using Horton Infiltration parameters. The soil types assumed in the original model ranged from recessional and alluvial outwash in the upper basin to fill in the lower basin. In the process of converting the model from XPSWMM to PCSWMM, Aqualyze converted the infiltration method from Horton to Green-Ampt. Green-Ampt parameters were selected to mimic the results from the Horton method.

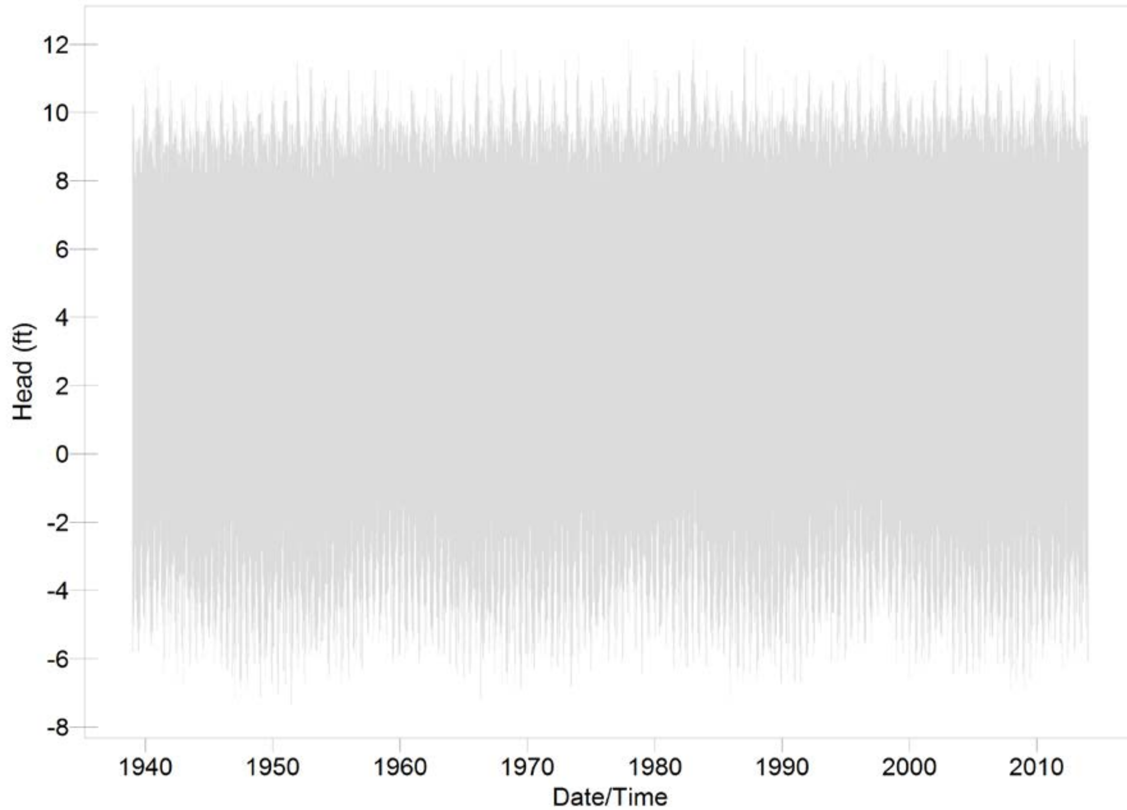

Figure 2-2. Hourly tidal elevations from 1939 through 2013

(NOAA gauge 9447130, NAVD datum)

SECTION 3

Model Modifications

As noted previously, the original South Park Drainage Area model has been subject to changes by various entities. This section discusses the revisions made by Brown and Caldwell for this latest round of modeling.

3.1 Base Model

Brown and Caldwell received the model Portland St_90pct_Design_Option4a.inp from SPU to begin this latest modeling project. This model was created for design of new storm drains on S Portland Street (Kennedy/Jenks, 2013). As a part of that project, the original model was converted from the XPSWMM platform to the PCSWMM platform using the EPA-SWMM5 v22 (SWMM5) engine.

The revised model includes the proposed S Portland Street storm drains. In addition, the infiltration method was changed from the Horton approach to the Green-Ampt approach.

3.1.1 Storm Drains and Maintenance Holes

MHs in the base model were labeled alphabetically starting in the upper basin and progressing downstream. The letter was followed by a feature—often a street name with some truncation (P-Cloverda for example). The corresponding SPU GIS label (D078-122 for P-Cloverda for example) has been added to the DESCRIPTION fields of the model at the principal MHs. This labeling scheme has been retained except where new features were added as discussed below.

Storm drains in the base model were labeled using the upstream and downstream MH letters (P-Q for the storm drain immediately downstream of MH P-Cloverda for example). This labeling scheme has generally been retained.

3.1.2 Subcatchments

Subcatchments were created in the base model to provide estimates of surface runoff to the MHs of the storm drain network. The subcatchments were labeled by the MH to which surface runoff was directed followed, by a # and a number to identify multiple subcatchments connected to a single MH (P-Cloverda#1_1 for example). The source from which the subcatchments were taken is included in the TAG fields of the model.

Surface runoff from the subcatchments is generated in the model using the standard SWMM5 methodology dependent on the parameters assigned to each subcatchment. The runoff so generated is connected directly to the storm drain network at the MH assigned for the subcatchment. This approach essentially assumes that the drainage system is built out sufficiently to efficiently deliver the runoff to the existing storm drains. Thus, any surface flooding that may occur today due to poor drainage does not occur in the model and all flows are delivered to the existing storm drains. As constructed, the model is not capable of simulating surface flooding that may currently occur. This approach also ignores restrictions to the entry of flow into the system through catch basins that may not be able to accept the peak flows predicted by the model.

3.2 Modifications to the Base Model

Brown and Caldwell made several modifications to the Base Model to improve performance and stability, and to update the subcatchments. These modifications are described below.

3.2.1 Tide Gate

A Tideflex TF-1 duckbill gate valve was installed in 1988 in the back-flow prevention vault near the end of the storm drain. This valve was replaced in 2011 after it had been damaged. The Base Model (from the original XPSWMM model) represented this valve as an orifice with a check valve. The difference in hydraulic calculations made by the two engines required that the duckbill valve be represented as an Outlet element with a curvilinear representation of the head loss characteristics. The head loss characteristics of the valve provided by Tideflex Technologies are shown in Figure 3-1.

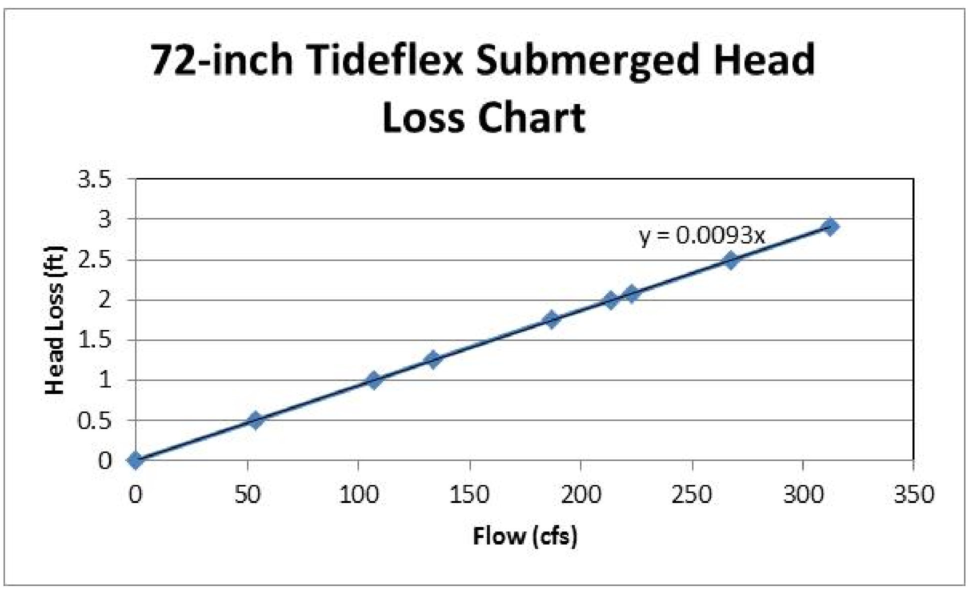

Figure 3-1. Tideflex valve head loss characteristic

3.2.2 Subcatchments

The Base Model contained the runoff-generating subcatchments that were included in the original model. Brown and Caldwell received a revised set of subcatchment polygons from SPU in April 2014 representing the latest estimation of areas that could contribute surface runoff to the storm drains. This revision resized some subcatchments, deleted one subcatchment, and added two subcatchments that were assumed to be part of the future basin in the original modeling work. In all, these revisions increased the overall size of the basin from about 230 acres to 238 acres. The imperviousness of the subcatchments ranges from 10 to 90 percent, with a basin-wide average of 61 percent.

Figures 3-2 through 3-6 show the subcatchments and storm drains included in the model.

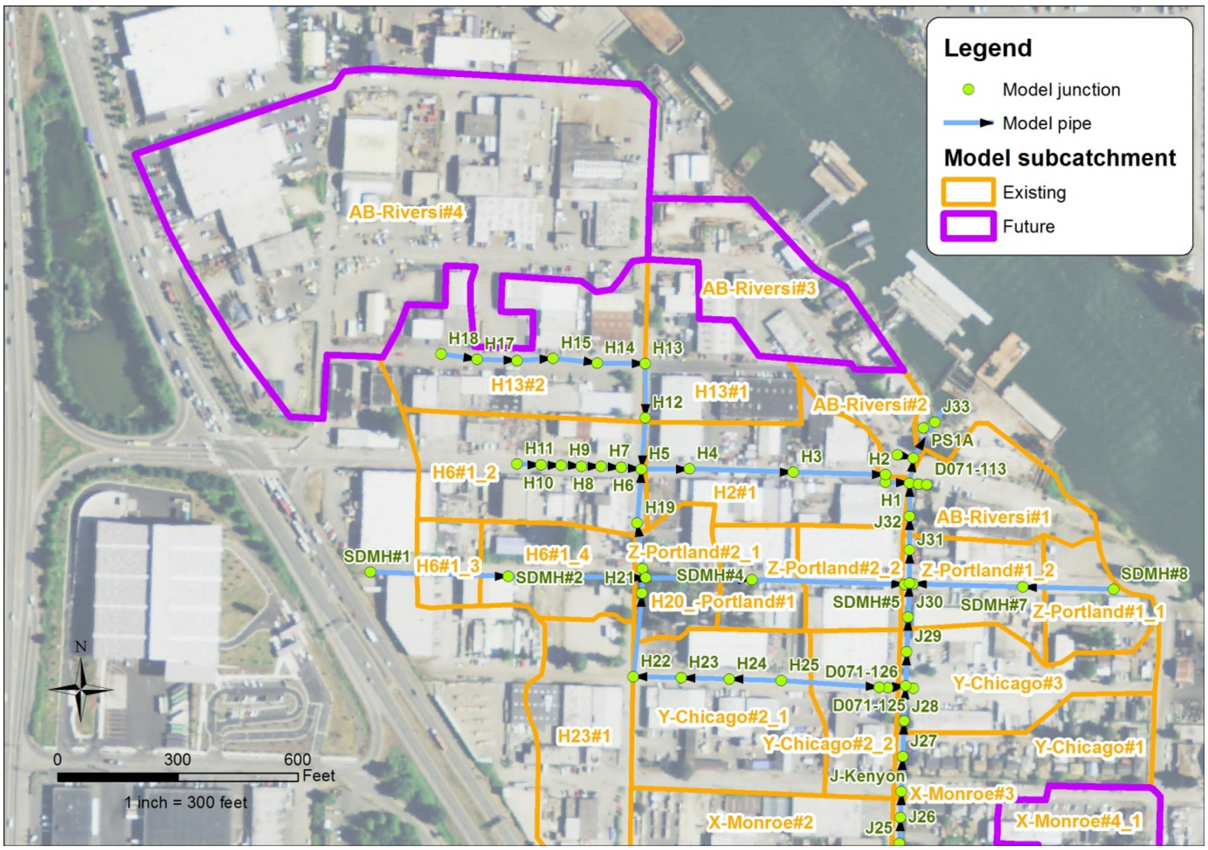

Figure 3-2. Model subcatchments and drains in the Lower Basin north of S Monroe Street

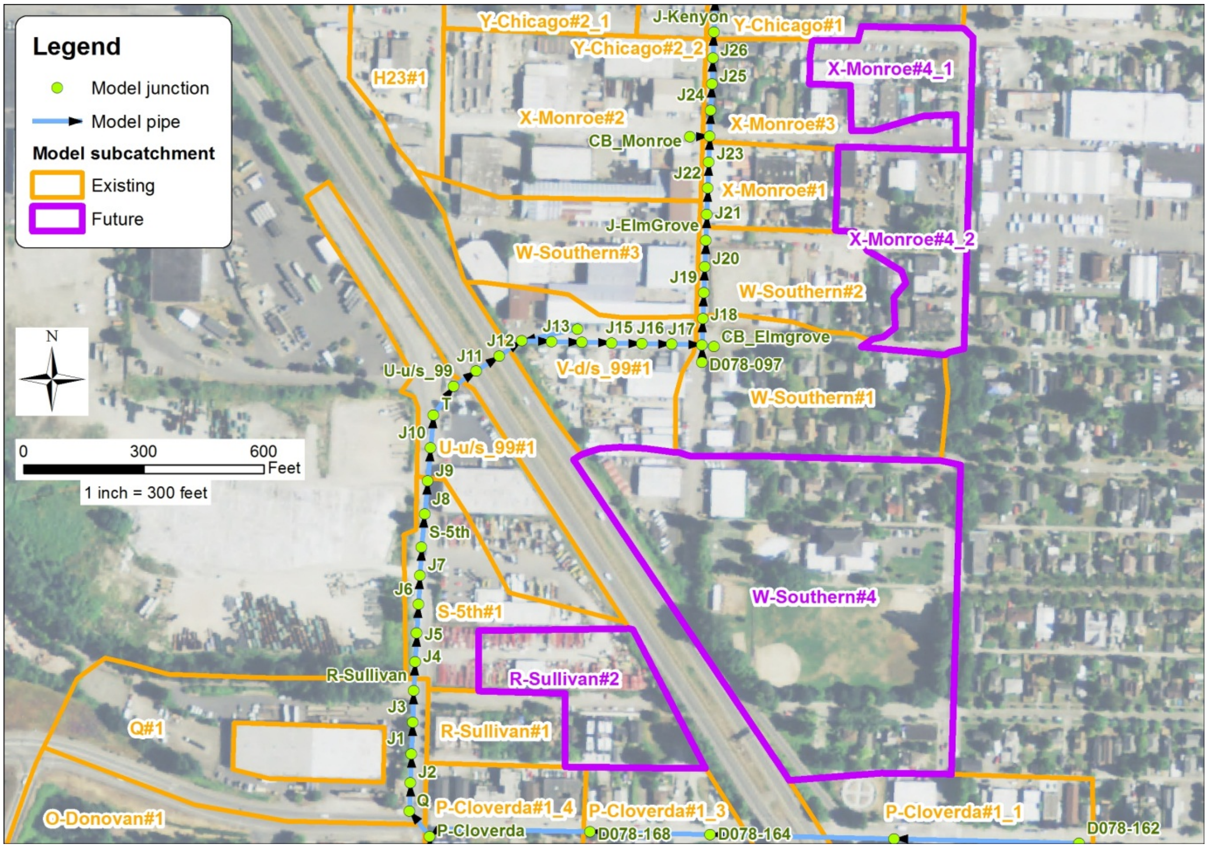

Figure 3-3. Model subcatchments and drains in the Middle Basin, S Cloverdale to S Chicago Street

Figure 3-4. Model subcatchments and drains in the Upper Basin, S Sullivan to S Director Street

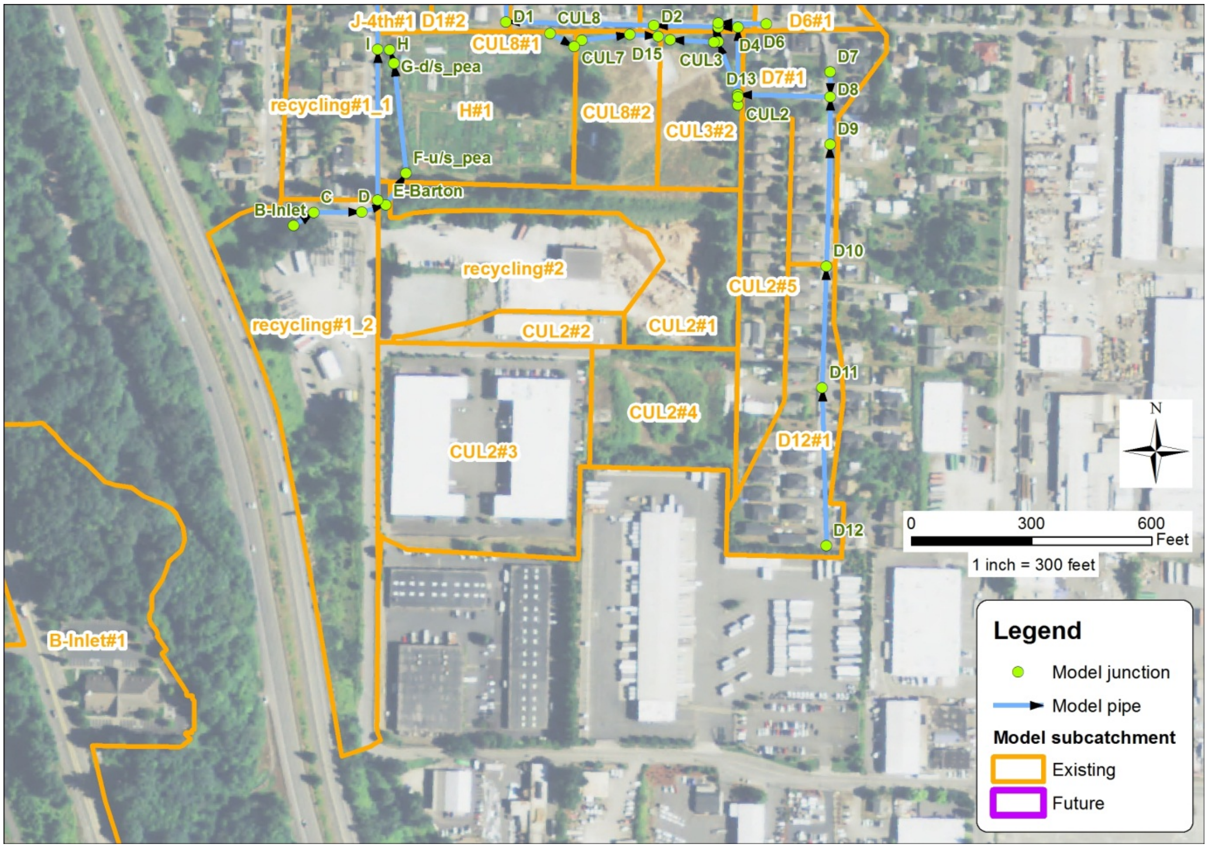

Figure 3-5. Model subcatchments and drains in the Upper Basin, S Director to S 96th Street

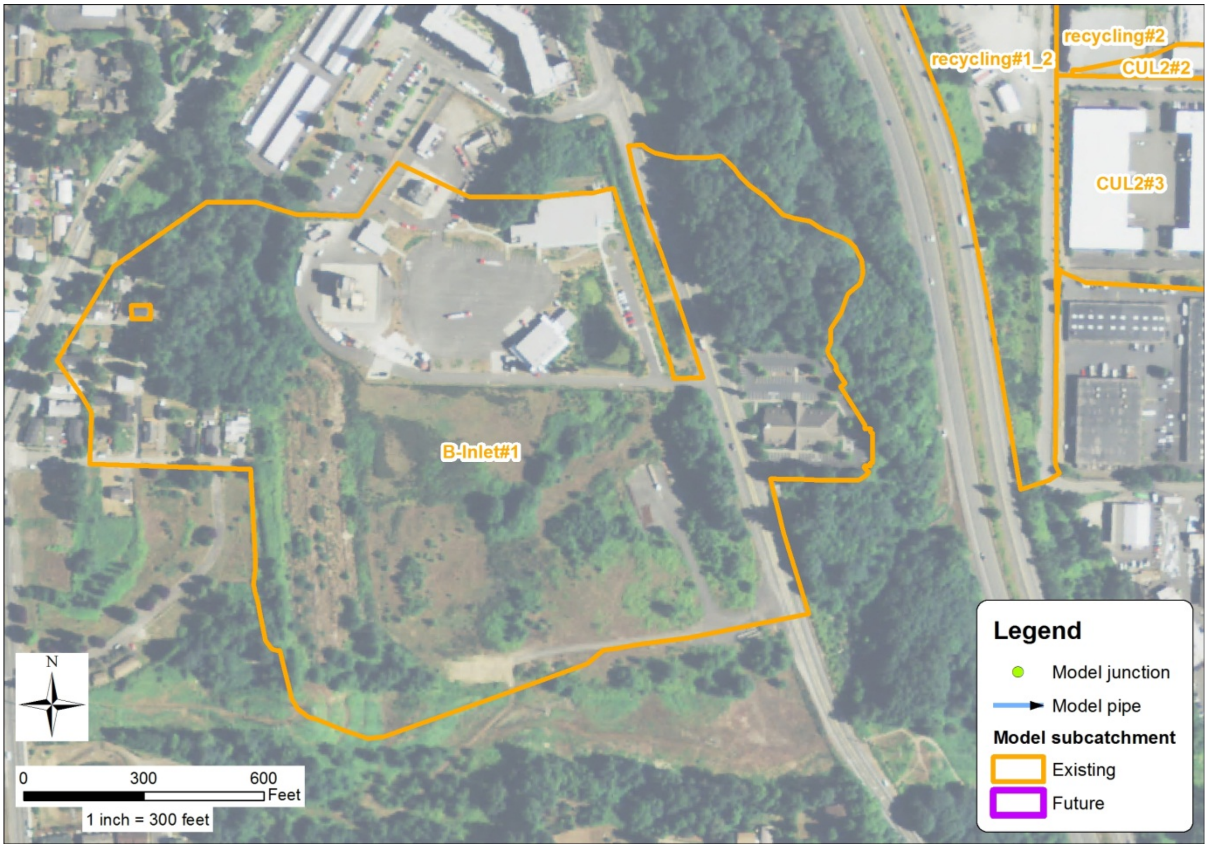

Figure 3-6. Model subcatchment B-Inlet#1 west of SR 509 bisected by SW Roxbury Street

3.2.3 Connections on S Portland Street

The Base Model shows storm drains from S Chicago Street connecting to the new S Portland Street drains at MH SDMH#3. SPU indicated that this connection was not to be implemented. Accordingly, the connection was eliminated and the S Chicago Street drainage now proceeds north to join the S Holden Street drains.

3.2.4 Pipeline Profile

The profile of the 72-inch-diameter storm drain in the upper basin (E Marginal Way crossing to S Cloverdale Street) had a profile with unexpected changes in elevation that were not exhibited in the GIS data. These were smoothed out so that the profile is on a constant slope.

The original and Base Model have invert elevations of the 72-inch-diameter drain in the lower basin about 0.4 foot lower than the GIS. This is generally consistent with the survey results reported in the description field of the Base Model at MH Z-Portland (D071-117).

3.2.5 Pipeline Lengths

The original and Base Model have lengths of the 72-inch-diameter storm drain ranging from about 250 to over 500 feet. Experience in the SPU Long-Term Control Plan modeling indicates that long conduit lengths in large drains subject to storage can result in discontinuity of flow calculations. This has been remedied by inserting dummy nodes along the segment length to decrease segment length. Accordingly, each 72-inch-diameter section was divided into as many smaller segments as possible without SWMM5 increasing the length to satisfy computational requirements. This resulted in the addition of a number of dummy nodes and pipe segment lengths of about 65 to 90 feet. The added nodes are labeled with a J followed by a number; J6 for example.

3.2.6 Node Surface Elevations

The original and Base Model had several low MHs with rim elevations below the anticipated maximum hydraulic grade line (HGL) elevation at locations away from the main 72-inch-diameter drain in the lower basin. Several of these had rim elevations of 5 feet or less, which is considered unreasonable. In the Base Model, these were configured with a Surcharge Depth, which, according to the SWMM5 manual, allows the HGL to rise above the rim without overflow. It was discovered, however, that use of a Surcharge Depth inappropriately limited the maximum HGL computed in downstream segments of the 72-inch-diameter drain in the reaches where storage occurs. Accordingly, these low MHs were given a rim elevation (by changing the maximum depth specification for the MHs) consistent with the GIS or immediately surrounding MHs. A Ponding Area was then assigned to these MHs in place of the Surcharge Depth. The Ponding Area stores water on the surface if the HGL exceeds the rim elevation, allowing the HGL to rise above the rim consistent with the ponding area assigned. Assignment of a ponding area to MHs is a standard requirement of the SPU modeling guidelines. The model assigns a ponding area to every MH in the tributary area.

3.2.7 Added Drains

The 18-inch-diameter storm drain in S Cloverdale Street was added to the model from GIS data (MH D078-169 to MH D078-162). This was done to provide a more accurate way to introduce surface runoff from the associated subcatchments into the model, and to facilitate comparisons of the model predictions to recently acquired flow data.

3.2.8 Modified Pipe Diameter

In the base model, model conduit between MHs K-Hendersn and L-Concord (see Figure 3-4) (GIS ID D078-127_D078-126) was specified with an 18-inch diameter. Based on information from SPU taken from record drawings, this conduit diameter was increased to 24 inches.

SECTION 4

Modeling Results

This section documents the studies performed with the model revised as described above.

4.1 Comparison to Flow Metering Data

SPU installed flow meters in April 2014 reporting flow in storm drains on S Cloverdale Street (MH D078-168) and on 5th Avenue S (MH D078-125). A comparison of the observed and model-predicted flows during the monitoring period is discussed below.

4.1.1 S Cloverdale Street MH D078-168

Figure 4-1 presents the observed and model-predicted flows at MH D078-168 following installation of the flow meter. The fit of the model output to the meter data is accurate, indicating that no changes are required to the model parameters.

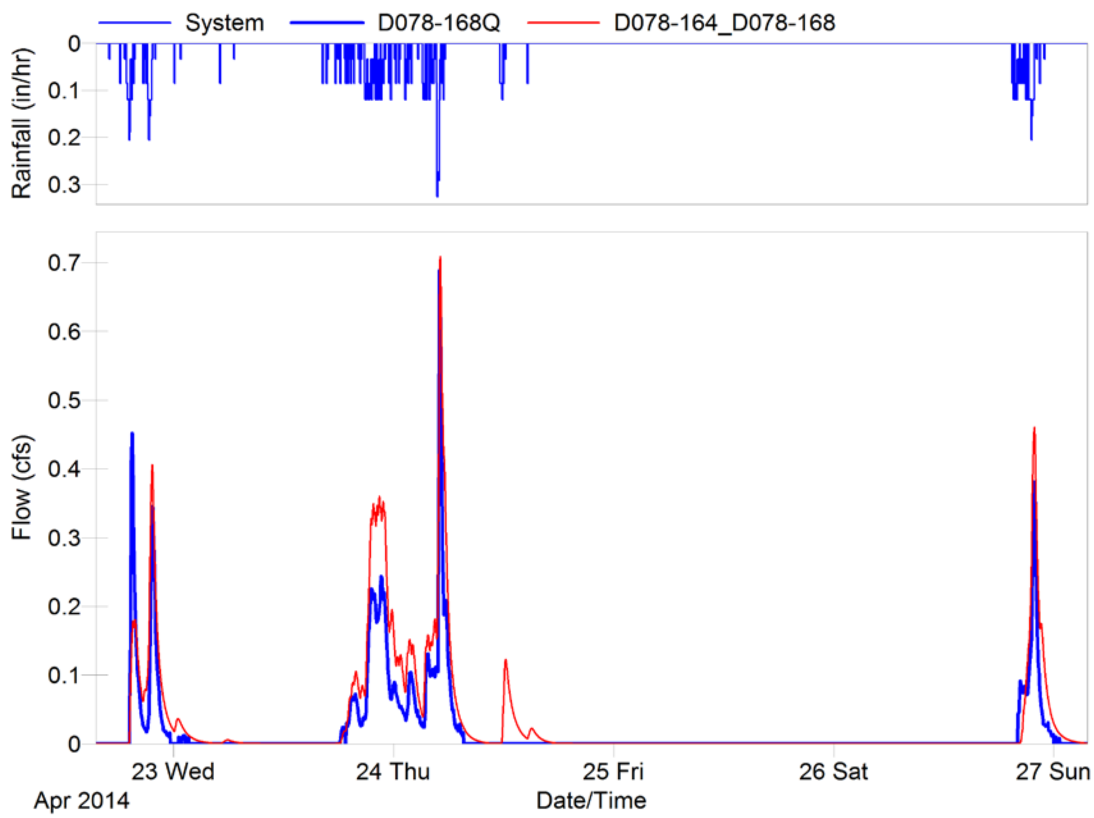

Figure 4-1. Comparison of observed (blue line) with model-predicted flow (red line) at MH D078-168

4.1.2 5th Avenue S MH D078-125

At this meter (reporting flow in conduit D078-126_D078-125), the model significantly over-predicted the flows when the full imperviousness of the upstream subcatchments was considered to be hydraulically connected to the storm drains. This suggests that, unlike the S Cloverdale system, the surface drainage in this area is not well developed, and runoff from impervious surfaces is ponding on the surface or being directed to pervious areas where it infiltrates.

The model was adjusted to route between 60 and 100 percent of the impervious area runoff in the catchments upstream of S Trenton Street to the pervious areas. This simulates portions of the impervious surfaces being drained to pervious areas where it infiltrates or ponds and is not connected directly to the storm drains in the model.

Figure 4-2 presents the observed and model-predicted flows at MH D078-125 during the monitoring period. Some groundwater response is evident in the observed data, which is not simulated in the model. The flow peaks and volumes predicted by the model are considered accurate, confirming the low effective imperviousness in the tributary area.

Because it is anticipated that the surface drainage will be improved in the future, the model studies for the flood control pump station consider the impervious areas in this and all other portions of the basin to be 100 percent hydraulically connected to the storm drains in the model.

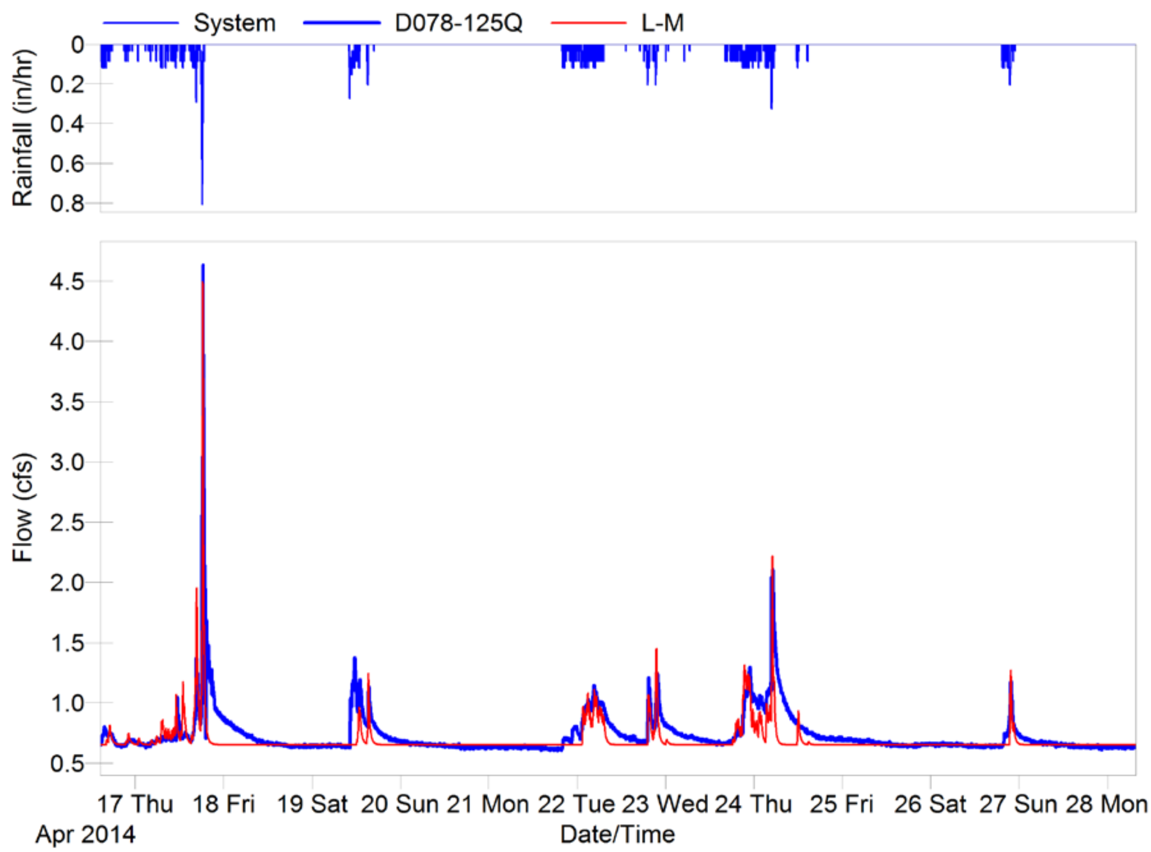

Figure 4-2. Comparison of observed (blue line) with model-predicted flow (red line) at MH D078-125 after routing 60 to 100 percent of upstream imperviousness to pervious areas

4.2 Current Flooding

The revised model with existing tide levels and the flood control pump station disabled was run for the period 1996 through 2013. This run was made to confirm the flooding frequency of 2 to 3 times per year reported by local businesses near S Portland Street and 7th Avenue S. The HGL information will support an analysis by SPU of the extent of flooding in the current condition. This simulation assumes that 100 percent of the imperviousness in the basin is hydraulically connected to the storm drains.

The model results were queried for HGL values greater than elevation 10.58 feet, which is the apparent elevation of a catch basin shown on the Plan and Profile Drawings for the West Duwamish Trail project (SPU vault 777-988) just east of 7th Avenue S (MH SDMH#6). Table 4-1 shows the events and maximum HGL elevations from this analysis. In summary, the HGL equaled or exceeded the rim elevation of the catch basin 42 times in the 18-year simulation period, for an occurrence of 2.3 times per year on average. This frequency reflects the tide level. Figure 4-3 shows the event count in each year of the simulation.

Table 4-1. Events with HGL Level Greater than 10.6 feet		
Date	Duration (h)	Maximum head (ft)
12/16/1997	2.1	11.64
12/3/2007	3.4	11.55
2/4/2006	1.5	11.54
1/30/2006	2.6	11.44
1/2/2003	12.5	11.41
12/19/2012	26.0	11.31
1/4/2003	1.8	11.28
1/13/1998	25.7	11.27
11/19/2012	2.5	11.27
1/5/1998	2.3	11.26
11/23/2011	26.4	11.26
1/5/2006	0.7	11.23
1/2/1997	1.8	11.20
1/7/2002	2.3	11.15
11/21/2012	1.2	11.07
12/1/2012	2.3	11.06
11/17/2009	1.9	11.04
3/29/2010	0.5	11.02
2/12/1998	1.7	10.99
11/4/2006	1.2	10.96
3/22/2003	1.8	10.95
11/22/2001	2.0	10.94
12/21/2009	1.4	10.94
1/4/2010	1.6	10.92
1/28/2006	1.7	10.90
11/20/2009	0.8	10.88
1/16/1998	1.7	10.87
11/22/2009	1.2	10.83
2/8/1996	1.3	10.82
12/23/2006	1.3	10.80
1/20/1996	15.0	10.77
2/28/2006	0.8	10.74
1/8/2008	0.3	10.73
12/19/2007	1.3	10.70
1/1/2010	0.8	10.69
12/24/2003	0.7	10.68

Table 4-1. Events with HGL Level Greater than 10.6 feet		
Date	**Duration (h)**	**Maximum head (ft)**
11/29/1998	0.1	10.67
12/12/2010	0.8	10.66
11/25/1998	0.3	10.63
11/14/2001	0.3	10.63
1/12/2010	0.3	10.62
12/9/2010	0.3	10.62

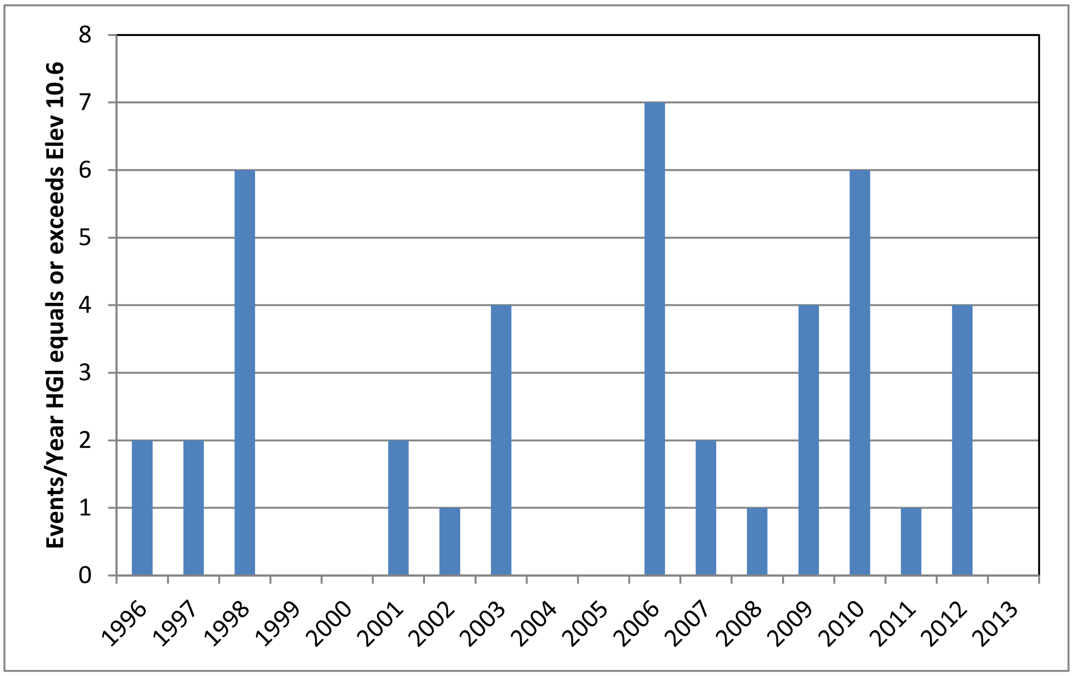

Figure 4-3. Annual counts of HGL exceeding elevation 10.58 feet at 7th Avenue S and S Portland Street

4.3 Flood Control for Existing System

A series of runs were conducted varying installed flood control pump capacity and pump controls to provide an estimate of capacity needed in the flood control pump station to meet the service level. This analysis was conducted with the following assumptions:

1. The tide level was increased 2 feet above the existing level to account for climate change.

2. Four flood control pumps of equal size will be installed. One of these is to be considered a standby pump and should be used for flood control rarely.

3. The existing basin of 238 acres has an average imperviousness of 61 percent. The drainage system is also assumed to be built out sufficiently to deliver all surface runoff generated by the subcatchments to the modeled drain system.

4. The start elevations for the pumps should be relatively high to minimize the number of pump starts.

5. The start levels of individual pumps should be approximately 1 foot apart.

Using these constraints, iterative runs were made to select the pump capacity and associated start levels. Through this process, the following configuration was found to result in no more than one exceedance of the service level (maximum water surface elevation of 6.81 feet at the S Holden Street connection) in the 35-year simulation (1978–2013) using actual rainfall and tide raised 2 feet:

- Four 18 cfs flood control pumps (total installed capacity 72 cfs)
- Pump start and stop levels at elevations shown in Table 4-2 (depths over the invert at the connection to the pump station range from 3 to 7 feet)

Table 4-2. Flood Control Pump Start and Stop Elevations

Pump no.	Start elevation (ft)	Stop elevation (ft)
1	0.64	-1.36
2	1.64	-0.36
3	2.64	-0.36
4	4.64	0.64

Using the start and stop elevations for the pumps shown in Table 4-2, model simulations indicated no exceedances of the service level (water surface elevation at S Holden Street of 6.81 feet) in the 35-year simulation using actual rainfall. The maximum water surface elevation in the simulation was 6.6 feet with a corresponding depth above the invert of 8.8 feet, occurring on September 22, 1978. Subsequent simulations indicated that the start level for the first two pumps could be increased by 0.5 foot.

With this configuration, the estimated frequency in average number of days per year each pump was activated is shown in Table 4-3. Note that there were more than 13,000 days in the 35-year simulation and that multiple starts in some days may have occurred. The standby pump was activated only five times in the 35-year simulation, or once every 7 years on average.

The first pump will run frequently to drain the base flow with the assumed start level. The first pump could start every 1 to 12 hours depending on tide and runoff in small events. The control system might be adjusted to decrease the number of starts in dry weather.

Table 4-3. Flood Control Pump Activation Frequency with Assumed Start Levels, Existing System, 35-year Simulation

Pump no.	Activation frequency (per year)
1	--[a]
2	8.66
3	1.51
4	0.14

a. The first pump is activated multiple times per day.

4.4 Level of Service Verification: Existing System

To verify the service level, the model for the existing system was run using the 158-year SPU synthetic rainfall series, the 72 cfs flood control pumping capacity, and the tide level raised 2 feet. The simulation indicated that the pump start elevations in Table 4-2 were sufficient in the vast majority of events, but resulted in 11 exceedances of a water surface elevation of 6.81 feet at S Holden Street in the 158-year period, which does not meet the service level. The number of exceedances can be reduced by lowering the pump start elevations. Subsequent simulations assuming that all four 18 cfs pumps were running when the 72-inch drain was about half full indicated only three events exceeding the service level, well below the seven events required to meet the service level. These results indicate that the identified total station capacity of 72 cfs will achieve the service level goal with the 158-year rainfall series with appropriate pump start elevations. If required, start elevations can be adjusted as the basin drainage is built out and experience is gained with operation.

4.5 Flood Control for Future Basin

As noted in Section 2.3, the future basin was increased by 40 total acres. The increase in impervious surface in the existing and future subcatchments increased the overall imperviousness of the enlarged basin to 79 percent.

Because additional drainage capacity in the lower basin is anticipated in the future, the storm drains in the model on S Chicago Street, S Austin Street, and S Holden Street were enlarged and re-graded similar to the recent additions on S Portland Street. This ensures that stormwater generated in these areas has an unimpeded pathway to the 7th Avenue S drain and the South Park pumping station.

For this analysis, the tide elevation was also increased by 4 feet above the existing level. In addition, a water quality facility was assumed to be in operation with a pumping capacity of 6 cfs. This is a value estimated from early considerations of the design of the water quality facility, and may be revised in the future.

The total pumping capacity required for the future condition depends on how the runoff from the B-Inlet#1 subcatchment in the southwest of the basin (Figure 3-6) is managed. The imperviousness of this 45-acre subcatchment increases from a current 20 percent to 95 percent in the future. The associated increase in runoff would cause flooding at several locations between S Barton Street and S Trenton Street (model MHs I, J-4th, K-Hendersn, and L-Concord; see Figure 3-4). This could be managed by enlarging the affected storm drains or by installing a detention basin to reduce the peak outflow rate from the basin.

To estimate the detention requirement, the runoff from this subcatchment was passed through a storage basin with a pumped outlet. The pump capacity was set to 8 cfs, which is the maximum runoff rate from the current subcatchment in the September 22, 1978, storm that is critical to meeting the service level. With this simplified approach, detention storage of about 108,000 ft^3 was required. With this detention in place, the model indicated that the future capacity of the flood control pump station would need to be 100 cfs (four 25 cfs pumps). This approach not only maintains future flow conditions in the downstream system the same as current conditions, but also improves the performance of the flood control pump station.

The alternative approach of enlarging the affected storm drains would result in a greater total flow capacity requirement for the flood control pump station. This approach was examined in order to provide a conservative capacity estimate. The storm drains affected, together with their existing diameters and required diameters, are shown in Table 4-4.

Table 4-4. Storm Drains Enlarged to Accommodate Increased Flows from Subcatchment B-Inlet#1

Model MH ID[a]	GIS ID	Existing diameter (in.)	Enlarged diameter (in.)
I to J-4th	D078-145_D078-143	18	24
J-4th to K-Hendersn	D078-143_D078-127	18	30
K-Hendersn to L-Concord	D078-127_D078-126	24	36
L-Concord to M-Trenton	D078-126_D078-125	24	36

a. See Figure 3-4 for MH locations.

Using the pump start levels for the first three pumps as indicated in Table 4-2 and with the start level for the fourth pump 1 foot lower (elevation 3.64), it was found that a total flood control pumping system capacity of 120 cfs (four 30 cfs pumps) would result in only one exceedance of the service level in the 35-year simulation. The maximum water surface elevation in the simulation was 9.1 feet with a corresponding depth above the invert of 11.3 feet, occurring on December 14, 1978. This large event (the so-called Hanukah Eve storm) caused an exceedance due to the increase in tide level for the future condition. It is noted that the maximum elevation reached in this event is still below the lowest MH rim in the model (elevation 10.37 feet on S Chicago Street).

Table 4-5 shows the simulation results for the largest three events in the simulation.

Table 4-5. Maximum Depths in the 72-inch Drain at S Holden Street, Future Basin, 35-year Simulation

Rank	Start date	Max elevation (ft)	Max depth (ft)
1	12/14/2006	9.1	11.3
2	9/22/1978	6.8	9.0
3	9/4/1995	5.2	7.4

SECTION 5

Summary and Conclusions

The model for the South Park Drainage Area has been through several modifications. Recently, Aqualyze, Inc. updated the original model previously worked on by R.W. Beck and Associates, Herrera Environmental Consultants, and Brown and Caldwell. Aqualyze converted the updated model, created for analysis of the S Portland Street drain, to the PCSWMM interface using the EPA-SWMM5 v22 engine from the XPSWMM platform.

Brown and Caldwell was provided the S Portland Street model to begin the latest round of analysis for the flood control pump station. Modifications were made to that model as described in Section 3. These include an increase in the basin area and the assumption of a 2-foot rise in the tide level as specified by SPU. The revised model was used to:

1. Estimate the flood control pump station capacity necessary to meet the service level in the existing system and the future basin
2. Estimate the maximum HGL reached in the system without a flood control pump station
3. Confirm the service level for the existing system using the SPU 158-year synthetic rainfall series

Section 4 describes the analyses conducted for each of the above items. A summary of those findings is given below.

5.1 Flood Control Pump Station

For the existing 238-acre basin with 61 percent overall imperviousness and a tide increase of 2 feet, it was found that the installation of four 18 cfs pumps (one of which would be considered emergency standby) would meet the service level in simulations of the 35 years of actual rainfall data (1978–2013). In this simulation, the standby pump was activated only five times in the 35 years of simulation. Startup elevations assumed for the pumps are given in Table 4-2.

The above model was also run using the SPU 158-year synthetic rainfall time series. In this simulation, it was found that the service level could be achieved with four 18 cfs pumps, but only if the startup depths were reduced in the largest events in the record. This indicates that the pump start levels are key to meeting the service level. If required, start elevations can be adjusted as the basin drainage is built out and experience is gained with operation.

For the future basin, the model was run with the 278-acre basin with 79 percent overall imperviousness, a tide level increased by 4 feet from existing, and the water quality facility assumed to be operating with a capacity of 6 cfs. It was found that the total flood control PS capacity would need to be increased from 72 cfs to 100 or 120 cfs, depending on how increased runoff is managed in the upper basin. A single exceedance of the service level was observed in these simulations.

5.2 Existing Surface Flooding Analysis

As described in Section 4.1, the existing model with 238 acres of tributary area at 61 percent overall imperviousness was run for the last 18 years of record without the flood control pump station. This run indicated a frequency of flooding at the intersection of S Portland Street and 7th Avenue S of 2.5 to 3 times per year, respectively. The maximum HGL in this simulation reached about 11.6 feet.

5.3 Conclusions

The following conclusions are drawn as a result of the analyses described above:

1. For the existing 238-acre basin at 61 percent overall imperviousness and a tide level increase of 2 feet, a flood control pump station including four 18 cfs pumps (one considered an emergency standby) was shown to meet the service level. Under certain assumptions, the startup elevation of the pumps may have to be adjusted downward in extreme events. This result was confirmed by running both the last 35 years of actual rainfall, and the SPU 158-year synthetic rainfall series.
2. An analysis of the anticipated future basin of 278 acres at 79 percent overall imperviousness and a tide level increase of 4 feet, the flood control pump station would require installation of four 25 cfs pumps if detention is assumed in the upper basin, or four 30 cfs pumps if the downstream drains were enlarged to accommodate increased runoff. This configuration would meet the service level by exhibiting no more than one exceedance in the last 35 years of actual rainfall record. In addition, the model simulations assumed that the water quality facility was in operation with a separate pumping facility with a 6 cfs capacity. Thus, the total pumping rate assumed was 106 cfs if detention is assumed in the upper basin to 136 cfs if the affected drains are enlarged.
3. The control system for the pumps might have to consider elevation in the 72-inch storm drain, the rate of rise of elevation, tide level, and rainfall in order to minimize pump starts and achieve the service level in extreme events.

SECTION 6

References

Brown and Caldwell, 2007. South Park Pump Station Predesign Project-Hydraulic Modeling Report.

Kennedy/Jenks Consultants, Osborn Consulting Inc., and Aqualyze Inc., 2013. S. Portland Street Drainage Improvements – West Duwamish Trail Preliminary Engineering Report.

R.W. Beck and Associates, 2002. South Park Drainage Study.

Appendix A: Model Subcatchment Characteristics

Name	Historical model source	Rain gauge	Outlet	Area (ac)	Width (ft)	Slope (%)	Existing imperv (%)	Future imperv (%)	Suction head (in.)	Conduct-ivity (in./hr)	Initial deficit (frac.)
AB-Riversi#1	Existing	RG16	AB-Riversi	2.160	200	0.4	90	95	4	0.3	0.03
AB-Riversi#2	Existing	RG16	AB-Riversi	1.136	90	0.2	90	95	4	0.3	0.03
B-Inlet#1	Existing-enlarged	RG17	B-Inlet	44.741	500	4.6	20	95	1.95	4.64	0.01
CUL2#1	Davido's	RG17	CUL2	3.242	250	2	25	25	1.95	4.64	0.01
CUL2#2	Davido's	RG17	CUL2	0.858	300	2	90	90	1.95	4.64	0.01
CUL2#3	Davido's	RG17	CUL2	6.228	400	4	90	90	1.95	4.64	0.01
CUL2#4	Davido's	RG17	CUL2	2.561	280	3.5	90	90	1.95	4.64	0.01
CUL2#5	Davido's	RG17	CUL2	2.304	110	2	60	60	1.95	4.64	0.01
CUL3#1	Davido's	RG17	CUL3	0.746	100	2	60	60	1.95	4.64	0.01
CUL3#2	Davido's	RG17	CUL3	1.883	350	4	10	10	1.95	4.64	0.01
CUL8#1	Davido's	RG17	CUL8	0.649	100	2	60	60	1.95	4.64	0.01
CUL8#2	Davido's	RG17	CUL8	1.880	350	4	10	10	1.95	4.64	0.01
D1#1	Davido's	RG17	D1	2.867	550	5.5	60	60	1.95	4.64	0.01
D1#2	Davido's	RG17	D1	1.381	194	3.1	60	60	1.95	4.64	0.01
D12#1	Davido's	RG17	D12	3.036	186	0.9	60	60	1.95	4.64	0.01
D6#1	Davido's	RG17	D6	1.229	220	6	60	60	1.95	4.64	0.01
D7#1	Davido's	RG17	D7	2.569	90	1	60	60	1.95	4.64	0.01
H#1	Davido's	RG17	H	4.297	311	1.5	10	10	1.95	4.64	0.01
H13#1	Existing	RG16	H13	2.426	140	0.5	90	95	4	0.3	0.03
H13#2	Existing	RG16	H13	4.361	1,320	1.3	90	95	4	0.3	0.03
H2#1	Existing	RG16	H2	3.607	1,300	0.9	90	95	4	0.3	0.03
H20_-Portland#1	Existing	RG16	SDMH#4	1.436	290	0.3	90	95	4	0.3	0.03
H23#1	Existing	RG16	H22	3.795	260	1.7	90	95	4	0.3	0.03

Name	Historical model source	Rain gauge	Outlet	Area (ac)	Width (ft)	Slope (%)	Existing imperv (%)	Future imperv (%)	Suction head (in.)	Conduct-ivity (in./hr)	Initial deficit (frac.)
H6#1_2	Existing	RG16	H5	3.580	252	0.3	90	95	4	0.3	0.03
H6#1_3	Existing	RG16	SDMH#2	0.788	189	0.3	90	95	4	0.3	0.03
H6#1_4	Existing	RG16	SDMH#3	1.952	189	0.3	90	95	4	0.3	0.03
J-4th#1	Davido's	RG17	J-4th	1.748	205	1.8	73.9	73.9	1.95	4.64	0.01
K-Henderson#1	Future-Added	RG17	K-Hendersn	1.785	300	1	58	60	1.95	4.64	0.01
L-Concord#1	Davido's	RG17	T2	4.010	270	1.7	45.6	45.6	3	0.5	0.03
L-Concord#2	Davido's	RG17	L-Concord	3.900	300	4	60	60	1.95	4.64	0.01
L-Concord#3	Davido's	RG17	L-Concord	0.541	120	1.9	60	60	1.95	4.64	0.01
M-Trenton#1	Davido's	RG17	M-Trenton	4.772	370	0.5	32.2	32.2	3	0.5	0.03
M-Trenton#2	Davido's	RG17	M-Trenton	1.839	230	3.6	83.9	83.9	3	0.5	0.03
O-Donovan#1	Existing	RG17	O-Donovan	15.403	440	1	84	91	3	0.5	0.03
O-Donovan#2	Future-Added	RG17	O-Donovan	4.528	300	1	58	91	3	0.5	0.03
P-Cloverda#1_1	Existing	RG17	D078-163	5.365	400	0.1	58	84	3	0.5	0.03
P-Cloverda#1_3	Existing	RG17	D078-168	2.795	400	0.1	58	84	3	0.5	0.03
P-Cloverda#1_4	Existing	RG17	P-Cloverda	2.787	400	0.1	58	84	3	0.5	0.03
Q#1	Existing	RG17	Q	5.654	300	0.7	89	94	3	0.5	0.03
recycling#1_1	Davido's	RG17	J-4th	3.497	263	5.6	60	60	1.95	4.64	0.01
recycling#1_2	Davido's	RG17	D	7.355	263	5.6	60	60	1.95	4.64	0.01
recycling#2	Davido's	RG17	recycling	4.163	250	2	90	90	1.95	4.64	0.01
R-Sullivan#1	Existing	RG17	R-Sullivan	1.454	190	1.1	69	85	3	0.5	0.03
S-5th#1	Existing	RG17	S-5th	2.635	520	0.8	44	59	3	0.5	0.03
T2#1	Davido's	RG17	T2	3.766	272	3.4	50	50	3	0.5	0.03
T2#2	Davido's	RG17	T2	0.378	50	2	60	60	3	0.5	0.03
T4#1	Davido's	RG17	T4	3.546	272	2.3	60	60	3	0.5	0.03

Name	Historical model source	Rain gauge	Outlet	Area (ac)	Width (ft)	Slope (%)	Existing imperv (%)	Future imperv (%)	Suction head (in.)	Conduct-ivity (in./hr)	Initial deficit (frac.)
T5#2	Davido's	RG17	T5	4.142	241	5.4	60	60	3	0.5	0.03
U-u/s_99#1	Existing-enlarged	RG17	U-u/s_99	3.881	260	0.6	58	59	3	0.5	0.03
V-d/s_99#1	Existing	RG16	V-d/s_99	3.320	790	0.3	89	94	4	0.3	0.03
W-Southern#1	Existing	RG16	W-Southern	4.749	300	0.6	50	94	3	0.5	0.03
W-Southern#2	Existing	RG16	J-ElmGrove	2.586	230	0.8	73	80	4	0.3	0.03
W-Southern#3	Existing	RG16	J-ElmGrove	3.751	300	0.3	89	94	4	0.3	0.03
X-Monroe#1	Existing	RG16	X-Monroe	1.653	210	0.7	86	90	4	0.3	0.03
X-Monroe#2	Existing	RG16	X-Monroe	6.094	400	0.2	90	95	4	0.3	0.03
X-Monroe#3	Existing	RG16	X-Monroe	2.360	270	0.2	89	94	4	0.3	0.03
Y-Chicago#1	Existing	RG16	Y-Chicago	2.852	110	0.1	90	95	4	0.3	0.03
Y-Chicago#2_1	Existing	RG16	H23	4.242	411	1.4	90	95	4	0.3	0.03
Y-Chicago#2_2	Existing	RG16	D071-125	1.875	297	1.4	90	95	4	0.3	0.03
Y-Chicago#3	Existing	RG16	CB_Chicago	2.834	260	0.1	90	95	4	0.3	0.03
Z-Portland#1_1	Existing	RG16	SDMH#7	1.398	235	0.1	90	95	4	0.3	0.03
Z-Portland#1_2	Existing	RG16	SDMH#6	1.892	235	0.1	90	95	4	0.3	0.03
Z-Portland#2_1	Existing	RG16	SDMH#4	1.251	389	0.1	90	95	4	0.3	0.03
Z-Portland#2_2	Existing	RG16	SDMH#5	1.504	468	0.1	90	95	4	0.3	0.03
AB-Riversi#3	Future	RG16	AB-Riversi	3.205	500	0.5	NA	90	4	0.3	0.03
AB-Riversi#4	Future_buildout	RG16	AB-Riversi	16.246	500	0.5	NA	90	4	0.3	0.03
R-Sullivan#2	Future_buildout	RG17	R-Sullivan	2.832	500	0.5	NA	95	3	0.5	0.03
W-Southern#4	Future_buildout	RG16	W-Southern	12.726	500	0.5	NA	35	3	0.5	0.03
X-Monroe#4_1	Future_buildout	RG16	X-Monroe	1.935	500	0.5	NA	90	4	0.3	0.03
X-Monroe#4_2	Future_buildout	RG16	X-Monroe	2.938	500	0.5	NA	90	4	0.3	0.03