Section 6: Collection System Evaluation

6.1 Collection System Development

A key element of understanding the capacity of the City's collection system under existing and future sewer flows involved the development of the collection system hydraulic model. AquaTwin Sewer Modelling software was used to develop the hydraulic model. The development of the hydraulic model involved several steps including importing the City's existing GIS sewer geodatabase, reviewing model connectivity, calculating and assigning existing and future flows to the model, and calibrating the model using flow meter data. This section provides an overview of the hydraulic model development, model calibration process, comprehensive capacity analysis, and modeling results.

6.2 Hydraulic Model Development

The City's hydraulic modeling needs were assessed to help define the software and approach needed in the development of the hydraulic model. The wastewater collection system model was developed in AquaTwin Sewer Version 15.0. The hydraulic model horizontal datum is based on the Washington State Plane Coordinate System, North American Datum (NAD 83).

The preferred approach was to develop a trunk model that includes all pipes with diameters greater than or equal to 10-inches within the existing collection system. A trunk model is a skeletal model which is less detailed than a full pipe model. These types of hydraulic models are used for high level decision-making and system planning. This approach was selected based on the level of detail and analysis required for this WWCP.

The model uses Manning's equation to calculate friction losses throughout the system with a Manning's roughness coefficient of 0.013. The model does not include cleanouts and fittings included in the City's GIS sewer geodatabase. Portions of the 8-inch pipes were added to the model to assure proper connectivity and stability for running the model.

6.2.1 Model Approach, Assumptions and Update

During model development, the physical network of sewers, manholes, and pump stations was established based on the City's GIS database, specifically the physical information contained therein for the existing collection system. A review of the existing collection system, parcel, and land use information in the GIS database was completed prior to model development which provides quality enhancements to the sanitary GIS data. The data review was undertaken in GIS in both ESRI ArcMap and hydraulic modeling software utilizing built-in AquaTwin Sewer analysis tools.

A review was conducted of the hydraulic continuity and suitability of physical sewer data in terms of profile connectivity. The following approach was taken where information, such as diameters and inverts, were not available for conduits:

- When inverts were missing in intermediate gravity sections where upstream and downstream information was known, the inverts were interpolated using the known inverts as well as the length of the conduit with missing invert information.
- When only downstream inverts were available, the upstream inverts were calculated using the known downstream invert, the length of the conduit, and an assumed constant slope.
- When connectivity information was completely missing, appropriate assumptions were made based on engineering judgement and consultation with the City's staff to build the network.
- For manholes for which information was unknown, a rim elevation was taken from Google Earth and verified using the latest topographic data available from United States Geological Survey (USGS).
- For manholes for which rim elevation was populated based on NGVD 29 and the former City of Bremerton datum; approximately 112 feet was subtracted to covert to NAVD 88.
- South Bremerton has been defined for this WWCP to include the Gorst, Anderson Hill Road, and PSIC basins as shown in the inset in Figure 6-1.

Figure 6-1 shows the collection system hydraulic model elements, basin boundaries, and the locations of the pump stations.

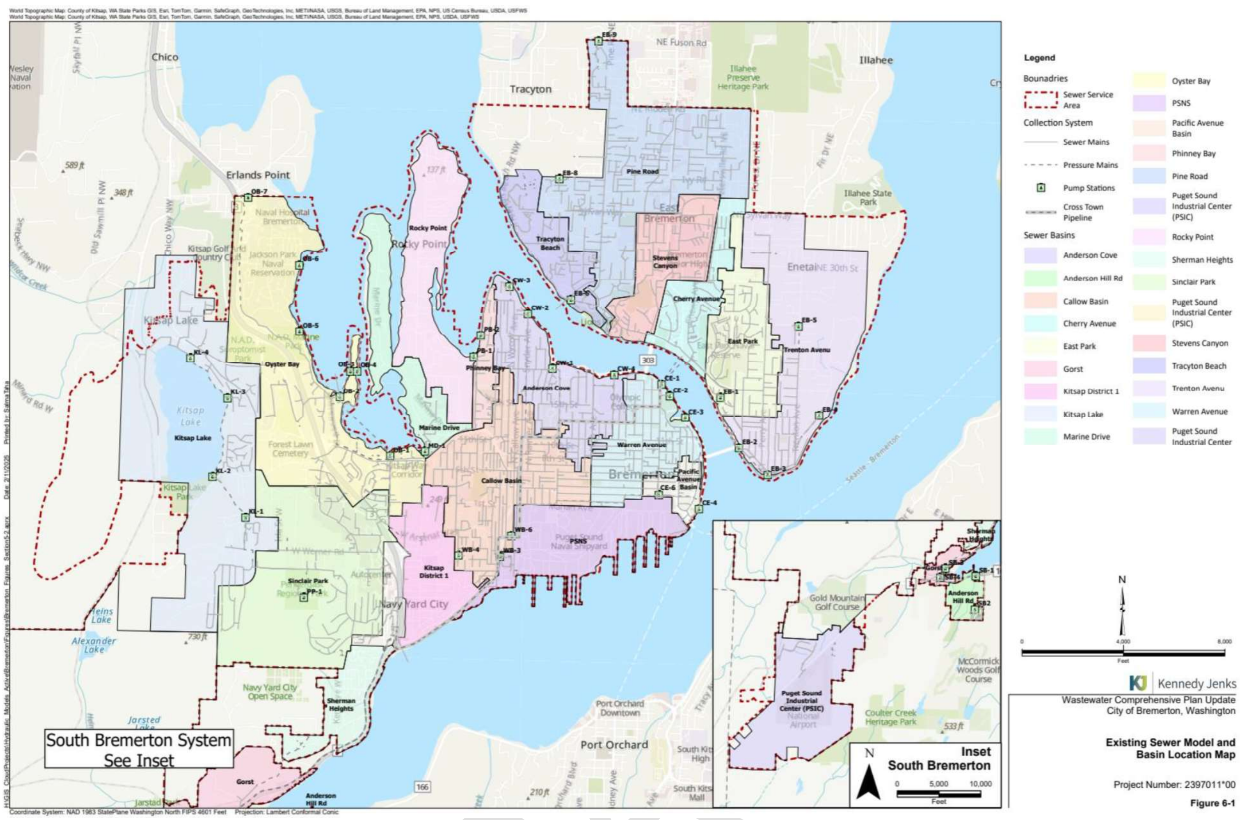

Figure 6-1: Existing Sewer Model and Basin Location Map

6.2.2 Sewer Inflow Allocation

This section explains the methods used to allocate both the average existing and future sewer flows to the hydraulic model.

6.2.2.1 Existing Sewer Average Daily Flows Calculation from Water Billing Data

Sewer flows are not directly metered at each individual customer and must be estimated based on the customer's corresponding water demand. The billed water demand was calculated based on one year of complete water billing data from 2023. Table 6-1 shows both the total billed water and sewer demand by usage type. Customers that use water for irrigation have no return flow, as the water is fully consumed and does not return to the sewer collection system. Regarding water demand, 45% of it is comprised of residential customers, 18% multifamily residential, and 13% commercial. Sewer only usage from various classes, such as the U.S. Navy, hospital sewer, and others, comprises 24%.

Sewer annual and winter average flows were calculated for each water customer usage type. The calculation was accomplished by dividing the total flow between meter readings by the number of days between meter readings. Sewer return flow ratios were then developed for each water customer usage type to calculate average sewer demand for each customer. The sewer

return flow ratio represents what portion of the water demand being consumed by a water customer is then returned to the collection system as waste. The estimated billed water demand is 3.16 MGD, with a 96% sewer generation rate, resulting in an average annual sewer flow of 3.04 MGD.

Table 6-1: Annual (2023) Billed Water and Sewer Demand by Usage Type

Land Use	Total Water Use (Hundred Cubic Feet)	Total Water Use (gpm)	Total Water Use (MGD)	Sewer Return Flow Ratios (%)	Average Sewer Demand (MGD)
COMM I	109,000	155	0.22	95	0.21
COMM II	58,012	83	0.12	95	0.11
COMM III	31,030	44	0.06	95	0.06
RESIDENTIAL	691,583	984	1.42	95	1.35
MULTI-FAMILY	273,668	389	0.56	95	0.53
VARIOUS CLASS	378,365	538	0.78	100	0.78
TOTAL	**1,544,835**	**2,198**	**3.16**		**3.04**

As described in Section 5.3.2, calculated sewer generation with each historical ADWF from 2020 through 2023 shows an ADWF of 3.60 MGD. The calculated average sewer demand in Table 6-1 shows a value of 3.04, therefore an 18% global adjustment was applied to align with the WWTP influent data, which is considered the more consistent data source, spanning several years.

6.2.2.2 Existing Sewer Inflow Allocation

Sewer inflow allocation consists of assigning sewer flow to the appropriate manholes (nodes) in the model. The goal is to distribute the flow throughout the model to best represent actual system response.

The base flow component is comprised of sanitary flow (i.e., residential, commercial, and industrial sewage) and groundwater infiltration during dry periods. This component was developed based on the methodology described in the previous section, using 2023 water billing records. The water billing data was geolocated within GIS based on parcel number and address information. Using GIS, each customer account was geocoded and spatially joined within the existing sewer collection system. AquaTwin Sewer inflow allocation tool was then used to load the hydraulic model with the calculated sewer demands, using the "nearest pipe" allocation method**.** The sewer demand was assigned to the next upstream manhole. This process was the first step, prior to the manual adjustments made as detailed below:

- **Location 1:** East Bremerton flow was assigned incorrectly to locations like CW-4 and CE-4. After manual adjustments, all flow from East Bremerton was redirected to the manhole upstream of Pump Station CE-1, as it is conveyed to CE-1 via the inverted siphons and then pumped into the WWTP through the Crosstown Pipeline.

- **Location 2:** For South Bremerton, a portion of the inflow was redirected to the manhole upstream of Pump Station SB-3, as this portion is discharged into the Southwest Bremerton Sewer Force Main and directly conveyed to the WWTP. The remaining portion was assigned to South of Sinclair Inlet.

6.2.2.3 Future Sewer Demand Allocation

A buildout model was developed to assess the future infrastructure needs of the City's collection system as development progresses. In this context, "buildout" refers to the scenario in which all areas designated for service by the collection system are fully developed according to the densities specified in the current land use plan.

To project future demands on the system, the model accounts for expected increases in water usage and sewer flows driven by new developments. These projections are based on the 2044 Traffic Analysis Zone projections provided by Kitsap County, as discussed in Section 5.4.2.

This growth, however, is not evenly distributed across the City. The basins in the East Bremerton and South Bremerton areas are expected to see the largest increases, each contributing approximately 28% of the total projected growth in demand. These projections provide a foundation for determining the appropriate sizing of the collection system infrastructure to meet both existing and future flows.

The future sewer demand of 1.4 MGD associated with these areas was allocated in the hydraulic model using the allocation method described earlier. The specific locations of these demands are illustrated in Figure 6-2, which shows how the projected growth is distributed across the various basins. This allocation ensures that the infrastructure is appropriately sized to handle the anticipated growth and maintain system efficiency as development continues.

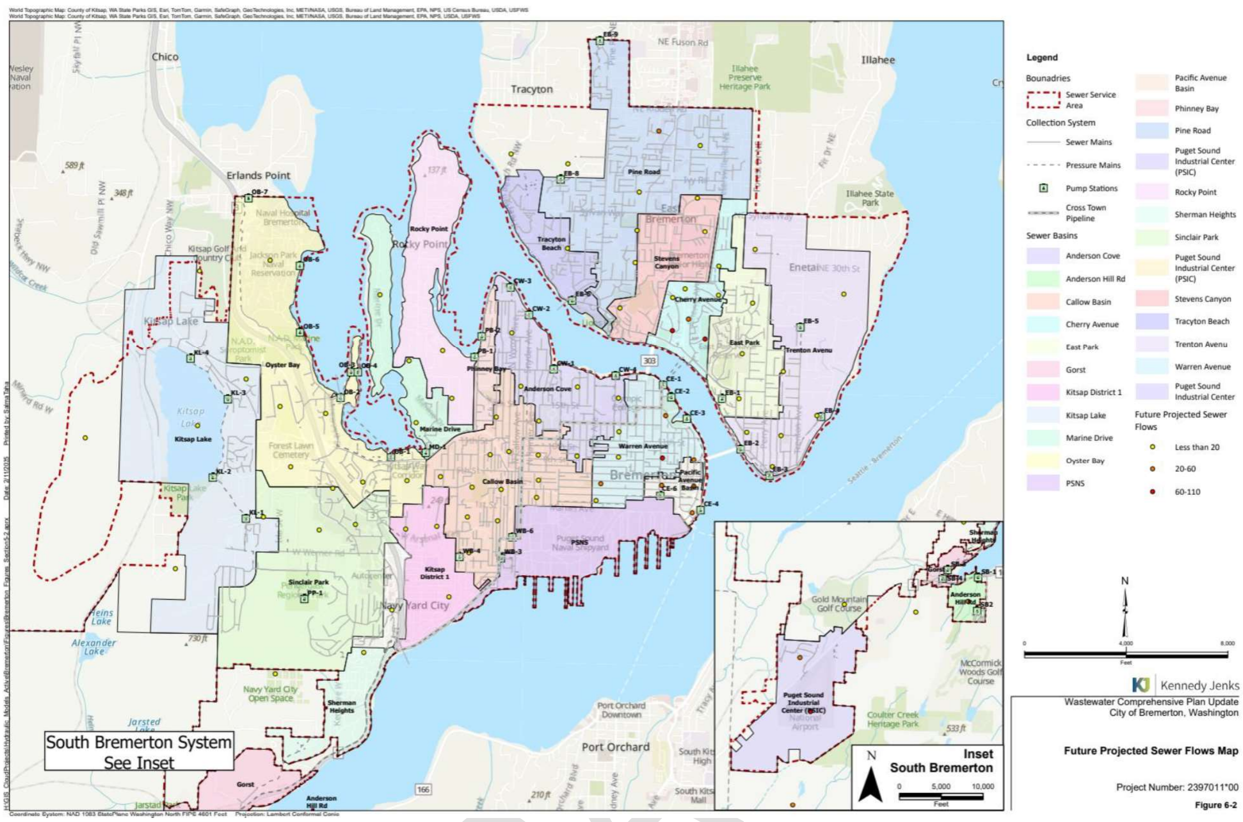

Figure 6-2: Future Projected Sewer Flow Map

6.2.3 Flow Meter Data Collection

Due to lack of good flow metering data throughout the City's collection system, the City provided total volume data for the one-year period from January 1, 2023 to January 1, 2024 which was based on the permanent flow meters located at several City pump stations. A schematic showing the observed ADWF at each of the pump stations during the dry season months (June through September) resulting in a total flow of 3.15 MGD is shown in Figure 6-3.

The Kitsap Lake Basin consists of several pump stations, with KL-1 being the primary station in the area. All other pumps within this basin are tributary to KL-1. The KL-2 pump station, which is a large facility, discharges into KL-1. KL-3 discharges to KL-2, and KL-4 discharges to KL-3.

The Anderson Cove Basin includes CW-4. Discharge from CW-4 can be redirected depending on the season. During wet weather months, CW-4 pumps into the Crosstown Pipeline in order to minimize overflows at OF-11. During the dry weather months, it pumps into the beach main at the end of High Avenue, which discharges to CW-1 to ensure that flushing velocities are maintained in the beach main during periods of low flow. CW-1 discharges into the Crosstown Pipeline at 13th and Naval Avenue. Lift station CW-2 discharge can also be redirected seasonally. CW-2 normally pumps to the Callow Basin, making it tributary to WB-3. During dry weather, flow from CW-2 can be directed (via valving at 19th Street and Snyder Avenue) to CW-

1 to minimize the wet well retention time during dry weather low flows to reduce odors. Recently WW staff decided to direct the CW-2 flow to the Callow Basin at all times, though the option still exists to redirect to CW-1 as needed.

The Pacific Avenue Basin is served by two main pump stations. CE-4 receives all the wastewater flow from the downtown waterfront area and the east end of the shipyard, and it pumps this flow into the Central Bremerton Force Main. CE-6, which receives flow from streets west of Pacific Avenue, east of Warren Avenue, and bounded on the north by MLK Way, pumps into a force main along Park Avenue, enters the gravity sewer near MLK Way, then flows to CE-1.

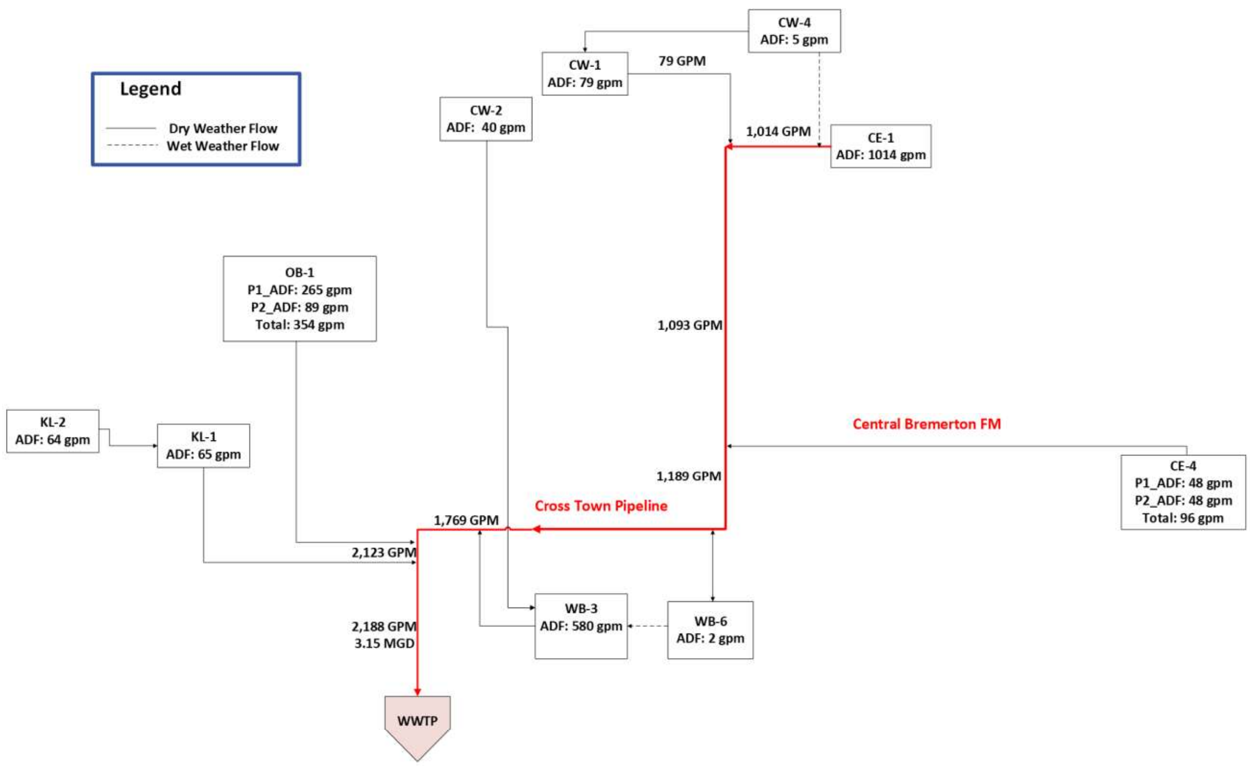

Figure 6-3: ADWF Schematic from PS Flow Meters

6.2.4 Model Calibration Methodology

In this calibration effort, the discrepancy of total ADWF from flow meters, water billing data and historical influent data from WWTP was considered. Due to limited availability of data sets that occurred during a similar interval of time, an average daily wastewater flow generated from water billing was imported into the model as a basis of our ADWF calibration.

Following the data import, the 2023 water billing were adjusted to align with the WWTP influent data set, which, as discussed earlier, covers the period from 2018 to 2023. Given that 2023 was a dry year with minimal rainfall and that the ADWF at the WWTP remained stable from 2019 to 2022, a global adjustment of 18% was applied to the calculated sewer generation based off the

2023 water billing data. This adjustment was made to adopt a conservative approach and ensure consistency with the broader dataset.

The calibration of the WWF involved reviewing rainfall data and flow data from the collection system to establish the relationship between rainfall and system flow, both across the entire system and at the WWTP. The storm chosen for calibration occurred on November 4, 2023, which saw the highest peak rainfall of 2.3 inches. Additionally, according to the previous annual CSO report, two CSO events took place in November 2023, coinciding with this storm event.

A peaking factor was applied to approximate peak WWF conditions, allowing a preliminary assessment of system performance under peak conditions. While this approach provided insights into peak flow locations, it is important to acknowledge the steady-state simulation may not fully capture the dynamic nature of the collection system. Future data collection efforts should aim to include ongoing flow monitoring meters at various points of the collection system to generate detailed flow patterns to enhance the calibration process and improve model accuracy.

6.2.5 Rainfall Data

The City provided rainfall data for two City rain gauges which were monitored at fifteen-minute intervals from 3/2/2023 to 12/31/2023. During this period, 38 rainfall events were recorded. The meter data was reviewed to identify suitable storms for calibration. The storm selected for calibration occurred on November 4, 2023, which had the highest peak rainfall of 2.3 inches. Although the storm on December 4, 2023, lasted longer, its peak rainfall was only 0.47 inches. Table 6-2 presents the rainfall summary statistics (event, date, time, duration, and peak rainfall) for events with rainfall of at least 0.25 inches.

Table 6-2: Rainfall Event Summary

Event	Date	Duration (h)	Maximum Rainfall (in)
1	3/11/2023 12:00	7.25	1.003
2	3/12/2023 20:45	16.25	0.28
3	3/31/2023 18:30	7.75	0.262
4	4/2/2023 16:00	7.25	1.329
5	4/3/2023 13:15	7.25	0.352
6	4/6/2023 13:45	7.25	0.251
7	4/10/2023 17:30	7.25	0.2795
8	4/16/2023 21:30	7.25	0.271
9	4/17/2023 18:30	7.25	0.2515
10	4/19/2023 13:45	7.25	0.6195
11	5/3/2023 20:15	7.5	0.6375
12	6/18/2023 12:30	11	1.083
13	6/19/2023 16:45	7.5	0.49
14	9/19/2023 23:00	8.75	0.321
15	9/24/2023 19:45	12.25	0.2865
16	9/25/2023 16:00	7.75	2.157
17	9/26/2023 11:00	8.25	0.8885
18	9/27/2023 4:45	15.5	0.43
19	9/28/2023 11:30	7.25	0.3205
20	10/10/2023 7:45	7.25	0.27
21	10/11/2023 1:45	7.25	0.2875
22	10/16/2023 7:00	7.5	0.409
23	10/24/2023 18:45	23	0.332
24	11/1/2023 21:30	17	0.423
25	11/3/2023 20:00	20.25	2.318
26	11/5/2023 5:30	20.75	0.8275
27	11/6/2023 14:00	13.5	0.34
28	11/9/2023 14:15	7.25	0.282
29	11/11/2023 4:30	7.25	0.2815
30	11/12/2023 13:15	8.5	0.3175
31	12/1/2023 3:15	9.25	0.3105
32	12/1/2023 21:30	11	0.459
33	12/4/2023 6:30	30	0.4695
34	12/5/2023 19:00	7.25	0.2605
35	12/7/2023 0:15	13.5	0.6005
36	12/22/2023 8:15	7.75	0.288
37	12/25/2023 10:00	7.25	0.36
38	12/27/2023 21:15	8	0.4705

6.2.6 Calibration Statistics

When evaluating calibration results, it is crucial to define the acceptable tolerance between actual and measured data. For this calibration, best practice model calibration criteria, as obtained from the Wastewater Planning Users Group's (WaPUG) Code of Practice for Hydraulic Modelling of Sewers were adopted.

Additionally, the Chartered Institution of Water and Environmental Management (CIWEM) Code of Practice for the Hydraulic Modeling of Urban Drainage Systems (2017) provides the same criteria for assessing simulated peak flows during both dry- and wet-weather events. The following calibration statistics criteria listed in Table 6-3 were used to guide the calibration process.

Table 6-3: Calibration Statistics Criteria

Criteria	Recommended Range
Dry Weather Peak Flow	-10% to +20%
Wet Weather Peak Flow	-10% to +25%

6.2.7 Dry Weather Flow Calibration

The initial step in determining the Dry Weather Flow (DWF) component of the flow data involved identifying dry and wet days. By analyzing the rainfall data graph, the DWF period was selected during the dry season months (June to September). No flow data was available from pump stations located at Sinclair Park, Kitsap Sewer District No. 1 or South Bremerton.

Peak flow from the Average Dry Weather Flow (ADWF) scenario was recorded at the pipe upstream of each of the pump stations. The dry weather flows predicted by the model were compared with the afield-measured flow data from the 9 permanent flow meters. Table 6-4 presents the results of the dry weather flow calibration.

Table 6-4: Existing DWF Calibration Results

Pump Names	Basin Name	ADWF FROM PS DATA (gpm)	Conduit Model ID	Peak Flow Error	ADWF_Model Results (gpm)
CE-1	Warren Avenue	1,014	2610	13%	887
CE-4-P1	Pacific Avenue	48	3636	-10%	105
CE-4-P2		48			
CW-1& CW-4	Anderson Cove	79	4309	-72%	136
CW-2(1)		40	711	26%	30
KL-1	Kitsap Lake	65	1411	8%	60
OB-P1	Oyster Bay	265	1951	-1%	356
OB-P2		89			
OB-P3		Assumed it doesn't need to operate, only P1 & P2			
WB-3	Callow Basin	580	C152	-17%	680
No pump info received	Sinclair Park & Kitsap Sewer District No. 1 & South Bremerton	None	3892	N/A	244
Total (gpm)		**2,228**			**2,498**
Total (MGD)		**3.15**			**3.60**

Notes:
(1) CW-2 normally pumps to the Callow Basin, making it tributary to WB-3

Overall, the model calibration results for peak flow during dry weather conditions aligned with the criteria outlined in Section 6.2.6, with the exception of the Callow Basin (WB-3) and Anderson Cove (CW-1), which receives flow from CW-4 under dry weather flow conditions.

6.2.8 Wet Weather Flow Calibration

As previously mentioned, the calibration of Wet Weather Flow (WWF) involved reviewing rainfall data and peak hourly flow measurements gathered at pump stations during the November 4, 2023 storm event. The flow data from this event was utilized to estimate peaking factors for the different basins within the collection system. These factors were adjusted to represent the impact of a storm event that would yield a total peak flow rate of approximately 47 MGD, corresponding to the total peak hourly flow projection for 2024, as described in Section 5.4.3.

No pump station data was available for the Sinclair Park, Kitsap Sewer District No. 1, and South Bremerton basins, a peaking factor of 8.5 was assumed for these areas, based on the peaking factors calculated for the other basins.
Table 6-5 summarizes the results of the wet weather flow calibration and the corresponding peaking factors applied in the collection system model.

Table 6-5: Existing WWF Calibration Results

Pump Names	Basin Name	PHF from PS DATA (gpm)	Total PHF from PS DATA (gpm)	Calculated PF	PHF_Model Results (gpm)	Percentage Error
CE-1	Warren Avenue	6,774	6,774	7.3	6,748	0.39%
CE-4-P1	Pacific Avenue	729	1,458	13.9	1,462	-0.27%
CE-4-P2		729				
CW-4	Anderson Cove	254	5,398	30.6	254	-0.17%
CW-1		3,921			3,516	10.32%
CW-2		1,223		41.1	1,224	-0.11%
KL-1	Kitsap Lake	749	749	12.5	748	0.15%
OB-P1	Oyster Bay	1,325	2,188	6.1	2,172	0.73%
OB-P2		863				
OB-P3		Assumed it doesn't need to operate, only P1 & P2				
WB-6	Callow Basin	5,635	12,968	19.1	13,069	-0.78%
WB-3		7,333				
No pump info received	Sinclair Park & Kitsap Sewer District No. 1 & South Bremerton	None		8.5[1]	2,693	N/A
Total (gpm)		**29,535**	**29,535**		**31,887**	
Total (MGD)		**42.53**	**42.53**		**45.92**	

Notes:
[1] Assumed peaking factor in relation to other basins

The calibration results for peak flow during wet weather conditions were consistent with the criteria outlined in Section 6.2.6. However, due to the limited availability of data sets from similar time periods and the peaking factor derived from the 2024 projected peak hourly flow, future monitoring efforts should focus on collecting ongoing flow data at various points within the system. This will allow for a more detailed understanding of flow patterns, which will improve the calibration process and enhance the overall accuracy of the model.

6.2.9 Future Flow Results

Two scenarios, based on the TAZ 2044 projections developed by Kitsap County, were assessed for future growth. To represent the projected flow, a growth projection of 1.4 MGD was applied, reflecting anticipated localized growth within each basin, which accounted for future dry weather flow conditions.

A proportional inflow and infiltration (I/I) rate was assumed, consistent with the existing peak wet weather flow, to model the impact of a storm event resulting in a total peak flow rate of approximately 57 MGD. This aligns with the peak hourly flow projections for 2044, as outlined in Section 5.4.3. Consequently, the same peaking factors derived from the existing wet weather flow (WWF) were applied to the future WWF scenario. Table 6-6 summarizes the results for the peak wet weather flow projections.

Table 6-6: Future Peak Hourly Flow Results

Pump Names	Basin Name	Added 2044 Future Demands (gpm)	PHF_Future Model Results (gpm)
CE-1	Warren Avenue	507	9,098
CE-4-P1	Pacific Avenue	106	2,932
CE-4-P2			
CW-4	Anderson Cove	4	254
CW-1			3,889
CW-2			1,251
KL-1	Kitsap Lake	53	1,405
OB-P1	Oyster Bay	24	2,321
OB-P2			
OB-P3		Assumed it doesn't need to operate, only P1 & P2	
WB3 & WB6	Callow Basin	16	13,358
No pump info received	Sinclair Park & Kitsap Sewer District No. 1 & South Bremerton	270	4,844
Total (gpm)		**981**	**39,350**
Total (MGD)		**1.4**	**56.7**

6.2.10 Hydraulic Model Capacity Evaluation

This section describes the capacity analyses performed using the calibrated sewer system model as well as the hydraulic model findings.

6.2.10.1 Existing Flow Conditions

6.2.10.1.1 d/D Capacity Analysis

The capacity requirements for the wastewater collection system vary between the combined and separated sewer basins. In the combined basins, combined sewer overflows (CSOs) are allowed no more than once per year per outfall. In the separated basins, sanitary sewer overflows (SSOs) are not permitted. Existing facilities must meet these capacity requirements.

One of the key performance indicators (KPIs) of a sewer collection system is limiting the occurrence of sanitary sewer overflows (SSOs). This objective hinges on the ability of each pipe within the collection system to convey accumulated upstream sewer flows without exceeding full pipe capacity. When a gravity sewer pipe transitions from gravity to pressurized flow, known as surcharging, the risk of an SSO drastically increases.

Flow depth has long been a key design parameter in sewer design, exemplified by the flow depth-to-diameter (d/D) ratio. This ratio is calculated as d/D, where 'd' represents the flow depth in inches, and 'D' is the sewer diameter in inches. Gravity sewers are typically designed with some reserve capacity, the degree of which can vary based on various design recommendations. For instance, the American Society of Civil Engineers (ASCE) and the Water Environment Federation (WEF) recommend that sewers up to 15 inches in diameter maintain dry weather d/D ratios below 50%, and larger sewers keep these ratios under 75%. Sewers are

not intended to operate under surcharge conditions, hence wet weather d/D ratios should ideally remain below 100%.

In the scenario analysis, pipes were categorized based on their flow depth-to-diameter ratio into five groups: less than 20%, 20%-40%, 40%-60%, 60%-80%, and greater than 80% full. Pipes with a d/D ratio greater than 80% were flagged for capacity improvements and marked in red on the model output. Figure 6-4 shows the results of the existing flow DWF d/D capacity analysis. Figure 6-5 shows the results of the existing flow WWF d/D capacity analysis.

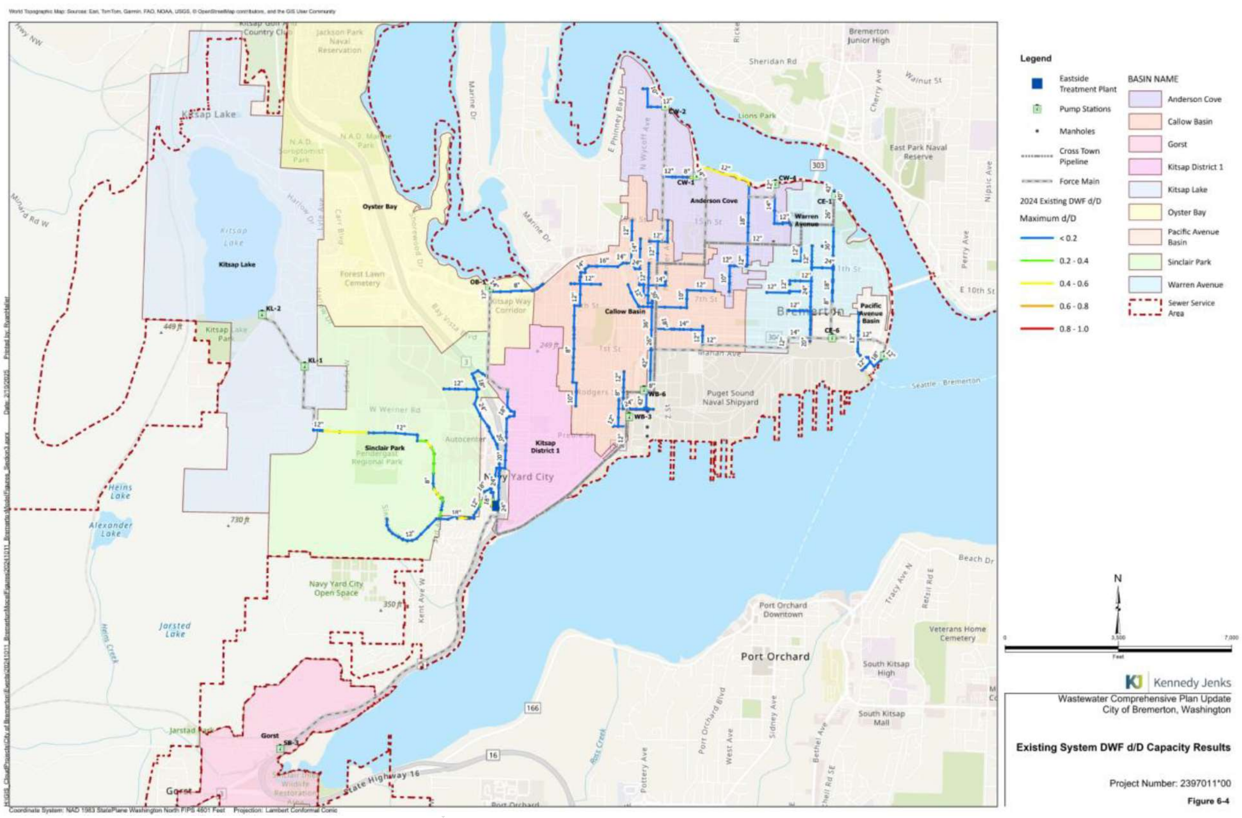

Figure 6-4: Existing System DWF d/D Capacity Results

The output from the existing WWF scenarios indicates localized capacity limitations in several segments of the collection system, including the Sinclair Park, Callow Basin, Anderson Cove and Kitsap Sewer District No. 1 Basins. These deficiencies are primarily observed in 8-inch, 10-inch and 12-inch pipes, which can be addressed by upsizing the sewer lines to 10-inch, 12-inch and 14-inch, respectively. Additionally, in the Kitsap Sewer District No. 1 Basin, the 24-inch main along Bayview Drive to the Westside WWTP also shows capacity deficiencies.

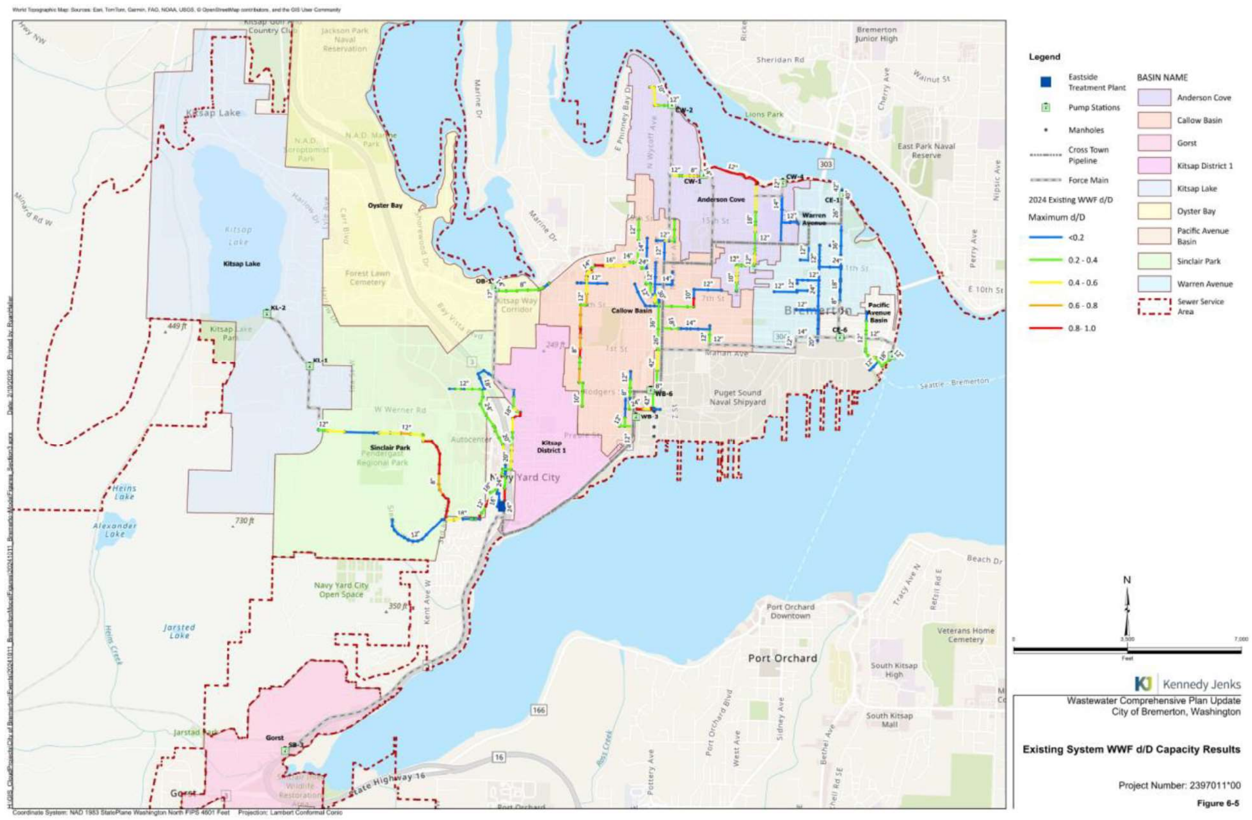

Figure 6-5: Existing System WWF d/D Capacity Results

6.2.10.1.2 Pipe Velocity Analysis

The other main KPI for collection systems is the range of sewer flow velocities. Pipe velocity is driven by pipe diameter, pipe slope, and sewer flow rates. Pipe velocities should be high enough to prevent debris and solids from settling and collecting at the bottom of the pipe, but low enough to avoid significant flow turbulence/hydraulic jumps, which releases more corrosive gasses that could lead to manhole corrosion and/or odor complaint issues. The calibrated hydraulic model was used to evaluate system pipe velocities to identify any potential areas not achieving the City's design velocity criteria. According to the Department of Ecology's Criteria for Sewage Works Design, sewer mains must uphold a minimum velocity of 2 feet per second (fps) and a maximum of 15 fps, adhering to a minimum pipe slope of 0.4%.

The primary goal under ADWF conditions is to maintain a minimum velocity to achieve effective scouring. Scour velocity is crucial in addressing issues related to impacts of Fats, Oils, and Greases (FOGs) as well as sanitary products, such as baby wipes. The typical industry standard for minimum sewer velocity is 2 fps. Even using this less conservative minimum velocity criterion, the hydraulic evaluations reveal that under ADWF, 66% of collectors and 71% of trunk sewers did not meet the 2 fps criterion. Consequently, these areas with low velocities may require more maintenance to maintain system efficiency and reliability. This goal will

improve both the effectiveness of wastewater management and the longevity of the sewer infrastructure. Figure 6-6 shows the results of the analysis.

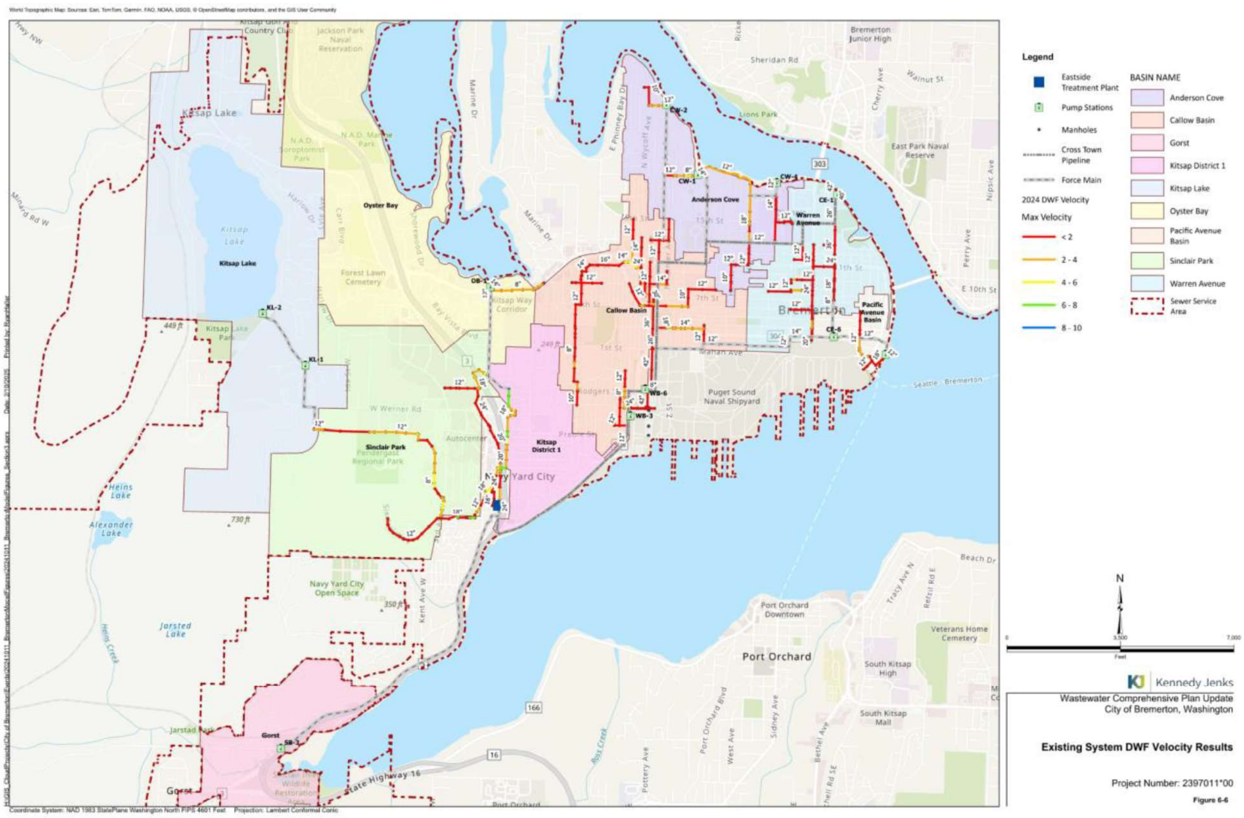

Figure 6-6: Existing System Maximum Velocity Results

6.2.10.2 Future Flow Conditions

6.2.10.2.1 d/D Capacity Analysis

An identical d/D capacity analysis was performed for the future sewer flow conditions. Figure 6-7 illustrates the results of the future flow DWF d/D capacity analysis. Figure 6-8 illustrates the results of the future flow PWWF d/D capacity analysis.

The growth projection model discussed in Section 6.2.9. indicates that a significant proportion of the projected growth is anticipated to occur within Sinclair Park & South Bremerton. The existing WWF scenario indicates that infrastructure within the Sinclair Park, Callow Basin, Anderson Cove, and Kitsap Sewer District No. 1 are already at capacity. When considering the future growth, these basins, along with the Pacific Avenue Basin, are projected to experience capacity limitations. These deficiencies are most easily addressed by upsizing sewer lines and correcting notable I/I concerns.

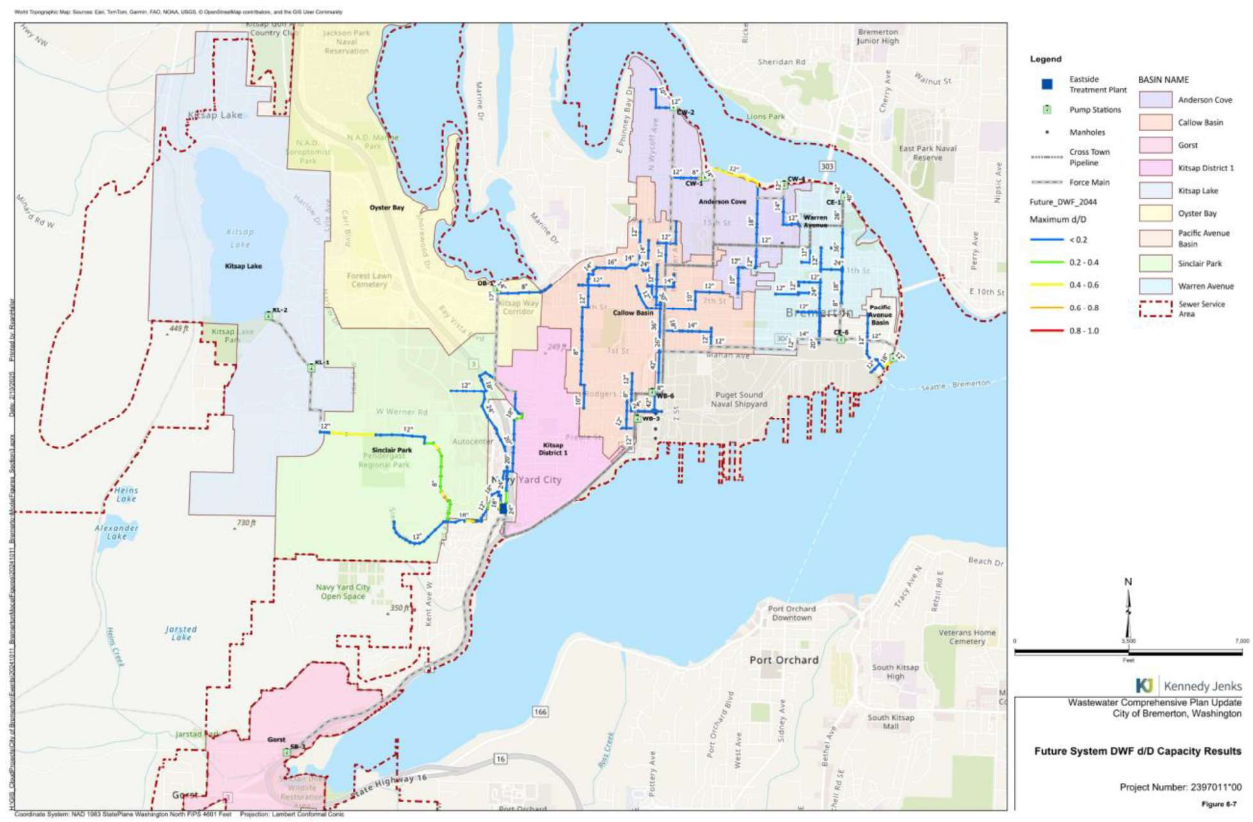

Figure 6-7: Future Flow DWF d/D Capacity Results

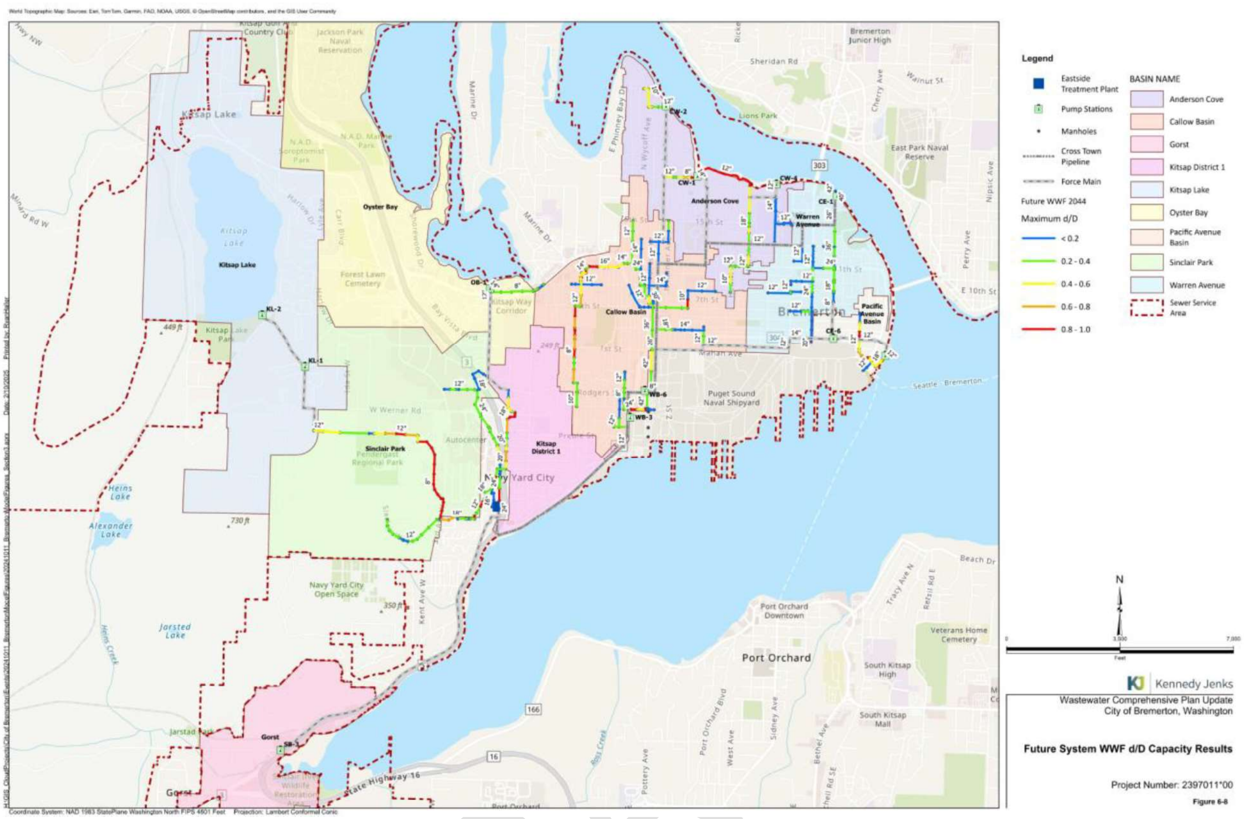

Figure 6-8: Future Flow WWF d/D Capacity Results

6.2.10.2.2 Pipe Velocity Analysis

An identical pipe velocity analysis was performed for the future sewer flow conditions. Figure 6-9 illustrates the results of the future pipe velocity analysis.

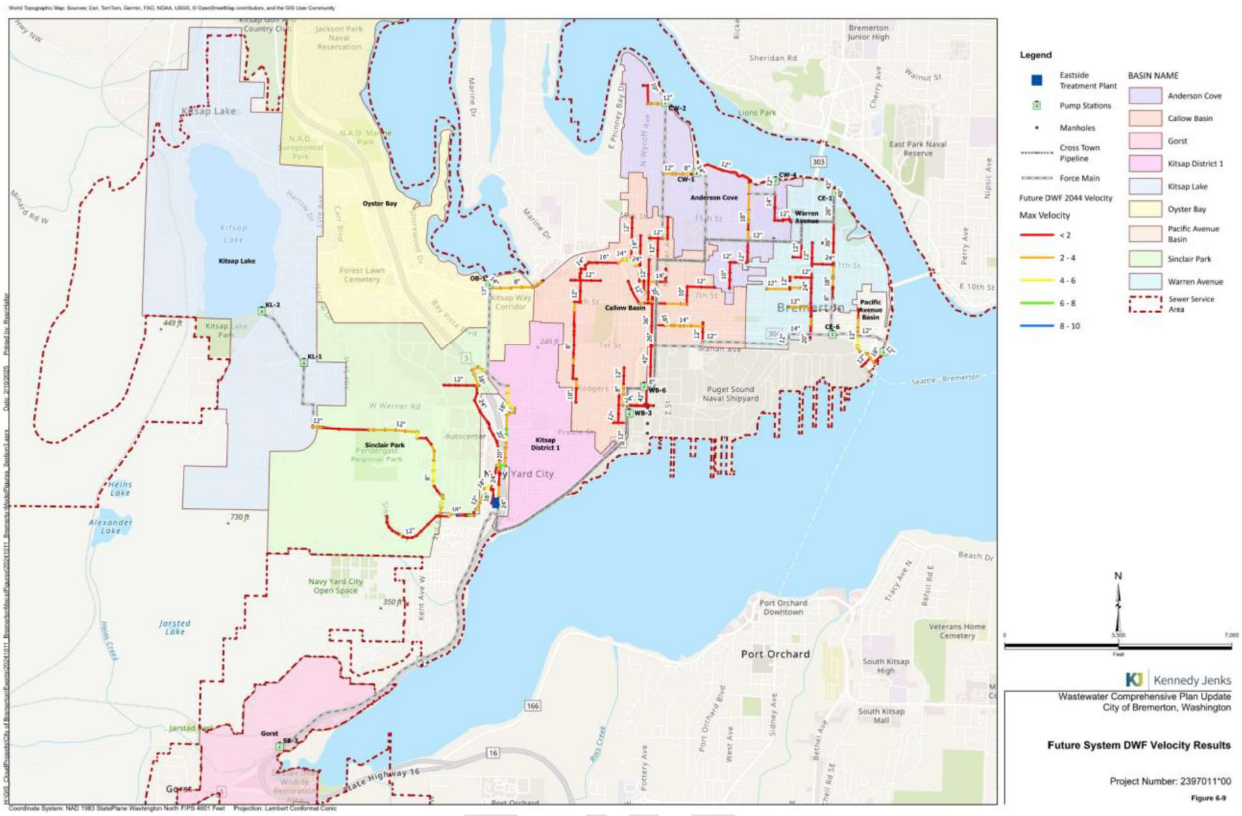

Figure 6-9: Future Flow DWF Pipe Velocity

6.2.11 Inflow & Infiltration Analysis

I&I refers to the entry of groundwater (infiltration) and stormwater (inflow) into a sewer system. When a collection system is compelled to handle a greater flow than which it was designed, I&I can result in pipeline surcharging and downstream impacts to pumping and/or wastewater treatment facilities. This increased effluent not only poses challenges for wastewater treatment facilities but also escalates costs since the stormwater and groundwater mix with sewage and the entire volume must be processed at the treatment plant.

I&I can constitute a significant portion of annual flow to treatment plants; this excess burden can overwhelm the collection system and lead to several adverse consequences.

- Uncontrolled SSOs: I&I can exceed the capacity of pipelines, pumping stations, or treatment plants, causing raw sewage to overflow and pollute the environment.
- Pipe Failures: Excess flow due to I&I, can stress and damage pipes, increasing the risk of leaks and breaks.
- Soil Subsidence and Sinkholes: I&I can undermine the supporting soil around pipes, potentially leading to ground collapse and sinkholes.

These issues highlight the importance of addressing I&I effectively to safeguard public health, environmental quality, and infrastructure integrity.

Calculating I&I involves flow monitoring, where wastewater flow in the system is measured over time and compared with a baseline flow estimate. These measurements are crucial for determining the extent of I&I in the sewer system. Flow monitoring not only aids in identifying and prioritizing areas for further inspection but also serves as a basis for visual inspection methods to assess the severity of I&I in a given area.

6.2.11.1 Graphical Identification of I&I

Inflow is usually recognized graphically by large-magnitude, short-duration spikes immediately following a rain event. Infiltration is often recognized graphically by a gradual increase in flow after a wet-weather event. The increased flow typically sustains for a period after rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels.

Figure 6-10 illustrates rainfall and wastewater flow data for CE-1 collected between March and December 2023. The storm event used for I&I analysis was November 4, 2023 storm as the calibration period. The wet-weather flow hydrograph represents the total inflow volume, while it is overlaid with the dry-weather flow hydrograph, which includes sanitary flow and infiltration from the days preceding the storm. The difference between these two-time series represents the volume of I&I entering the collection system. Figure 6-11 provides a visualization from a portion of flow meter data for CE-1. The total I&I volume serves as an indicator of the combined inflow and infiltration, which was calculated for all basins using available flow meter data to identify the basins with the highest levels of I&I.

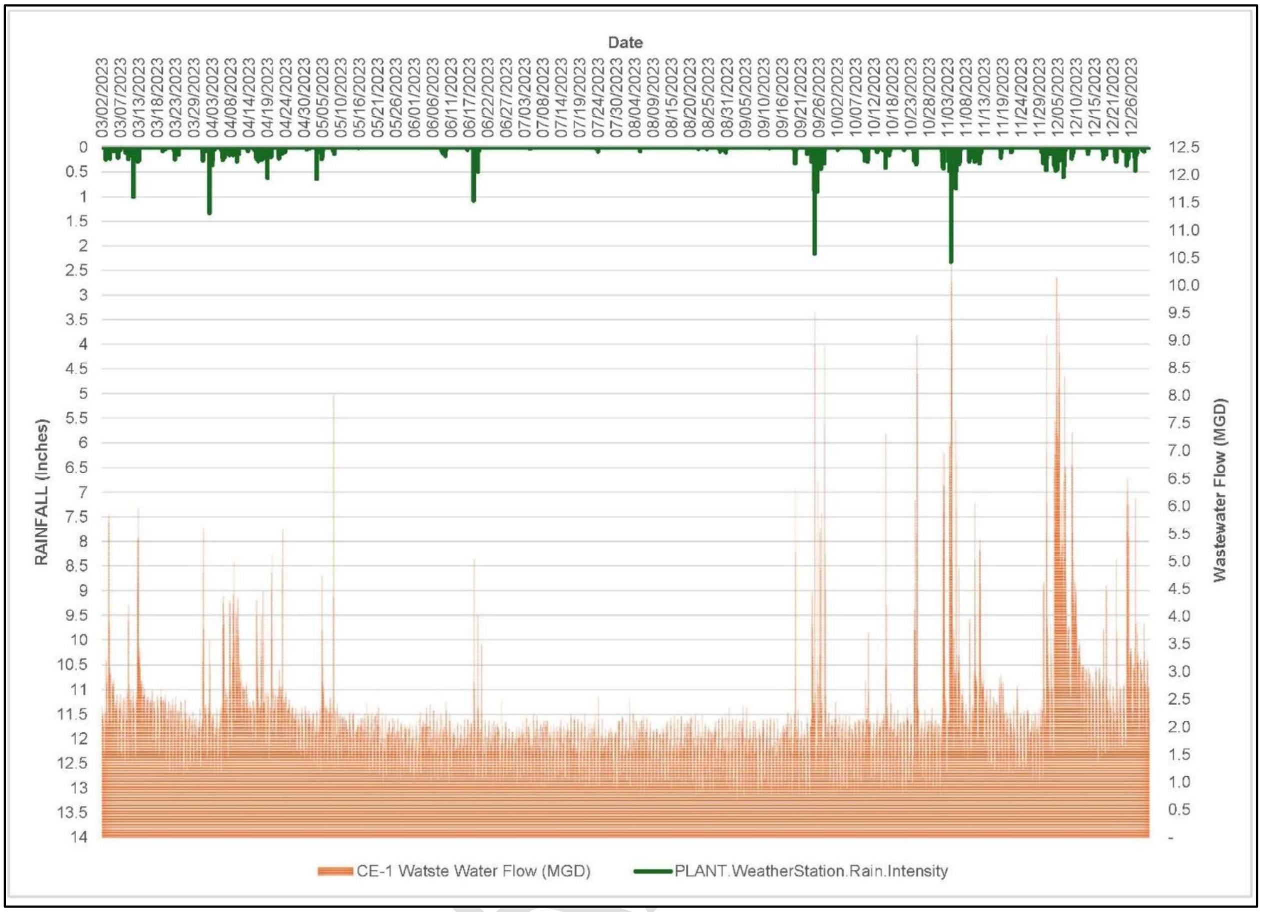

Figure 6-10: CE-1 Wastewater & Rainfall Hyetograph

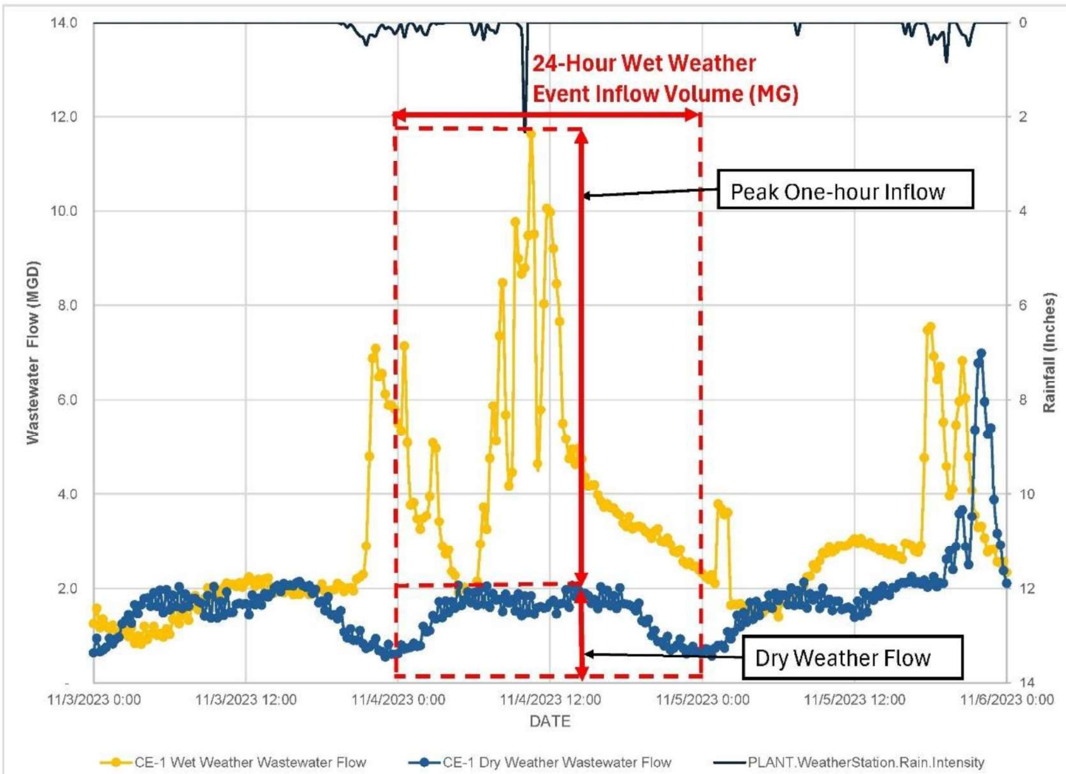

Figure 6-11: Overlay of Wet and Dry Weather Events for CE-1

6.2.11.2 I&I Results Analysis

To compare I&I levels across different basins, several methods can be used to normalize the analysis metrics. One common approach is to report as gallons per day per inch-diameter mile (gpd/idm). This unit accounts for both pipe diameter and length, allowing for meaningful comparisons even when basins have different layouts.

Using gpd/idm, basins can be ranked based on their highest I&I rates. This ranking helps prioritize areas with the most severe I&I problems. Table 6-7 summarizes the calculations and rankings for the basins based on highest I&I rate using this unit.

Table 6-7: I&I Analysis – November 2023, 24-Hour Storm

Ranking[1]	Basins	Average Pipe Diameter (Inch)	Total Length (Feet)	Inch-Diameter Mile (idm)	Design Storm Total Inflow Volume (Gallons)	Design Storm Total Inflow Rate (Gpd/idm)	% Total I&I[2]
1	Callow Basin	9.9	114,280	214	2,368,704	11,059	34%
2	Anderson Cove	8.8	62,939	105	952,668	9,090	14%
3	Oyster Bay	7.9	46,234	69	483,638	6,990	7%
4	Warren Ave & East Bremerton	9.2	379,401	662	3,035,922	4,588	43%
5	Kitsap Lake	8.0	50,517	77	155,418	2,027	2%
6	Pacific Avenue	10.4	14,374	28	52,012	1,839	1%
7	**Kitsap Sewer District No. 1	10.7	3,713	8	0	0	N/A
8	**Sinclair Park	8.9	56,181	94	0	0	N/A
	TOTAL		**727,639**		**7,048,362**		**100%**

Notes:
(1) Ranking based on Highest I&I Rate (gpd/idm)
(2) Sinclair Park and Kitsap Sewer District No. 1 not included in analysis because of missing flow data.

KJ brought two teams to perform prioritized, as-needed condition and performance assessments at the WWTP and collection system PSs for assets identified as risks during the BRVA. Selection of these assets was based on first the overall risk score, and subsequently assets that received scores of 4 and 5 for either their physical condition or safety condition. Assessments focused on rotating and fixed mechanical equipment and structural visual, non-destructive evaluations. Field investigations included condition assessments at the WWTP and throughout the collection system.

6.2.12 Field Inspections

Eight of the City's PSs and one odor control station were evaluated to identify major equipment needing to be replaced or rehabilitated. The team assessed: CE-4, CW-1, EB-2, EB-3, KL-1, KL-2, OB-1, WB-3, and OCS-1. Notable observations are described below.

- **CE-4:** There is excessive corrosion on piping and the painting/coating systems are in poor condition. The knife gate valves are difficult to close. Consider actuated valves which staff would access without entering confined space. Excessive corrosion was noted in the wet well, most likely due to its proximity to the shoreline. The City should consider ultrasonic testing (UT) on pipe within the wet well. Arc Flash testing should be conducted as soon as possible.
- **CW-1:** The wet well lining is the biggest issue, staff indicated that the wet well was recently recoated, but with there is still visible evidence behind the coating showing heavy spalling and scaling. Unless the poor concrete underneath is addressed, new relining will only continue to serve as a short-term remedy. Additionally, level floats should be added at this station to provide backup control; when the transducer goes out, all three pumps turn on in idle mode and need to be manually turned off. This causes

unnecessary stress on pump motors. Smaller observations include dry rot on pipe gaskets, CV-1's weighted spring mechanism was removed and not replaced, and the motor for the HVAC system on the roof is chirping, indicating minor bearing issues. There are no safety alarms at this site, and Arc Flash testing should be conducted as soon as possible.

- **EB-2:** There is excessive corrosions on piping and the painting/coating systems are in poor condition. The wet well has excessive corrosion, most likely due to its proximity to the shoreline. The wet well concrete shows significant signs of spalling, efflorescence, delamination, scaling and cracking. Vault will fill with water from high tides which is evident from corroding pies and severely damaged seals. Consider UT on all exposed pipe. There are sink holes around the site due to tidal influence, there is a link seal bulge in the high-water check valve vault, and the DLC valve vault will backflow at high water levels, which O&M noted has historically caused the manhole lid to pop. O&M also indicated significant ragging at this site. Arc Flash testing should be conducted as soon as possible.
- **EB-3:** The wet well hatch was bolted down for odor control and could not be assessed as part of this evaluation. There is also an odor control 55-gallon drum buried behind the building that was not able to be assessed. There is a new generator onsite. Overall, the drywell is in good to fair condition, with no excessive signs of corrosion, leaks, vibration, or noise from piping and pumping equipment and all alarms are functional.
- **KL-1:** The concrete wet well has spalling, scaling, delamination, and the coating is in failure The concrete wet well has spalling, scaling, delamination, and the coating is in failure, which O&M indicated to inspectors was original (1970s). Most notably, issues within the wet well are located at the base were 14-inch holes were drilled to convert the original wet well/dry well into a single wet well. This causes unequal water levels due to ragging and plugging of the core-drill holes and the pumps to work harder. Consider further rehabilitation to fully convert this into a more functional wet well that is not inhibited by clogging and intern, excess maintenance. Corrosion was also noted on piping. At the time of inspection, Pump No. 2 had failed and not all electrical systems were functioning so much of the equipment was not able to be observed in operation. Arc Flash testing should be conducted as soon as possible.
- **KL-2:** Overall, this site was in good condition. The drywell, piping and mechanical equipment and structure all look new and in good condition. Wet well observations were limited so consider future inspections when the well is able to be physically entered for a more comprehensive evaluation. O&M staff indicated they would like bigger pumps so as to bypass KL-1 and have this PS flow directly to the force main. Arc Flash testing should be conducted as soon as possible.
- **OB-1:** The concrete wet well has spalling, scaling, and delamination. Staff indicated that the wet well was recently recoated, but with there is still visible evidence behind the coating showing not just heavy spalling and scaling, but large chunks of concrete missing and noticeable structural deformities. Unless the poor concrete underneath is addressed, new relining will only continue to serve as a short term remedy. There was moderate corrosion on joints, and pipes should be recoated and UT should be performed. The corrosion is so extensive in the wet well, an entire section of pipe was broken off and observed in the base of the well. The pumps cycle every few minutes; with this low capacity, a surge tank should be considered to relieve pressure. Staff also

pointed out difficulties in removing the pumps from service for maintenance. Consider alternative methods. Arc Flash testing should be conducted as soon as possible.

- **WB-3:** The concrete in the wet well was in good to fair condition. There is notable corrosion, poor paint/coating, and minor leaks along the piping system and pumps. The connecting pipes are in a configuration inhibiting proper operations. Staff noted capacity issues and there was evidence of leaking. Staff indicated to inspectors they are actively working on a bypass project to avoid these connections. Arc Flash testing should be conducted as soon as possible.
- **OCS-1:** The station walls, floors, doors, electrical switches and conduit, among other equipment, showed significant corrosion; the concrete was spalling and scaling, and major deterioration was noted to the building envelope. Considerable chemical deposits were built up in the containment area. Ventilation louvers where also physically blocked with packing material and taped over. Staff indicated this act is something they do in the winter to keep the room well insulated, however the time of the inspections indicate that this packing was in place through the spring, summer, and now fall. Without the ability to well ventilate the station, and by locking in added heat and humidity that would come during the spring and summer months, corrosion is only exacerbated and increases safety hazards for staff.

6.2.13 Conclusion

In conclusion, the current wastewater facilities generally have sufficient capacity to accommodate existing flows, with the exception of some localized capacity limitations in certain segments of the collection system, including the Sinclair Park, Callow Basin, Anderson Cove, and Kitsap Sewer District No. 1 Basins. The growth projection model predicts significant growth in East Bremerton, Sinclair Park and South Bremerton. South Bremerton has been defined for this WWCP to include the Gorst and PSIC basins. The main trunklines in these areas, along with those in the Callow Basin, Kitsap Sewer District No. 1, Pacific Avenue Basin, and Anderson Cove, are already facing capacity constraints, as indicated by the future WWF scenarios. These deficiencies can be addressed through upsizing sewer lines and mitigating I&I issues.

Based on the highest rankings from the combined I&I analysis, which accounts for the total volume (in gallons) of both inflow and rainfall-dependent infiltration during the November 4, 2023 storm event, the top-ranked basins with the highest normalized combined I/I are Callow Basin and Anderson Cove. It is recommended that future flow metering efforts prioritize these basins within the collection system to better understand and manage excessive I/I. This may lead to the need for improvements to the existing conveyance system to accommodate projected flows. Additionally, flow monitoring should be considered for the Sinclair Park and Kitsap Sewer District No. 1 basins, as they are currently facing capacity limitations. Although these areas were not included in the calibration or I/I analysis due to a lack of data, monitoring will help evaluate the potential impacts of future growth and inform necessary improvements.

Based on results from the field inspections, and among other observations noted, the most critical concerns revolve around pump station wet well degradation, pipe corrosion, and arc flash testing. Long term restoration efforts should be considered to increase the reliability of these assets. Some of the pump stations have limited accessibility, are near bodies of water, and/or would be difficult to have long-term bypassing effort in the event of a failure. Many observed wet wells have severely degraded concrete and placing a liner or simple coating over

this concrete will not address the underlying issues. UT should also be considered at all pump station locations, particularly sites noted above that have severe corrosion and known leaking or broken pipes. Additionally, system wide arch flash testing should be completed to maintain a safe working environment and meet WAC requirements. Proactive rehabilitation efforts to restore these pump stations now before a failure would lower the City’s risk and increase safety. Since these recommendations do not relate to a single station, a programmatic approach to addressing these concerns should be considered.