Section 5

Wastewater Flows and Loads

5.1 Introduction

This section summarizes existing flows and loads at the Water Pollution Control Authority's (WPCA) East Side and West Side wastewater treatment plants (WWTPs) and establishes projected flows and loads for the planning years of 2030, 2040, and 2050. The following items are included in this section:

- Analysis of three years of flow and concentration data from the WPCA, including discussion of trends and peaking factors;
- A breakdown of existing flows by flow type;
- Population and other community growth projections to predict flows and loads to planning year 2050; and
- Recommended flow and loading design criteria for upcoming facility upgrades.

The WPCA is split into two service areas with a sewer service area of approximately 3.4 square miles contributing to the East Side WWTP and approximately 10.2 square miles contributing to the West Side WWTP. There is also an intermunicipal agreement with the Town of Trumbull allowing up to an average of 4.2 million gallons per day (mgd) which connects to the West Side service area. Additionally, there are some direct bill customers in the neighboring towns of Fairfield and Stratford that discharge to the West Side WWTP and East Side WWTP respectively. A map of the existing service area was presented in Section 2.

5.2 Summary of Existing Flows and Loads

Three years of the historic influent flow and load records as reported in the Monthly Operating Reports (MORs) for both plants, from January 1, 2017 through December 31, 2019, were analyzed to establish existing conditions. A summary of the average influent flow and loads from 2017 to 2019 is presented below in **Table 5.2-1** and **Table 5.2-2**.

Table 5.2-1 West Side WWTP Average Influent Flow and Loads 2017-2019

	East Side WWTP
Average daily flow (mgd)	22.1
Biochemical oxygen demand (BOD_5) (lb/day)[1]	28,000
Total suspended solids (TSS) (lb/day)[1]	42,000
Total Kjeldahl nitrogen (TKN) (lb/day)[1]	4,500
Total phosphorus (TP) (lb/day)[1]	780

[1]Values including loadings from septage

Table 5.2-2 East Side WWTP Average Influent Flow and Loads 2017-2019

	East Side WWTP
Average daily flow (mgd)	5.7
Biochemical oxygen demand (BOD_5) (lb/day)	5,700
Total suspended solids (TSS) (lb/day)	6,200
Total Kjeldahl nitrogen (TKN) (lb/day)	1,200
Total phosphorus (TP) (lb/day)	160

A more detailed description of the recent flows and loads for each plant is provided in the following sections.

5.2.1 West Side WWTP Existing Flows and Loads

Wastewater treated at the West Side WWTP is generated from several sources including residential, municipal, commercial, institutional, and industrial properties; septage; and infiltration and inflow (I/I). The service area includes both separated and combined sewers. Although the WPCA has constructed many sewer separation projects, there are still areas with fully combined sewers. It is estimated that approximately 40% of the West Side service area consists of combined sewers.

Figure 5.2-1 illustrates the average flow for the West Side WWTP from the collection system as measured at the WWTP's influent flume, along with the 30-day rolling average flow. The NPDES permit lists an average design flow rate of 30 mgd. During wet weather, if flow exceeds 58 mgd, the excess flow (up to 90 mgd) receives primary treatment, bypasses secondary treatment and is disinfected. All flow discharges through a single outfall into Cedar Creek.

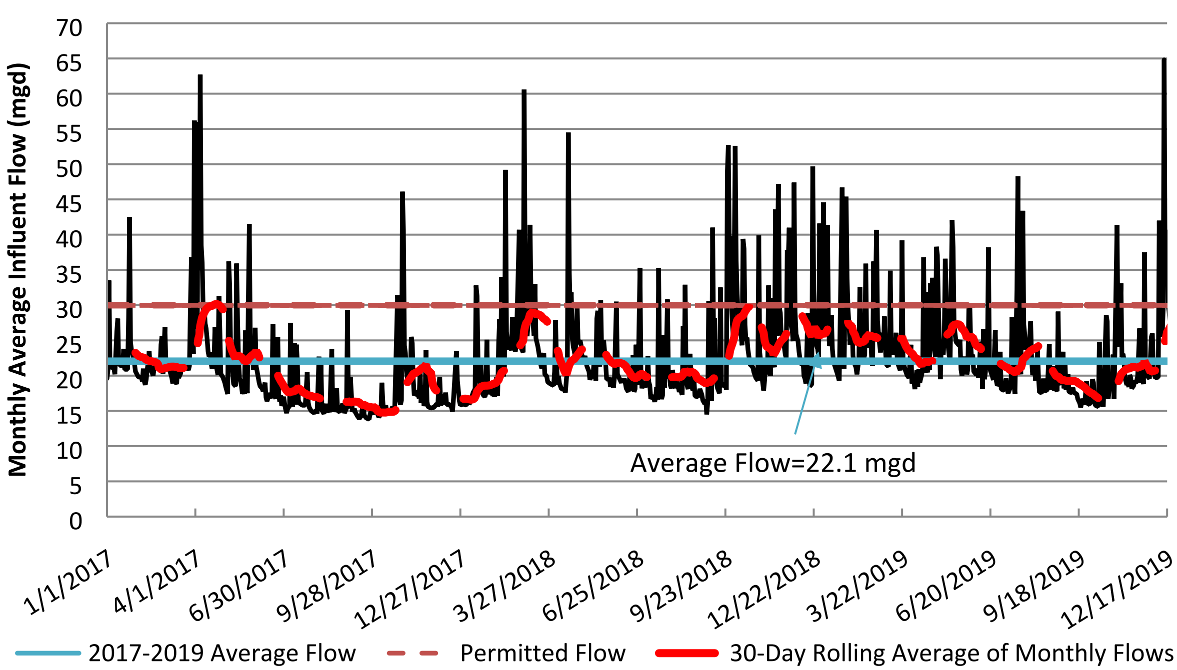

Figure 5.2-1
West Side WWTP Average Influent Flow 2017-2019

The average daily influent flow from January 1, 2017 to December 31, 2019 was 22.1 mgd, which is 74 percent of the permitted capacity of 30 mgd. The 30-day rolling average flow also remained below the permit limit.

5.2.1.1 Breakdown of Flows by Type

This section breaks down the West Side WWTP's influent flows by type. Specifically, flows are divided into the following categories: 1) sanitary 2) septage, and 3) infiltration and inflow.

Estimated sanitary wastewater flows were determined for Bridgeport which included all residential, commercial, institutional, and industrial flows. The sanitary flow component for Bridgeport was calculated by multiplying the average water usage for all customers in Bridgeport by 90 percent, which accounts for water usage that is not wasted to the sewer system. The Water Environment Federation's *Design of Municipal Wastewater Treatment Plants – Manual of Practice (MOP) 8*, 6th Edition (MOP-8) states that 60 to 90 percent of water consumption reaches the sanitary sewer. To be conservative, the 90 percent value was used to calculate the sanitary flow component. This was calculated using the last three years of the city's water billing records. The result is sanitary flow of 8.8 mgd for all of Bridgeport. When split between the two treatment plants, the average sanitary flow component was 6.5 and 2.3 mgd for West Side and East Side, respectively.

All flow from Trumbull's collection system discharges into the West Side collection system via two connection points: a gravity connection called the Sunnydale Crossover and a pumped connection from the Beardsley Pumping Station. Together these two connections contributed an average of 3.4 mgd between January 2017 and December 2019. Because this flow includes I/I, the sanitary component was estimated by analyzing months with lower flows. Additionally, a simple SWMM model was conducted to calibrate flow fluctuations with groundwater infiltration. This model was discussed further in Section 3. The model estimated that the sanitary component from the Trumbull collection system averages 1.7 mgd.

The West Side WWTP accepts septage from domestic and commercial sources. Septage is introduced to the treatment plant at the upstream end of the headworks facility. The plant's influent composite sampler is upstream of the headworks facility so septage loads are not accounted for in these samples. The additional flow due to septage, is included in the plant's influent flow data as the Parshall flume is located just upstream of primary treatment.

The WWTP records daily volumes of accepted septages to the facility. The average daily septage received over the last three years is summarized in **Table 5.2-3**. The daily septage received varies from 3,000 gallons/day to 105,000 gallons/day with an overall average of 42,500 gallons/day. Septage waste flows are a minimal flow component but contribute measurable influent loads due to their concentrations.

Table 5.2-3 Daily Average Septage Quantity by Year

Year	Septage Quantity
2017	44,000 gallons/day
2018	43,800 gallons/day
2019	41,200 gallons/day

Infiltration is groundwater that enters a sewer system through defects in pipes, joints, manhole walls, or improper connections. Inflow is usually rainfall-related water that enters the sewer system through public (on-street) or private (off-street) sources. Public inflow originates from interconnections of the sewer system with the storm drainage system, such as in a combined sewer or dual-invert manhole. Private inflow sources include sewer connections from roof drains, foundation drains, surface drains, and sump pumps. There is also some suspected inflow due to tidal influence, as some of the combined sewer regulator weirs are below high tide and defective tide gates allow water to enter the sewer system during high tides and storm surges.

Infiltration and inflow (I/I) contribute significant flow to be treated at the plant during maximum day and peak hour flow conditions. I/I generally does not contribute significantly to organic and solids loadings.

The service area's average annual inflow was estimated as the difference between the average wet day flow to the treatment facility minus the average dry day flow described below. Daily precipitation data is recorded in the MORs and this information was used to calculate the average dry day flow. Dry and non-dry days are defined in **Table 5.2-4**.

Table 5.2-4 Definition of Dry and Non-Dry Days

Day	Definition
Dry Day	Day on which 0.00 to 0.09 inch of precipitation occurs
Non-Dry Day(s)	Day on which 0.10 to 0.29 inch of precipitation occurs, or Day on which 0.30 to 0.99 inch of precipitation occurs, and the next day, or Day on which 1.00 to 1.99 inches of precipitation occur, and the next two days, or Day on which 2.00 or more inches of precipitation occur, and the next three days

The dry day flow includes sanitary flow plus infiltration and should exclude events that contribute inflow and storm flow, including stormwater from combined sewer areas.

The average daily dry day flow over the three-year period of 2017 through 2019 was determined to be determined to be 20.2 mgd at West Side WWTP, as compared to the overall average daily flow of 22.1 mgd. Therefore, the average annual inflow was estimated to be 1.9 mgd at the West Side WWTP.

To develop an estimate of infiltration entering the system, the sanitary flow component including septage (8.3 mgd) and the inflow flow component (1.9 mgd) were subtracted from the average daily flow to obtain a value of 11.9 mgd.

5.2.1.2 Summary of Flow Components – West Side WWTP

Using the information described above, average daily flows were divided into their various components for the West Side WWTP, as shown in **Table 5.2-5**. Following Table 5.2-5, the methodology for calculation of each value shown is reviewed.

Table 5.2-5 Average Daily West Side WWTP Influent Flow Breakdown by Flow Type

Parameter	Average Annual (mgd, 2017-2019)
Average Day	
Sanitary Wastewater Only	8.3
Infiltration (average)	11.9
Total Dry Weather Flow (no inflow)	20.2
Inflow	1.9
Average Daily Flow	22.1

The following summarizes how the values in Table 5.2-5 were estimated:

- The sanitary wastewater only flow (8.3 mgd) was calculated by multiplying the average water usage within the West Side service area by 90% to account for water usage that is not returned to the sewer system and adding the sanitary component developed from the Trumbull connection meter data. The sanitary-only flow is assumed to be the base wastewater flow attributable to all registered connections to the sanitary sewer system including commercial and industrial users, excluding I/I, and thus is largely independent of seasonal fluctuations.

- The average dry weather flow (20.2 mgd) was calculated as the average plant influent flow on dry days as defined in Table 5.2-3. It is assumed that there is no inflow to the system on dry days.

- The average annual inflow (1.9 mgd) was calculated as the total average daily flow (including wet days) minus the average dry day flow for the full three-year data set (22.1 minus 20.9 mgd).

- Average annual infiltration (11.9 mgd) consists of groundwater and tidal infiltration. The groundwater infiltration was calculated by subtracting the sanitary wastewater flow from the average dry weather flow (20.2 minus 8.3 mgd).

- The total average daily flow (22.1 mgd) is the average influent plant flow measured by the influent flume over the three-year time period (January 2017-December 2019).

Based on this flow analysis, combined I/I is estimated to average 13.8 mgd during the three-year period analyzed, representing approximately 62 percent of the total flow at the facility on an average annual basis.

5.2.1.3 Peak Flows and Flow Peaking Factors

Extreme flows at the wastewater treatment facility are a result of storm events in the service area, due to the interconnections between sanitary and storm sewer systems. The existing and projected sanitary component of the total flow was peaked based on the Merrimack Curve (TR-16 Guides for the Design of Wastewater Treatment Works, revised 2011 edition, prepared by the New England Interstate Water Pollution Control Commission). Estimates of peak infiltration and inflow were based on flow data measured at the facility.

Table 5.2-6 includes a summary of existing peak flow conditions and a description of how each flow condition was determined. This table also includes a comparison of the calculated peaking factors to the flow peaking factors expected for a facility of this size with a combined collection system according to MOP-8. The high peak flows observed may be artificially limited by the capacity of the influent pumps and intentional choking of influent flow by the operators to prevent flooding of the influent wet well.

Table 5.2-6 Summary of Existing Influent Flows and Flow Peaking Factors – West Side WWTP

Parameter	*Flow (mgd)*	*Peaking Factor*	*MOP 8 Peaking Factor*	*Basis of Flow Rate*
Average Daily Flow	22.1	1.0	1.0	Average of the total daily flow data in the three-year data set
Maximum Monthly Flow	29.6	1.34	1.32	98th percentile of the 30-day rolling average of the three-year data set
Maximum Day Flow	42.1	1.91	1.62	98th percentile of total daily flow data in the three-year data set
Peak Hour	81.2	3.68	N/A	98th percentile of maximum daily flow data in the three-year data set
Minimum Day Flow	14.6	0.66	0.68	2nd percentile of total daily flow data in the five-year data set

Table 5.2-7 presents a breakdown of the major components of the maximum day and peak hourly flows at the West Side WWTP. The breakdown of the maximum day and peak hourly flows in Table 5.2-7, as well as the peaking factors for maximum monthly flow and minimum flows in Table 5.2-6 were applied later in this section to select design flows through 2050.

Table 5.2-7 Existing Maximum Day and Peak Hourly Influent Flow Estimates at West Side WWTP

Parameter	Flow (mgd)
Average Day	
Sanitary Wastewater Only	8.3
Infiltration (average)	11.9
Total Dry Weather Flow (no inflow)	20.2
Inflow	1.9
Average Daily Flow	22.1
Maximum Day	
Sanitary Wastewater Only	14.1
Infiltration (maximum)	17.0
Dry Weather Flow (no inflow)	31.1
Inflow	11.0
Maximum Day Flow	42.1
Peak Hour	
Sanitary Wastewater Only	21.5
Infiltration (maximum)	17.0
Dry Weather Flow (no inflow)	38.5
Inflow	42.7
Peak Hour Flow	81.2

The following describes how the flows in Tables 5.2-6 and 5.2-7 were estimated.

Average Day Flow - Each of the average daily flow components in Table 5.2-7 is reproduced from Table 5.2-5.

Maximum Day Flow - The maximum day flow (12.7 mgd) is the 98th percentile of the daily totalized flow for each day in the three-year data set. The individual components of the maximum day flow were estimated as follows:

- *Maximum day sanitary wastewater only flow* (14.1 mgd) was calculated by taking the average day sanitary only flow (8.3 mgd) any multiplying it by a maximum day peaking factor of 1.7 estimated from the Merrimack Curve.
- *Maximum day dry-weather flow* (31.1 mgd) was determined based on the dry-day/wet-day analysis described in Table 5.2-3. The 31.1 mgd is the 98th percentile of average daily plant flow on dry days. This maximum day dry-weather flow assumes no inflow.
- *Maximum day infiltration* (17.0 mgd) was calculated as the difference between maximum day dry-weather flow and maximum day sanitary wastewater only (31.1 mgd minus 14.1 mgd). This methodology assumes that maximum day infiltration occurs concurrently with maximum day sanitary flow. If sanitary flow is, in fact, lower on the maximum day, the fraction of infiltration would increase.
- *Maximum day inflow* (11.0 mgd) was calculated as the total maximum day flow (42.1 mgd) minus the maximum day dry-weather flow (31.1 mgd), which, as noted above, both represent the 98th percentile values. Note that by using the 98th percentile with a three-year data set, the largest inflow peaks are eliminated from the dataset for maximum day calculations.

Peak Hourly Flow - The estimated peak hourly flow was determined to be 81.2 mgd. This 81.2 mgd was the 98th percentile of the maximum flow from each day. The maximum flow is assumed to be the maximum instantaneous flow. The components of the peak hourly flow were estimated as follows:

- *Peak hourly sanitary wastewater only flow* (21.5 mgd) is the average day sanitary only flow (8.3 mgd) multiplied by a peak hourly peaking factor of 2.6 estimated using the Merrimack curve.
- *Peak hourly infiltration* (17.0 mgd) was considered to be the same as maximum day infiltration because groundwater infiltration doesn't change dramatically from hour to hour.
- *Peak hourly dry weather flow* (38.5 mgd) was calculated as the sum of peak hourly sanitary wastewater only (21.5 mgd) and peak hourly infiltration (17.0 mgd).
- *Peak hourly inflow* (42.7 mgd) was determined to be the difference between peak hourly flow (81.2 mgd) and peak hour dry weather flow (38.5 mgd). Peak hourly flow was determined as the 98th percentile of the maximum instantaneous flow from each day.

5.2.1.4 Secondary Bypasses

As noted previously, the plant is permitted to bypass secondary treatment when flow, due to wet weather events, has exceeded 58 mgd. The secondary bypass directs flow directly to the effluent channel of the secondary clarifiers and recombines with the secondary effluent flow before disinfection. While the WPCA and their contractor operator, operate the WWTP in a manner to minimize secondary bypasses, they are unavoidable during many wet weather events to maintain secondary treatment and avoid future non-compliance due to washout.

Table 5.2-8 lists the secondary bypasses at each plant which have occurred over the three-year analysis period, along with the duration and total flow bypassed. Overall, the West Side WWTP has bypassed approximately 660 million gallons of flow between January 2017 and December 2019. This constitutes about 3% of the total flow.

Table 5.2-8 West Side WWTP Secondary Bypasses January 2017-December 2019

Event Duration	Date	Daily Rain Total (inches)	Volume through Bypass (MG)	Total Plant Daily Flow (MG)	BOD_5 (mg/L)	Chlorine (mg/L)	Fecal Coliform (#/100 mL)	Enterococci (#/100-mL)	TSS (mg/L)
3.5 hrs	1/3/2017	0.76	4.8	33.5	4				336
9.0 hrs	1/24/2017	1.16	11.8	42.5	93				140
7.8 hrs	3/27/2017	0.43	2.9	30.1	197				480
	3/28/2017	0.93	6.1	36.8	36				118
16.0 hrs	4/1/2017	1.25	14.3	39.1	9				82
9.3 hrs	4/4/2017	1.67	11.3	56	210				254
18.0 hrs	4/7/2017	0.89	22	38.5	9				48.5
3.0 hrs	4/21/2017	0.33	3.1	26.9	1.5				74
5.0 hrs	4/26/2017	0.73	5.4	26.4	44				154
7.0 hrs	5/5/2017	1.33	6.2	36.2	50				434
8.0 hrs	5/14/2017	1.23	7.3	28.1	5				538
11.0 hrs	5/25/2017	0.35	1.8	30.7	93				332
	5/26/2017	0.70	4.3	41.5	42				83
5.0 hrs	6/20/2017	0.00	3.7	20.3	50				142
2.0 hrs	6/24/2017	0.00	1.2	25.5	31				25.5
4.0 hrs	7/7/2017	0.00	4.3	27.5	7				31
2.0 hrs	8/5/2017	0.00	1.1	22.7	6				15.5
2.0 hrs	8/18/2017	0.00	2.3	23.8	22				25
9.0 hrs	10/24/2017	0.00	11.5	31.4	20				64
20.0 hrs	10/29/2017	0.00	11.4	46.1	16				20
4.8 hrs	1/12/2018	0.00	4.3	32.8	63				76
3.0 hrs	2/4/2018	0.00	6	24.7	112				517
5.0 hrs	2/7/2018	0.00	6.6	34	74				298

Table 5.2-8 West Side WWTP Secondary Bypasses January 2017-December 2019 (*continued)*

Event Duration	Date	Daily Rain Total (inches)	Volume through Bypass (MG)	Total Plant Daily Flow (MG)	BOD_5 (mg/L)	Chlorine (mg/L)	Fecal Coliform (#/100 mL)	Enterococci (#/100-mL)	TSS (mg/L)
9.5 hrs	2/11/2018	0.00	6.6	49.2	77				157
7.0 hrs	2/25/2018	0.00	4.5	40.7	47				616
17.0 hrs	3/2/2018	0.00	18	60.6	42				290
13.0 hrs	4/16/2018	0.00	18.1	54.5	6				178
3.0 hrs	5/15/2018	0.00	3.2	28.1	26				727
2.3 hrs	5/19/2018	0.00	1.9	30.7	11				538
4.3 hrs	6/4/2018	0.00	5.6	30.5	26				732
2.7 hrs	6/24/2018	0.00	3.8	24.4	34				286
6.5 hrs	6/28/2018	0.00	8	35.3	74				129
9.0 hrs	7/18/2018	420.00	12.7	20.1	56	0.04	420	4300	181
6.0 hrs	7/26/2018	18.00	8.4	30.8	18	0.04	18	710	53
5.8 hrs	8/13/2018	48.00	5.2	32.9	48	0.03	48	5200	152
2.0 hrs	9/6/2018	4.00	3.4	30.6	22	0	4	2000	308
12.0 hrs	9/11/2018	160.00	11.1	25.3	61	0	160	27	79
	9/12/2018	210.00	4.3	28.2	20	0	210	510	41
5.0 hrs	9/18/2018	85.00	6.6	32.5	8	0	85	2300	61
19.0 hrs	9/26/2018	32.00	26.8	52.7	39	0	32	1700	35
4.0 hrs	9/27/2018	89.00	5.5	38.2	21	0	89	8	6
3.5 hrs	9/28/2018	70.00	5	39.8	3	0	70	1350	26
12.0 hrs	10/3/2018	10.00	16.2	52.6	23	0	10	400	62
16.8 hrs	10/12/2018	7.00	14	38	44	0.01	7	830	63
6.0 hrs	10/27/2018	47200.00	7.5	39.9	138	0.04	47200	32800	374
4.8 hrs	11/6/2018	4.00	6.4	32.79	25	0	4	20	38
5.5 hrs	11/10/2018	10000.00	7.7	27.19	49	0	10000	7600	298
6.5 hrs	11/16/2018	2.00	8.93	47.19	24	0	2	1200	170
4.5 hrs	11/25/2018	9600.00	7.5	37.79	84	0.01	9600	13500	461
10.2 hrs	11/27/2018	7.00	13.8	40.99	13	0.01	7	2	56
9.5 hrs	12/2/2018	64.00	13	47.39	45	0	64	120	87
10.5 hrs	12/21/2018	2.00	14.4	49.69	27	0	2	760	90
8.5 hrs	12/28/2018	18.00	11.9	41.59	66	0	18	6	46
9.0 hrs	1/1/2019	20.00	12.9	44.59	66	0	20	80	172
3.5 hrs	1/5/2019	1100.00	4.7	41.39	66	0	1100	3400	172
10.8 hrs	1/20/2019	4400.00	15	46.69	52	0	4400	4700	59
10.7 hrs	1/24/2019	6.00	14.5	45.39	59	0	6	7	82

Table 5.2-8 West Side WWTP Secondary Bypasses January 2017-December 2019 (*continued*)

Event Duration	Date	Daily Rain Total (inches)	Volume through Bypass (MG)	Total Plant Daily Flow (MG)	BOD_5 (mg/L)	Chlorine (mg/L)	Fecal Coliform (#/100 mL)	Enterococci (#/100-mL)	TSS (mg/L)
5.0 hrs	2/24/2019	960.00	5.1	40.69	33	0	960	5800	111
3.8 hrs	3/10/2019	500.00	3.2	34.89	19	0	500	5400	377
4.1 hrs	4/13/2019	88.00	4.4	36.79	31	0	88	10	217
3.0 hrs	4/15/2019	210.00	3.9	31.89	15	0	210	300	40
3.0 hrs	4/20/2019	10.00	3.2	33.09	12	0	10	330	128
3.2 hrs	4/22/2019	1800.00	3.1	33.89	61	0.04	1800	4300	67
10.0 hrs	4/26/2019	3.00	11.9	38.29	38	0	3	97	33
11.5 hrs	5/12/2019	5300.00	5.9	42.09	129	0	5300	18500	302
	5/13/2019	1.00	6.3	38.19	37	0	1	3	50
16.5 hrs	5/30/2019	10800.00	5.3	30.19	107	0	10800	13800	117
	5/31/2019	30.00	6.4	24.19	32	0	30	30	32
5.2 hrs	6/19/2019	12.00	7	21.79	181	0	12	5100	171
3.8 hrs	7/12/2019	70000.00	3.9	28.29	353	0	70000	17600	392
18.8 hrs	7/17/2019	230.00	8.3	32.19	87	0	230	10400	145
	7/18/2019	11000.00	11.3	48.29	43	0	11000	1100	140
11.0 hrs	7/22/2019	1.00	9.7	33.19	27	0	1	1000	34
	7/23/2019	5.00	6	43.39	18	0	5	400	22
3.5 hrs	8/28/2019	20000.00	4.88	29.09	29	0	20000	20000	51
8.25 hrs	10/17/2019	3.00	11.5	27.69	19	0.03	3	920	28
10.0 hrs	10/27/2019	820.00	13.28	41.39	28	0.02	820	5400	103
4.25 hrs	11/24/2019	400.00	5.26	37.49	27	0.02	400	1200	63
9.0 hrs	12/9/2019	9.00	3.4	41.99	43	0.03	9	260	52
	12/10/2019	1.00	12	30.29	37	0	1	210	42
17.0 hrs	12/14/2019	40.00	23.96	64.79	28	0	40	600	38

5.2.1.5 Historical Wastewater Characteristics and Loads

The NPDES permit specifies the sample collection and testing methodologies for each parameter. To achieve the majority of the effluent reporting requirements, a 24-hour flow-based composite sample of the final effluent is collected after the introduction of sodium bisulfite. The sample is drawn from the effluent channel downstream of the chlorine contact tanks after the introduction of sodium bisulfite. From this sample, the required parameters are measured and reported on the MORs and monthly Discharge Monitoring Reports (DMRs) submitted to CT DEEP. Influent sampling is performed in a similar manner from the control well upstream of the Headworks Building.

Process control data are collected from many different locations throughout the facility, using both grab and composite samples. **Table 5.2-9** describes the location of each composite sampler used for compliance and/or process control. All composite samplers collect the sample for the period of 12:00 a.m. through 11:59 p.m., automatically switching to a new sample container at the beginning of each day.

Table 5.2-9 24-hour Composite Samplers

Flow Sampled	Sampler Location
Raw Influent	Control Well, upstream of Headworks
Primary Effluent	In the channel downstream of primary clarifiers.
Final Effluent	In the downstream end of effluent channel from the chlorine contact tanks.

CDM Smith used daily concentration data from the WPCA's MORs to determine the daily influent loads to the treatment plant. The WWTP's laboratory analyzes the raw influent, primary effluent and final effluent composite samples for total suspended solids (TSS), total Kjeldahl nitrogen (TKN), and five-day biochemical oxygen demand (BOD_5).

Figures 5.2-2 through **5.2-4** present the raw influent BOD_5, TSS, and TKN loads in comparison to their respective 30-day rolling averages. The daily influent loads shown were calculated using the raw influent daily concentrations from 2017 to 2019 and their corresponding average daily flows. The WWTP analyzes for BOD_5, TSS and TKN three days per week, except for the raw influent and primary effluent TKN which is analyzed two days per week. The figures illustrate that the influent loads to the facility have been relatively consistent throughout the past three years.

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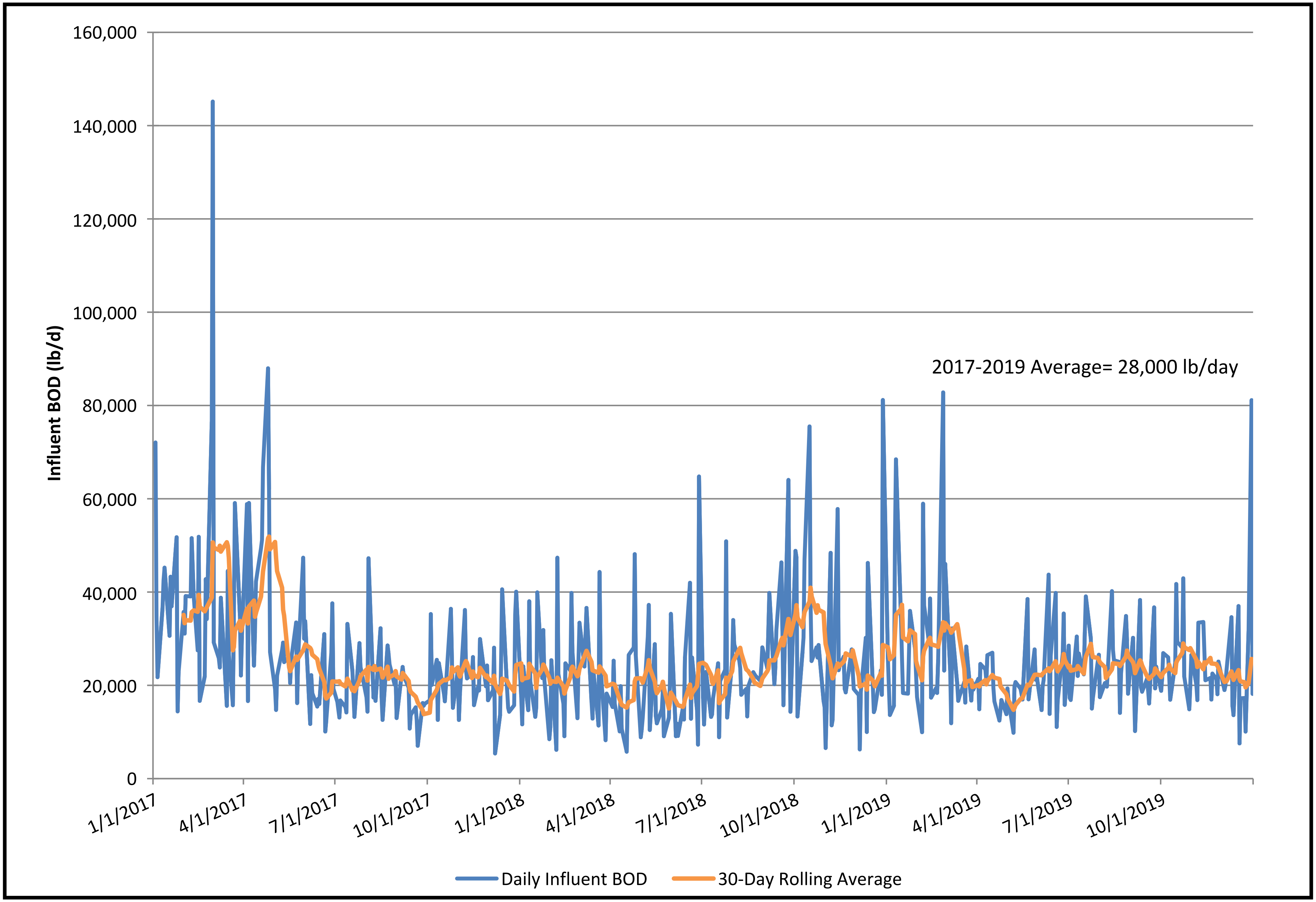

Figure 5.2-2
West Side WWTP Influent BOD5 Loading 2017-2019

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City of Bridgeport
Facilities Plan

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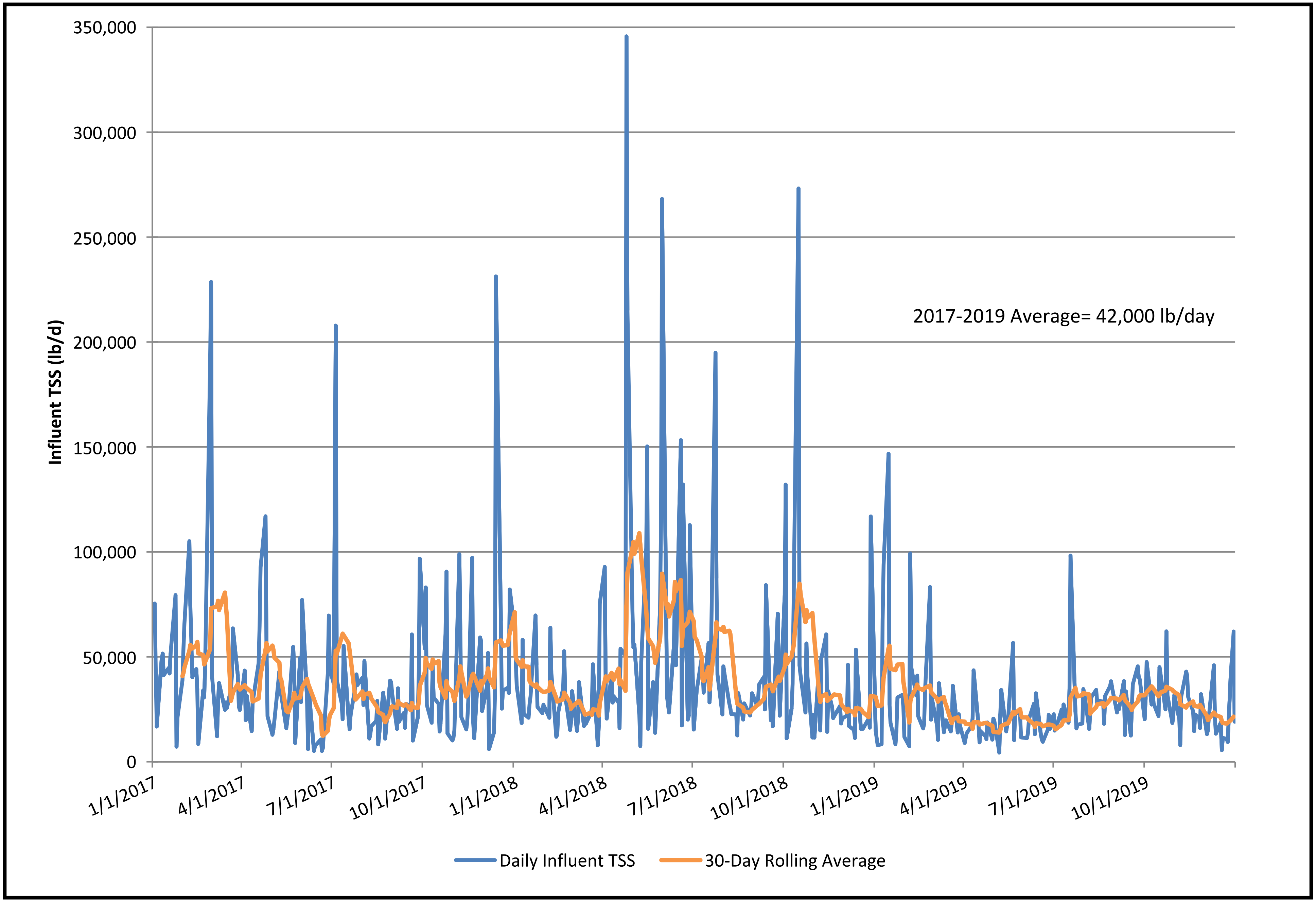

Figure 5.2-3
West Side WWTP Influent TSS Loading 2017-2019

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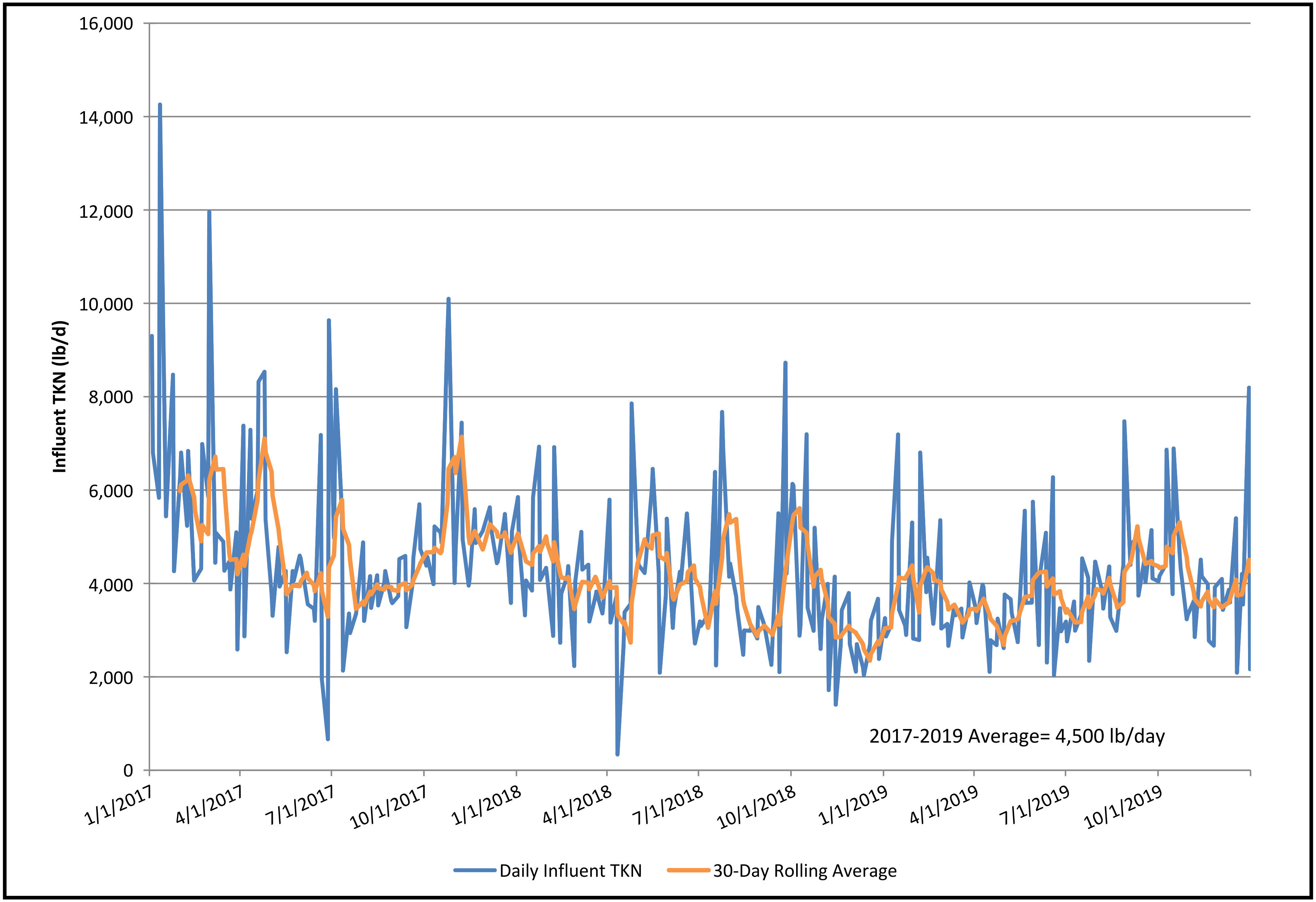

CDM Smith

Figure 5.2-4
West Side WWTP Influent TKN Loading 2017-2019

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Table 5.2-10 compares the WWTP's average influent BOD_5, TSS, and TKN concentrations with typical low and medium strength domestic wastewater concentrations taken from Metcalf & Eddy *Wastewater Engineering Treatment and Resource Recovery, 5th Edition* (2013) (Metcalf & Eddy). The BOD_5, and TKN concentrations are in between typical concentrations for low and medium strength wastewaters. The TSS concentration is very close to the medium strength wastewater. All parameters generally show good correlation with expected values.

Table 5.2-10 Average West Side WWTP Raw Wastewater Concentrations vs. Typical Average Concentrations

Constituent	Average West Side WWTP Concentration (mg/L)	Average Typical Concentrations (mg/L) [1]	
		Low Strength	Medium Strength
Biochemical oxygen demand (BOD_5)	140	110	190
Total suspended solids (TSS)	211	120	210
Total Kjeldahl nitrogen (TKN)	24	20	40

[1] *Low and medium strength wastewater concentrations from* Wastewater Engineering Treatment and Resource Recovery, *5th Ed. (2013).*

Table 5.2-11 compares the WWTP's average influent parameter relationships with typical wastewater ratios from Metcalf & Eddy and MOP-8. The values are all within the range of typical wastewater constituent ratios over the three-year period except for the BOD_5:TSS ratio which is lower than typical. This is likely due to increased TSS loadings from the combined sewer system.

Table 5.2-11 Average West Side WWTP Raw Wastewater Ratios vs. Typical Ratios

Year	BOD_5:TSS	BOD_5:TKN	BOD_5:TP	NH_3:TKN
2017	0.66	5.41	N/A	0.53
2018	0.51	5.89	27.19	0.56
2019	0.92	6.22	42.60	0.61
Average	0.70	5.84	34.90	0.57
Typical	*0.82 to 1.43*	*4.2 to 7.1*	*20 to 50*	*0.5 to 0.8*

As previously discussed, all septage enters the treatment facility at the influent to the headworks. The plant influent sampler is upstream of headworks, so all raw influent loads do not include the effects of septage from the septage receiving facility.

According to historical monthly data, the average daily volume of septage was 42,500 gallons per day (gpd). Industry standard septage concentrations consistent with TR-16 were applied to the average daily septage flow of 42,500 gpd to determine corresponding influent loads. These septage loads are presented in **Table 5.2-12**.

Table 5.2-12 Septage Constituent Concentrations and Loads

Constituent	Industry Standard Septage Concentrations (mg/L)	Average West Side Influent Load (lbs/day)
BOD_5	6,500	2,300
TSS	12,900	4,600
TKN	600	210
TP	210	74

Adding these influent septage loads to the influent loads results in the total influent loads to the plant. **Table 5.2-13** presents these loads and calculated concentrations.

Table 5.2-13 Influent Wastewater Pollutant Loads and Concentrations

Parameter	Septage Load (lbs/day)	Sewer System Influent Load (lbs/day)	Total Influent Load (lbs/day)	Sewer System Per Capita Influent Load (lbs/day)
BOD_5	2,300	25,300	27,600	0.19
TSS	4,600	37,300	41,900	0.28
TKN	210	4,300	4,500	0.03
TP	74	710	780	0.005

The concentrations in **Table 5.2-14** are close to those seen in low strength wastewater (previously presented in Table 5.2-10), indicating I/I greatly affects the sewer system, as would be expected in a partially combined collection system. Note that the per capita loads divide the total collection system load by the population and thus include commercial and industrial flows, in addition to residential flows. Typical per capita loads from TR-16 and from Metcalf & Eddy are compared to the calculated values in **Table 5.2-15** below.

Table 5.2-14 Influent Wastewater Pollutant Loads and Concentrations

Parameter	Total Influent Load (lbs/day)	Sewer System Per Capita Influent Load (lbs/day)	Sewer System Influent Concentration (mg/L)
BOD_5	28,000	0.19	140
TSS	42,000	0.28	211
TKN	4,500	0.03	24

Table 5.2-15 Comparison of Per Capita Loading to Typical Values

Parameter	TR-16 Typical Loads (lbs/capita/day)	M&E 2003 Typical Loads (lbs/capita/day)	Actual Estimated Loads (lbs/capita/day)
BOD_5	0.17 to 0.22	0.11 to 0.26	*0.19*
TSS	0.20 to 0.25	0.13 to 0.33	*0.28*
TKN	0.04	0.02 to 0.048	*0.03*

All per capita loads are within the Metcalf & Eddy range. The TSS load is higher than the TR-16 guidelines and the TKN is slightly lower. This could be a factor of I/I or due to industrial flows with higher solids loadings than residential flow.

5.2.1.6 Peak Loads and Load Peaking Factors

Table 5.2-16 presents the wastewater load peaking factors calculated from the plant data and compares them to MOP-8 for a combined collection system.

Table 5.2-16 Peaking Factors for Wastewater Influent Loads

Constituent	Maximum Month		Maximum Day		Minimum Day	
	MOP 8	Calculated	MOP 8	Calculated	MOP 8	Calculated
BOD_5	1.26	1.51	1.61	1.98	0.60	0.38
TSS	1.31	1.92	1.88	2.53	0.53	0.23
TKN	1.24	1.44	1.40	1.74	0.67	0.50
TP	1.20	1.50	1.36	1.83	0.73	0.45

The peaking factors for each design condition were calculated as follows:

- *Maximum day peaking factors* for BOD_5, TSS, TKN and TP were calculated by dividing the 95th percentile of the daily load data by the overall average mass load.
- *Maximum month peaking factors* for BOD_5, TSS, TKN and TP were calculated by dividing the 95th percentile of the 30-day rolling average of the daily load data by the overall average mass load.
- *Minimum day peaking factors* were calculated based on the 5th percentile of the daily load data divided by the overall average mass load.

Table 5.2-17 presents the resulting existing average and peak influent loads (including septage) based on the actual influent data as summarized above.

Table 5.2-17 Existing Average and Peak Influent Loads

Parameter	Load (lb/day)
BOD_5	
Average Daily Load	28,000
Maximum Monthly Load	42,000
Maximum Day Load	55,000
Minimum Day Load	11,000
TSS	
Average Daily Load	42,000
Maximum Monthly Load	80,000
Maximum Day Load	110,000
Minimum Day Load	9,800
TKN	
Average Daily Load	4,500
Maximum Monthly Load	6,400
Maximum Day Load	7,800
Minimum Day Load	2,200
TP	
Average Daily Load	780
Maximum Monthly Load	1,200
Maximum Day Load	1,400
Minimum Day Load	350

5.2.1.7 Removal Efficiencies Across Primary Settling Tanks and Primary Effluent Loads

Treatment process removal efficiencies were estimated across the primary settling tanks (PSTs) and the entire treatment facility over the three-year data set as follows:

- Average BOD_5, TSS, and TKN removal efficiencies across the primary tanks are -19%, -123%, and -4% respectively.
- Average BOD_5, TSS, and TKN removal efficiencies through the entire facility are 91%, 81%, and 72% respectively.

The negative removal efficiencies for the primary tanks are likely the result of a sidestream load that is being added before or after the primary tanks (ahead of the primary effluent sampler). Since the influent sampler is upstream of the headworks, any load added after the sampler is not accounted for in the removal efficiencies. Assumptions for future removals will use industry standard values for primary treatment. **Table 5.2-18** summarizes the calculated primary effluent loads based on existing influent and primary effluent data. TP loadings are not available as it is not sampled from the primary effluent.

Table 5.2-18 Existing Average and Peak Primary Effluent Loads

Parameter	2017-2019 Load (ppd)
Average Day	
BOD_5	32,000
TSS	72,000
TKN	4,500
Maximum Day	
BOD_5	70,000
TSS	210,000
TKN	8,100
Maximum Month	
BOD_5	56,000
TSS	160,000
TKN	6,000

The loads presented in Table 5.2-18 include the loads due to sidestreams. There is one main source of sidestream loading at the West Side WWTP:

- Gravity Thickener Supernatant – discharges upstream of headworks

The loads for the sidestreams were not calculated due to lack of flow or concentration data, and data inconsistencies. However, since the primary effluent sampler is located downstream of all sidestream addition points, it is assumed that the primary effluent loads account for all influent and sidestream loads.

5.2.2 East Side WWTP Existing Flows and Loads

Wastewater treated at the East Side WWTP is generated from several sources including residential, municipal, commercial, institutional, and industrial properties, and I/I. The service area includes both separated and combined sewers. Although the WPCA has constructed many sewer separation projects, there are still areas with fully combined sewers. It is estimated that slightly more than 40% of the East Side service area consists of combined sewers.

Figure 5.2-5 illustrates the average flow for the East Side WWTP from the collection system as measured at the WWTP's influent Parshall flume, along with the 30-day rolling average flow. The NPDES permit lists an average design flow rate of 10 mgd. During wet weather, if flow exceeds 24 mgd, the excess flow only requires primary and disinfection treatment and can bypass the secondary treatment process. All flow discharges through a single outfall into Bridgeport Harbor.

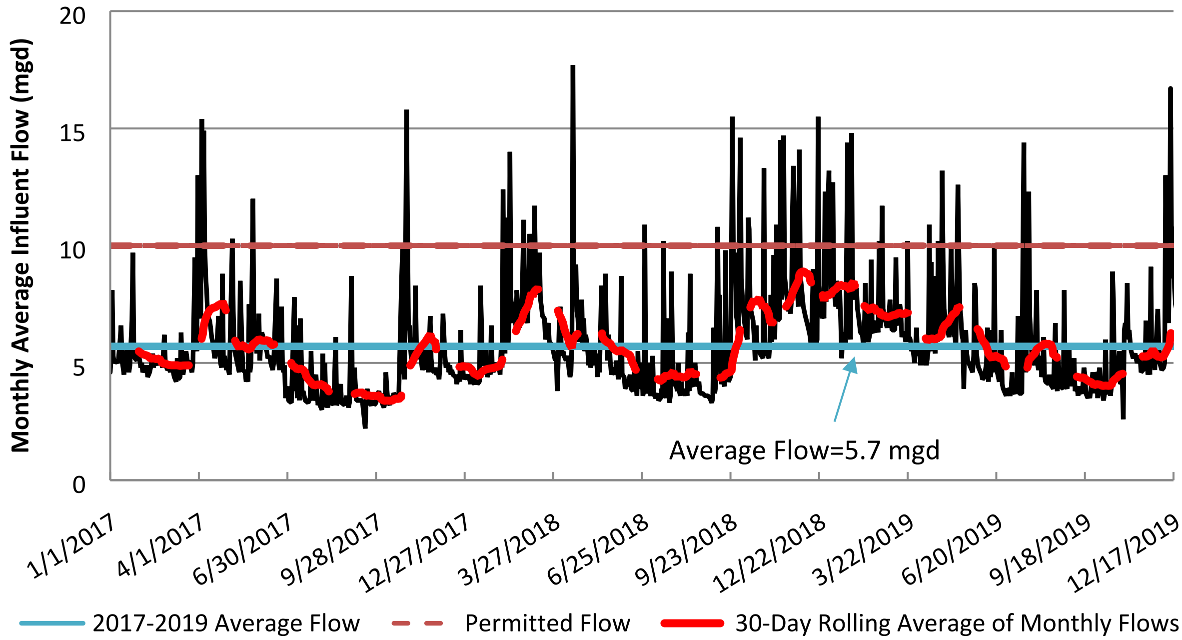

Figure 5.2-5
East Side WWTP Average Influent Flow 2017-2019

The average daily influent flow from January 1, 2017 to December 31, 2019 was 5.7 mgd, which is 57 percent of the permitted capacity of 10 mgd. The 30-day rolling average flow also remained comfortably below the permit limit.

5.2.2.1 Breakdown of Flows by Type

This section breaks down the East Side WWTP's influent flows by type. Specifically, flows are divided into the following categories: 1) sanitary and 2) infiltration and inflow. Estimated sanitary wastewater flows were determined for Bridgeport which included all residential, commercial, institutional, and industrial flows. As discussed in Section 5.2.1 above, the sanitary flow component attributed to the East Side collection system is 2.3 mgd.

The East Side WWTP does not accept septage. However, scum from the West Side WWTP is conveyed by tanker truck to the East Side WWTP. The tanker truck is decanted with the liquid discharged into the influent flow. Volume and concentration data are not measured, but the volume of decant liquid is considered minimal in comparison to the overall influent flow. The load associated with this is discussed later in this section.

Infiltration and inflow were determined for the East Side WWTP using the same methodology as described for the West Side WWTP. The average daily dry day flow over the three-year period of 2017 through 2019 was determined to be 5.2 mgd at East Side WWTP, as compared to the overall average daily flow of 5.7 mgd. Therefore, the average annual inflow was estimated to be 0.5 mgd at the East Side WWTP. This includes suspected inflow due to tidal influence, as some of the combined sewer regulator weirs are below high tide and defective tide gates allow water to enter the sewer system during high tides and storm surges.

To develop an estimate of infiltration entering the system, the sanitary flow component (2.3 mgd) and the inflow flow component (0.5 mgd) were subtracted from the average daily flow to obtain a value of 2.9 mgd.

5.2.2.2 Summary of Flow Components – East Side WWTP

Using the information described above, average daily flows were divided into their various components for the East Side WWTP, as shown in **Table 5.2-19**. Following Table 5.2-19, the methodology for calculation of each value shown is reviewed.

Table 5.2-19 Average Daily East Side WWTP Influent Flow Breakdown by Flow Type

Parameter	Average Annual (mgd, 2017-2019)
Average Day	
Sanitary Wastewater Only	2.3
Infiltration (average)	2.9
Total Dry Weather Flow (no inflow)	5.2
Inflow	0.5
Average Daily Flow	5.7

The following summarizes how the values in Table 5.2-19 were estimated:

- The sanitary wastewater only flow (2.3 mgd) was calculated by multiplying the average water usage within the East Side service area by 90% to account for water that is not returned to the sewer system. The sanitary-only flow is assumed to be the base wastewater flow attributable to all registered connections to the sanitary sewer system including commercial and industrial users, excluding I/I, and thus is largely independent of seasonal fluctuations.

- The average dry weather flow (5.2 mgd) was calculated as the average plant influent flow on dry days as defined in Table 5.2-4. It is assumed that there is no inflow to the system on dry days.

- The average annual inflow (0.5 mgd) was calculated as the total average daily flow (including wet days) minus the average dry day flow for the full three-year data set (5.7 minus 5.2 mgd).

- Average annual infiltration (2.9 mgd) consists of groundwater and tidal infiltration. The groundwater infiltration was calculated by subtracting the sanitary wastewater flow from the average dry weather flow (5.2 minus 2.3 mgd).

- The total average daily flow (5.7 mgd) is the average influent plant flow measured by the influent flume over the three-year time period (January 2017-December 2019).

Based on this flow analysis, combined I/I is estimated to average 3.4 mgd during the three-year period analyzed, representing approximately 60 percent of the total flow at the facility on an average annual basis. This 60 percent of total flow as I/I is much higher than the projected I/I reported in the 2000 Facility Plan, which was 25 percent.

5.2.2.3 Peak Flows and Flow Peaking Factors

Extreme flows at the wastewater treatment facility are a result of storm events in the service area, due to the interconnections between sanitary and storm sewer systems. The existing and projected sanitary component of the total flow was peaked based on the Merrimack Curve from TR-16. Estimates of peak infiltration and inflow were based on flow data measured at the facility. The influent flow is a measurement of the flow accepted at the plant and not the total flow from the collection system. At extreme flows, the WWTP operators throttle influent gates to prevent influent flow from exceeding plant capacity. The peaking factors calculated in this section may be lower than the actual peaking factors that the plant could experience if the influent gate were not throttled.

Table 5.2-20 includes a summary of existing peak flow conditions and a description of how each flow condition was determined. This table also includes a comparison of the calculated peaking factors to the flow peaking factors expected for a facility of this size with a combined collection system according to MOP-8. The high peak flows observed may be artificially limited by the capacity of the influent pumps and intentional choking of influent flow by the operators to prevent flooding of the influent wet well.

Table 5.2-20 Summary of Existing Influent Flows and Flow Peaking Factors – East Side WWTP

Parameter	Flow (mgd)	Peaking Factor	MOP 8 Peaking Factor	Basis of Flow Rate
Average Daily Flow	5.7	1.0	1.0	Average of the total daily flow data in the three-year data set
Maximum Monthly Flow	8.3	1.46	1.32	98th percentile of the 30-day rolling average of the three-year data set
Maximum Day Flow	12.7	2.23	1.62	98th percentile of total daily flow data in the three-year data set
Peak Hour	28.2	4.94	N/A	98th percentile of maximum daily flow data in the three-year data set
Minimum Day Flow	3.3	0.58	0.68	2nd percentile of total daily flow data in the five-year data set

Table 5.2-21 presents a breakdown of the major components of the maximum day and peak hourly flows at the East Side WWTP. The breakdown of the maximum day and peak hourly flows in Table 5.2-21, as well as the peaking factors for maximum monthly flow and minimum flows in Table 5.2-20 were applied later in this section to select design flows through 2050.

Table 5.2-21 Existing Maximum Day and Peak Hourly Influent Flow Estimates at East Side WWTP

Parameter	Flow (mgd)
Average Day	
Sanitary Wastewater Only	2.3
Infiltration (average)	2.9
Total Dry Weather Flow (no inflow)	5.2
Inflow	0.5
Average Daily Flow	5.7
Maximum Day	
Sanitary Wastewater Only	4.7
Infiltration (maximum)	5.1
Dry Weather Flow (no inflow)	9.8
Inflow	3.0
Maximum Day Flow	12.7
Peak Hour	
Sanitary Wastewater Only	7.5
Infiltration (maximum)	5.1
Dry Weather Flow (no inflow)	12.6
Inflow	15.6
Peak Hour Flow	28.2

The following describes how the flows in Tables 5.2-20 and 5.2-21 were estimated.

Average Day Flow - Each of the average daily flow components in Table 5.2-21 is reproduced from Table 5.2-19.

Maximum Day Flow - The maximum day flow (12.7 mgd) is the 98th percentile of the daily totalized flow for each day in the three-year data set. The individual components of the maximum day flow were estimated as follows:

- *Maximum day sanitary wastewater only flow* (4.7 mgd) was calculated by taking the average day sanitary only flow (2.3 mgd) any multiplying it by a maximum day peaking factor of 2.0 estimated from the Merrimack Curve.
- *Maximum day dry-weather flow* (9.8 mgd) was determined based on the dry-day/wet-day analysis described in Table 2. The 9.8 mgd is the 98th percentile of average daily plant flow on dry days. This maximum day dry-weather flow assumes no inflow.
- *Maximum day infiltration* (5.1 mgd) was calculated as the difference between maximum day dry-weather flow and maximum day sanitary wastewater only (9.8 mgd minus 4.7 mgd). This methodology assumes that maximum day infiltration occurs concurrently with maximum day sanitary flow. If sanitary flow is, in fact, lower on the maximum day, the fraction of infiltration would increase.
- *Maximum day inflow* (3.0 mgd) was calculated as the total maximum day flow (12.7 mgd) minus the maximum day dry-weather flow (9.8 mgd), which, as noted above, both represent the 98th percentile values. Note that by using the 98th percentile with a three-

year data set, the largest inflow peaks are eliminated from the dataset for maximum day calculations.

Peak Hourly Flow - The estimated peak hourly flow was determined to be 28.2 mgd. This 28.2 mgd was the 98th percentile of the maximum flow from each day. The maximum flow is assumed to be the maximum instantaneous flow. The components of the peak hourly flow were estimated as follows:

- *Peak hourly sanitary wastewater only flow* (7.5 mgd) is the average day sanitary only flow (2.3 mgd) multiplied by a peak hourly peaking factor of 3.2 estimated using the Merrimack curve.
- *Peak hourly infiltration* (5.1 mgd) was considered to be the same as maximum day infiltration because groundwater infiltration doesn't change dramatically from hour to hour.
- *Peak hourly dry weather flow* (12.6 mgd) was calculated as the sum of peak hourly sanitary wastewater only (7.5 mgd) and peak hourly infiltration (5.1 mgd).
- *Peak hourly inflow* (15.6 mgd) was determined to be the difference between peak hourly flow (28.2 mgd) and peak hour dry weather flow (12.6 mgd). Peak hourly flow was determined as the 98th percentile of the maximum instantaneous flow from each day.

5.2.2.4 Secondary Bypasses

As noted previously, the plant is permitted to bypass secondary treatment when flow has exceeded 24 mgd, due to wet weather events. The secondary bypass directs flow directly to the effluent channel of the secondary clarifiers and recombines with secondary effluent flow before disinfection. While the WPCA operates the WWTP in a manner so as to avoid secondary bypasses, this is required during extreme peak flow events to maintain secondary treatment and avoid future non-compliance due to washout.

Table 5.2-22 lists the secondary bypasses at each plant which have occurred over the three-year analysis period, along with the duration and total flow bypassed. Overall, the East Side WWTP has bypassed approximately 82 million gallons of flow between January 2017 and December 2019. This constitutes about 1% of the total flow.

Table 5.2-22 East Side WWTP Secondary Bypasses January 2017-December 2019

Event Duration	Date	Daily Rain Total (inches)	Volume through Bypass (MG)	Total Plant Daily Flow (MG)	BOD_5 (mg/L)	Chlorine (mg/L)	Fecal Coliform (#/100 mL)	Enterococci (#/100-mL)	TSS (mg/L)
5.8 hrs	4/4/2017	1.35	5.8	15.4	23	0.03	11	1200	48.5
3.0 hrs	4/6/2017	1.03	1.75	14.9	8	0.04	1	1	49
2.8 hrs	5/5/2017	1.40	1.4	10.3	15	0.03	20	400	20
1.8 hrs	5/26/2017	1.40	0.89	12	18	0.03	600	3400	21.5
7.8 hrs	10/25/2017	0.03	4.29	10	13	0.01	13	280	49
2.8 hrs	2/8/2018	0.75	1.08	11.2	37	N/A	28	39	54
8.0 hrs	3/2/2018	0.24	3.91	8.2	15	0.03	86	2200	35
6.0 hrs	4/16/2018	1.80	3.49	17.7	6	0.04	53	1500	107
2.3 hrs	5/15/2018	0.67	1.42	8.3	28	0.1	20000	20000	62
7.5 hrs	6/28/2018	0.51	2.27	10.9	38	0.03	120	6400	313
	6/29/2018	0.90	1.2	4.1	14	0.03	20000	20000	18
4.3 hrs	7/17/2018	2.33	2.04	10.2	38	0.04	78	4100	35
3.0 hrs	8/13/2018	0.72	1.08	8.8	9	0.01	1960	4900	18
3.1 hrs	9/12/2018	0.94	1.21	7.9	15	0.01	260	270	22
3.0 hrs	9/18/2018	1.28	1.53	9.8	2	0.02	3	2300	10
13.3 hrs	9/25/2018	3.48	4.51	15.5	21	0.02	20	1350	7.5
	9/26/2018	0.40	1.06	9.4	10	0.03	31	10	12
9.3 hrs	10/2/2018	3.62	4.74	11.3	15	0.03	6	270	5
2.8 hrs	10/11/2018	1.60	1.17	11.2	18	0.03	98	1110	18
3.3 hrs	11/13/2018	0.65	1.53	14.5	13	0.04	18	104	29
5.5 hrs	11/26/2018	0.92	2.76	13.4	15	0.02	12	10	12
6.8 hrs	12/21/2018	1.44	3.01	15.5	8	0.03	54	420	8
6.1 hrs	1/1/2019	1.47	3.18	13.2	6	0.02	80	176	19
7.1 hrs	1/24/2019	1.66	3.07	14.8	60	0.03	24	6	28
3.0 hrs	4/27/2019	1.35	1.87	9.4	14	0.03	200	6900	12
2.5 hrs	7/12/2019	0.65	1.42	7	42	0.02	85000	24000	42
12.1 hrs	7/17/2019	1.63	3.29	9.9	35	0.02	20000	23600	32
	7/18/2019	1.90	2.4	14.4	22	0.03	9900	1700	6
7.0 hrs	7/23/2019	2.54	3.02	12.3	22	0.02	20000	12200	18
3.5 hrs	7/31/2019	0.75	1.44	8.1	30	0.03	20000	14400	33
3.1 hrs	8/28/2019	1.00	1.51	8.1	19	0.01	20000	2600	30
8.3 hrs	10/17/2019	1.63	3.67	7.7	40	0.03	9	6	29
8.0 hrs	10/27/2019	2.31	3.06	2.6	18	0.03	200	1020	19
4.0 hrs	12/9/2019	1.57	1.6	13	26	0.04	20	1320	20

5.2.2.5 Historical Wastewater Characteristics and Loads

The NPDES permit specifies the sample collection and testing methodologies for each parameter. To achieve the majority of the effluent reporting requirements, a 24-hour flow-based composite sample of the final effluent is collected. The sample is drawn from the effluent channel downstream of the chlorine contact tanks. From this sample, the required parameters are measured and reported on the MORs and monthly DMRs submitted to CT DEEP. Influent sampling is performed in a similar manner at the manhole upstream of Junction Chamber A and the Headworks Building.

Process control data are collected from many different locations throughout the facility, using both grab and composite samples. **Table 5.2-23** describes the location of each composite sampler used for compliance and/or process control. All composite samplers collect the sample for the period of 12:00 a.m. through 11:59 p.m., automatically switching to a new sample container at the beginning of each day.

Table 5.2-23 24-hour Composite Samplers

Flow Sampled	Sampler Location
Raw Influent	Manhole upstream of Junction Chamber A, which is upstream of Headworks
Primary Effluent	In the channel downstream of primary clarifiers.
Final Effluent	In the junction chamber downstream of effluent channel from the chlorine contact tanks.

CDM Smith used daily concentration data from the WPCA's MORs to determine the daily influent loads to the treatment plant. The WWTP's laboratory analyzes the raw influent, primary effluent and final effluent composite samples for TSS, TKN, and BOD_5.

Figures 5.2-6 through **5.2-8** present the raw influent BOD_5, TSS, and TKN loads in comparison to their respective 30-day rolling averages. The daily influent loads shown were calculated using the raw influent daily concentrations from 2017 to 2019 and their corresponding average daily flows. The WWTP analyzes for BOD_5, TSS and TKN three days per week, except for the raw influent and primary effluent TKN which is analyzed two days per week. The figures illustrate that the influent loads to the facility have been relatively consistent throughout the past three years.

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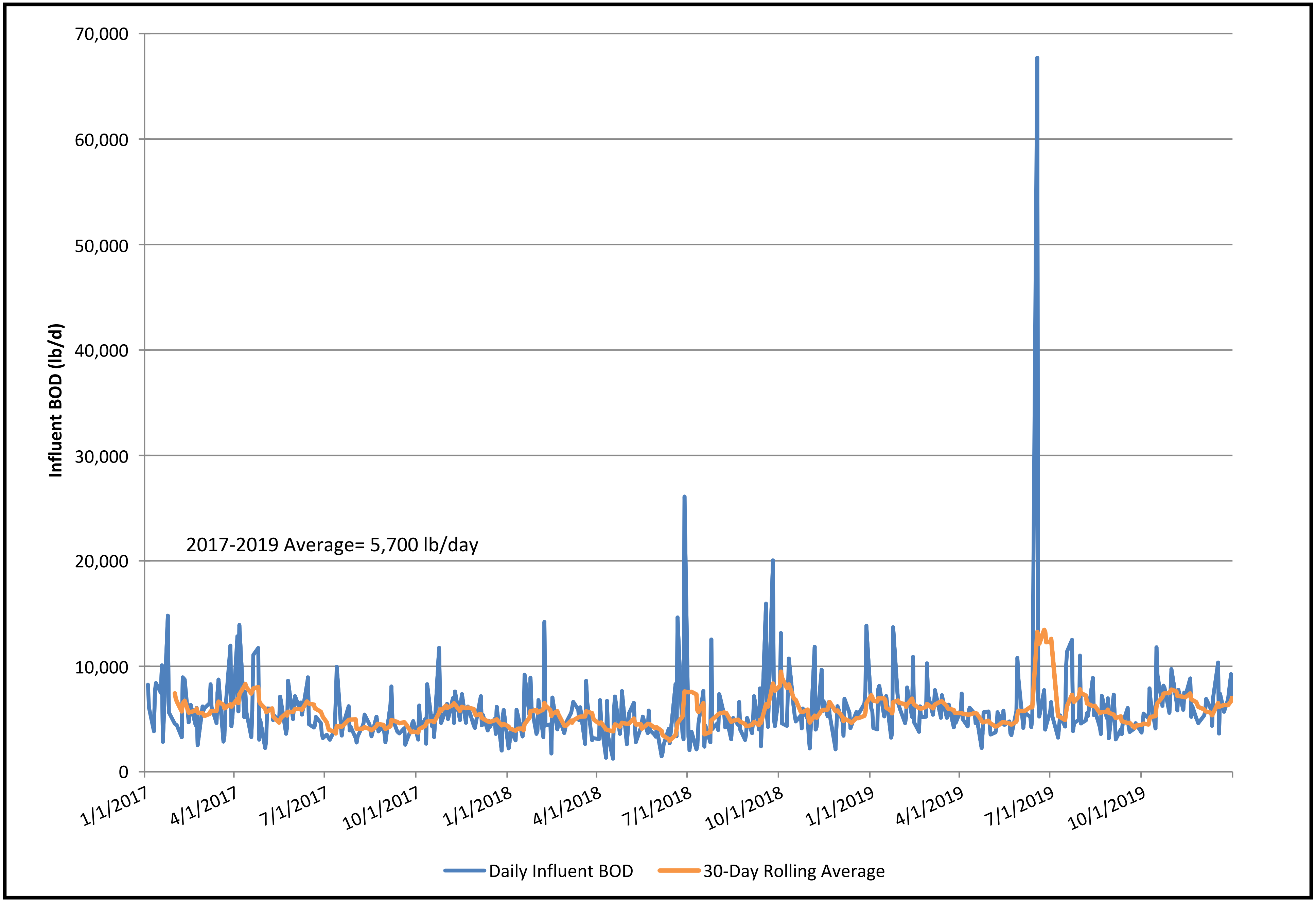

Figure 5.2-6
East Side WWTP Influent BOD_5 Loading 2017-2019

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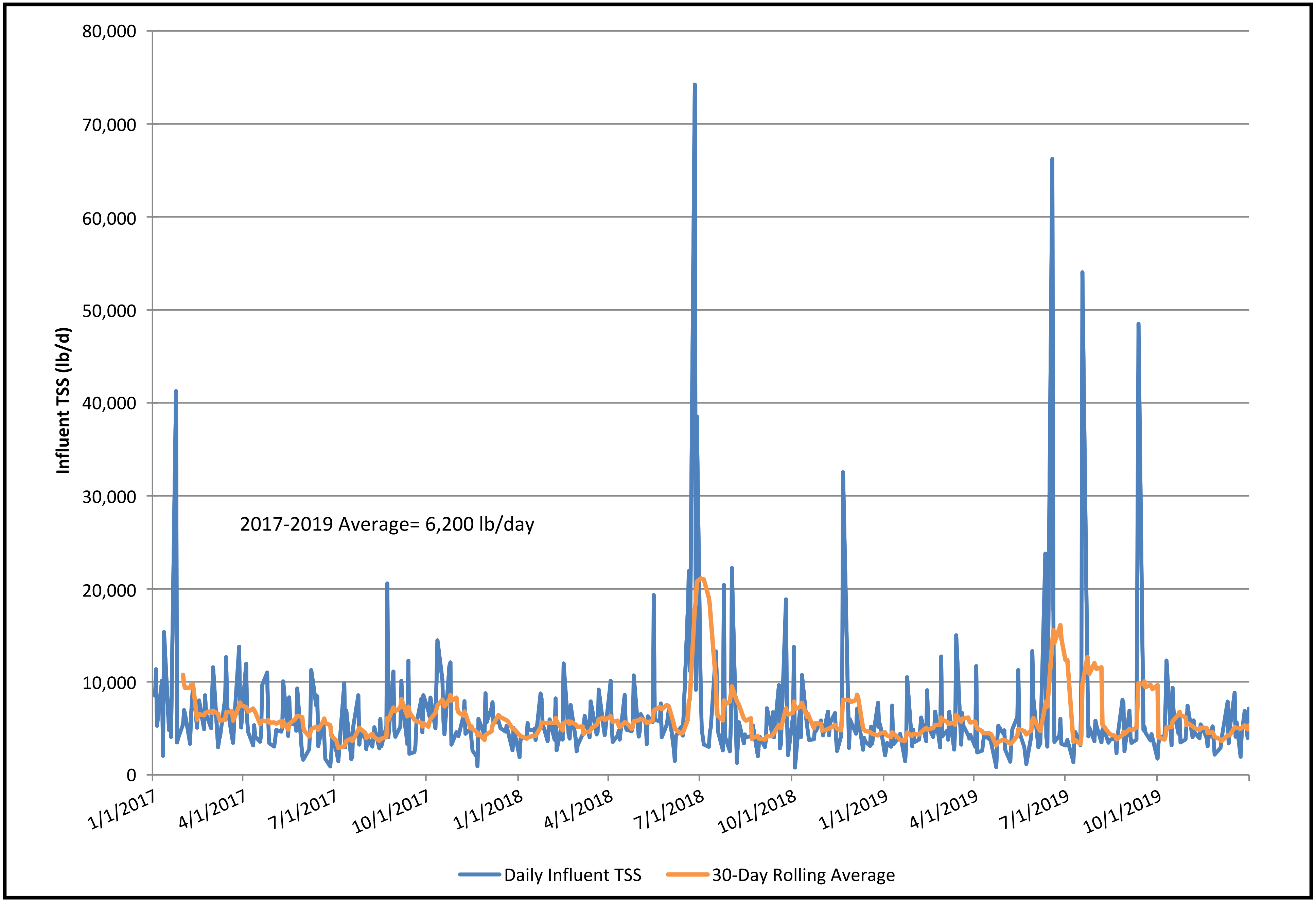

Figure 5.2-7
East Side WWTP Influent TSS Loading 2017-2019

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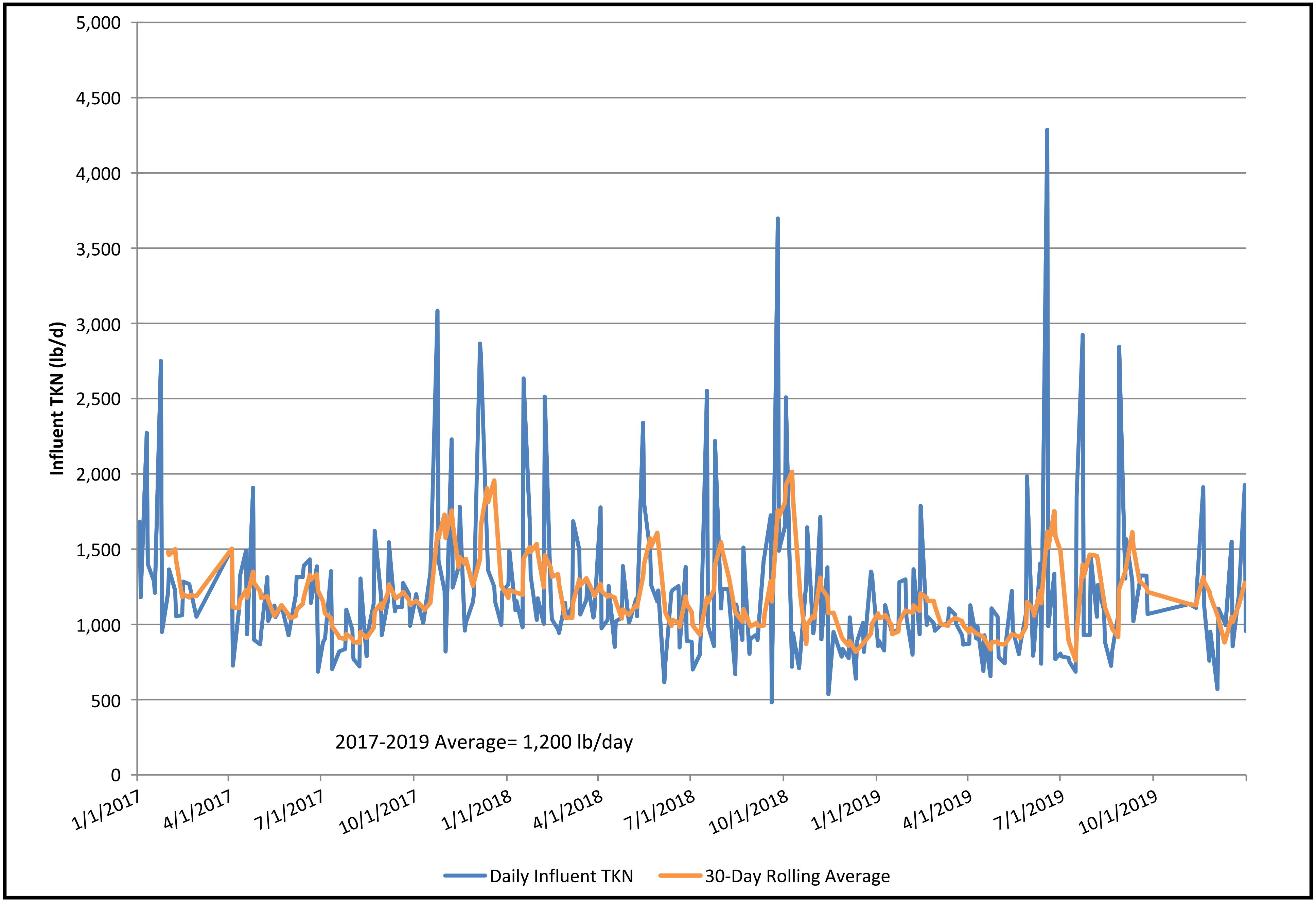

Figure 5.2-8
East Side WWTP Influent TKN Loading 2017-2019

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Table 5.2-24 compares the WWTP's average influent BOD_5, TSS, and TKN concentrations with typical low and medium strength domestic wastewater concentrations taken from Metcalf & Eddy *Wastewater Engineering Treatment and Resource Recovery, 5th Edition* (2013) (Metcalf & Eddy). The BOD_5, TSS, and TKN concentrations are in between typical concentrations for low and medium strength wastewaters. All parameters generally show good correlation with expected values.

Table 5.2-24 Average East Side WWTP Raw Wastewater Concentrations vs. Typical Average Concentrations

Constituent	Average East Side WWTP Concentration (mg/L)	Average Typical Concentrations (mg/L) [1]	
		Low Strength	Medium Strength
Biochemical oxygen demand (BOD_5)	120	110	190
Total suspended solids (TSS)	140	120	210
Total Kjeldahl nitrogen (TKN)	26	20	40

[1] *Low and medium strength wastewater concentrations from* Wastewater Engineering Treatment and Resource Recovery, *5th Ed. (2013).*

Table 5.2-25 compares the WWTP's average influent parameter relationships with typical wastewater ratios from Metcalf & Eddy and MOP-8. The BOD_5:TSS, BOD_5:TKN, BOD_5:TP, and NH3:TKN values are all within the range of typical wastewater constituent ratios over the three year period.

Table 5.2-25 Average East Side WWTP Raw Wastewater Ratios vs. Typical Ratios

Year	BOD_5:TSS	BOD_5:TKN	BOD_5:TP	NH3:TKN
2017	0.91	4.31	N/A	0.61
2018	0.75	4.32	32.63	0.63
2019	1.07	5.54	42.16	0.67
Average	0.91	4.72	37.40	0.64
Typical	*0.82 to 1.43*	*4.2 to 7.1*	*20 to 50*	*0.5 to 0.8*

The concentrations in **Table 5.2-26** are close to those seen in low strength wastewater (previously presented in Table 5.2-24), indicating I/I greatly affects the sewer system, as would be expected in a partially combined collection system. Note that the per capita loads divide the total collection system load by the population and thus include commercial and industrial flows, in addition to residential flows. Typical per capita loads from TR-16 and from Metcalf & Eddy 5th edition are compared to the calculated values in **Table 5.2-27** below.

Table 5.2-26 Influent Wastewater Pollutant Loads and Concentrations

Parameter	Total Influent Load (lbs/day)	Sewer System Per Capita Influent Load (lbs/day)[1]	Sewer System Influent Concentration (mg/L)
BOD_5	5,700	0.15	120
TSS	6,200	0.16	140
TKN	1,200	0.03	26

Table 5.2-27 Comparison of Per Capita Loading to Typical Values

Parameter	TR-16 Typical Loads (lbs/capita/day)	M&E 2003 Typical Loads (lbs/capita/day)	Actual Estimated Loads (lbs/capita/day)
BOD_5	0.17 to 0.22	0.11 to 0.26	*0.15*
TSS	0.20 to 0.25	0.13 to 0.33	*0.16*
TKN[1]	0.04	0.02 to 0.048	*0.03*

All per capita loads are within the Metcalf & Eddy range, but lower than the TR-16 guidelines. This could be a factor of I/I or due to industrial flows with lower loadings than residential flow.

5.2.2.6 Peak Loads and Load Peaking Factors

Table 5.2-28 presents the wastewater load peaking factors calculated from the plant data and compares them to MOP-8 for a combined collection system.

Table 5.2-28 Peaking Factors for Wastewater Influent Loads

Constituent	Maximum Month		Maximum Day		Minimum Day	
	MOP 8	Calculated	MOP 8	Calculated	MOP 8	Calculated
BOD_5	1.26	1.38	1.61	1.90	0.60	0.48
TSS	1.31	1.85	1.88	2.05	0.53	0.34
TKN	1.24	1.38	1.40	1.86	0.67	0.60
TP	1.20	1.66	1.36	2.18	0.73	0.60

The peaking factors for each design condition were calculated as follows:

- *Maximum day peaking factors* for BOD_5, TSS, TKN and TP were calculated by dividing the 95th percentile of the daily load data by the overall average mass load.
- *Maximum month peaking factors* for BOD_5, TSS, TKN and TP were calculated by dividing the 95th percentile of the 30-day rolling average of the daily load data by the overall average mass load.
- *Minimum day peaking factors* were calculated based on the 5th percentile of the daily load data divided by the overall average mass load.

Table 5.2-29 presents the resulting existing average and peak influent loads based on the actual influent data as summarized above.

Table 5.2-29 Existing Average and Peak Influent Loads

Parameter	Load (lb/day)
BOD_5	
Average Daily Load	5,700
Maximum Monthly Load	7,800
Maximum Day Load	11,000
Minimum Day Load	2,700
TSS	
Average Daily Load	6,200
Maximum Monthly Load	11,000
Maximum Day Load	13,000
Minimum Day Load	2,100
TKN	
Average Daily Load	1,200
Maximum Monthly Load	1,600
Maximum Day Load	2,200
Minimum Day Load	720
TP	
Average Daily Load	160
Maximum Monthly Load	270
Maximum Day Load	350
Minimum Day Load	100

5.2.2.7 Removal Efficiencies Across Primary Settling Tanks and Primary Effluent Loads

Treatment process removal efficiencies were estimated across the primary settling tanks (PSTs) and the entire treatment facility over the three-year data set as follows:

- Average BOD_5, TSS, and TKN removal efficiencies across the primary tanks are 11%, 52%, and 14% respectively.
- Average BOD_5, TSS, and TKN removal efficiencies through the entire facility are 95%, 91%, and 89% respectively.

Table 5.2-30 summarizes the calculated primary effluent loads based on existing influent and primary effluent data. TP loadings are not available as it is not sampled from the primary effluent.

Table 5.2-30 Existing Average and Peak Primary Effluent Loads

Parameter	2017-2019 Load (ppd)
Average Day	
BOD_5	4,600
TSS	2,200
TKN	1,000
Maximum Day	
BOD_5	8,900
TSS	4,300
TKN	1,600
Maximum Month	
BOD_5	6,900
TSS	3,800
TKN	1,300

The loads presented in Table 5.2-30 include the loads due to sidestreams. There are three sources of sidestream loading at the East Side WWTP:

- Gravity Thickener Supernatant – discharges into primary effluent channel
- Gravity Belt Thickener Returns – discharges upstream of headworks
- Scum/Skimmings Decant Water (from both WWTPs) – discharges upstream of headworks

The loads for the sidestreams were not quantified due to lack of flow or concentration data, and data inconsistencies. However, since the primary effluent sampler is located downstream of all sidestream addition points, it is assumed that the primary effluent loads account for all influent and sidestream loads.

5.3 Community Growth Projections

This section analyzes projected growth within the East Side and West Side WWTPs service areas and describes how the growth projections were used to develop future flow and load estimates through the year 2050.

A discussion of specific components that could contribute future flows and loads to each plant are presented below including:

- Population Projections
- Residential Flow within Bridgeport
- Inter-municipal Flows
- Industrial, Commercial and Large Residential Flows within Bridgeport

5.3.1 Project Population Growth

The population in Bridgeport has been close to stagnant over the past decade with a total population of 145,000 in 2010 per the United States Census. Current estimates from the Census and from the Connecticut Data Collaborate show a similar population. Per the WPCA and City of Bridgeport, the current population is 147,000 which will be used as the baseline population for 2020. Of the 147,000, 26 percent reside in the East Side WWTP's service area. Therefore, the service area population is estimated to currently be approximately 38,000 for East Side WWTP and 109,000 for the West Side WTTP.

Population is currently about 10 percent lower than the maximum historical population of 159,000 from the 1950s. **Figure 5.3-1** shows the historical population of Bridgeport (US Census) as well as the project population used to determine future flows.

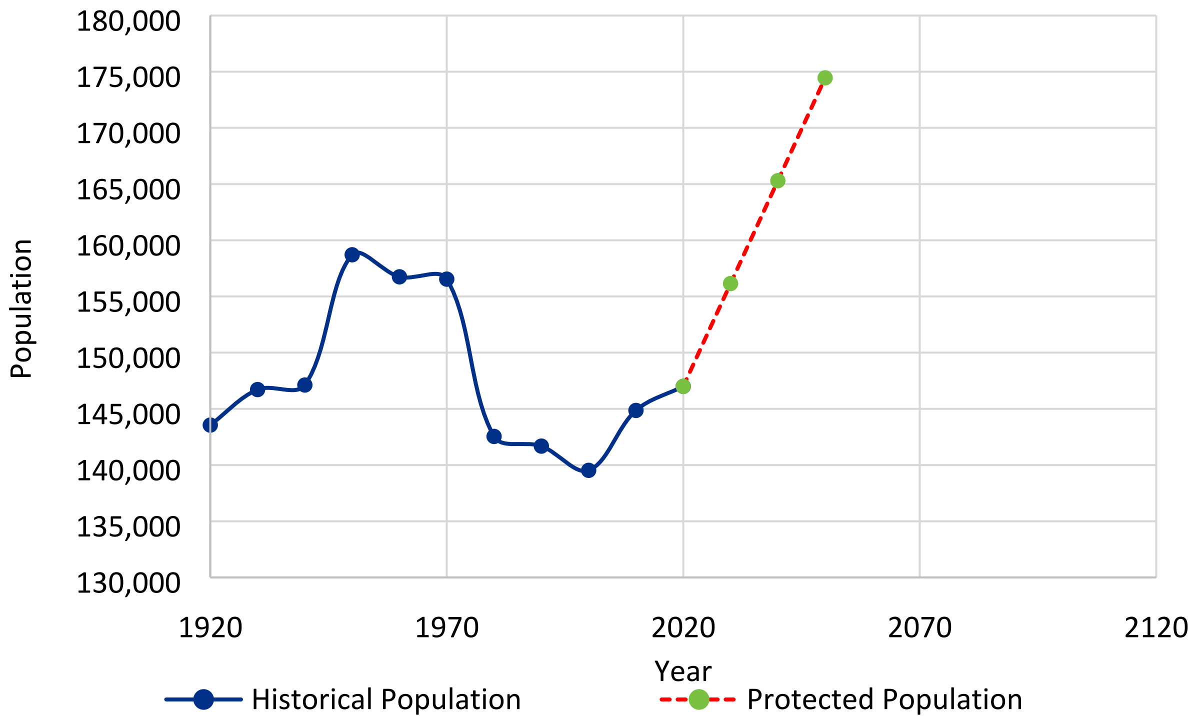

Figure 5.3-1
Bridgeport Historical and Projected Population

Future populations were estimated to increase by about 9,150 over the next decade as noted in Plan Bridgeport, the Master Plan of Conservation and Development for City of Bridgeport, adopted on April 22, 2019. This matches closely with the Connecticut Data Collaborative estimate for 2030. For 2040, the CT Data Collaborative estimated a very small increase in population to approximately 155,000. Plan Bridgeport does not provide population estimates beyond 2030, but in conversations with the WPCA and the City of Bridgeport, they estimate population to continue growing beyond 2030. Therefore, the same population increase used from 2020 to 2030 was used to project population to 2040 and 2050. This estimate shows a sharp increase in comparison to historical growth but will provide reserve treatment capacity. A summary of the population projections is shown in **Table 5.3-1**.

Table 5.3-1 City of Bridgeport Population Projections

Year	Population	Change in Population	% Change from previous
2020 (Existing)	147,000	-	-
2030	156,150	+9,150	+6.3%
2040	165,300	+9,150	+5.9%
2050	174,450	+9,150	+5.6%

Using the resulting sewered population increases, flow increases from growth within the service areas were estimated to be approximately 2.7 mgd. The total existing average daily sanitary flow for both plants (8.7 mgd) was divided by the total existing sewered population (147,000) to arrive at an overall per-capita wastewater flow. The resulting value, 60 gallons per capita per day (gpcpd), is lower than the minimum recommended value by TR-16 of 70 gpcpd. To add another level of conservancy and to account for commercial growth exceeding the rate of residential growth, a value of 100 gpcpd was applied to the population to arrive at a projected flow resulting from infill growth over the planning period.

5.3.2 Residential Flow within Bridgeport

Bridgeport is fully developed so there isn't a possibility of new residential development. Additionally, there are no known septic systems so it is assumed that all future residential flows will come from redevelopment within the current service area.

As mentioned above, the WPCA expects to see a total increase of approximately 27,000 people by 2050. They expect to fulfill this population increase with 1.5 people/household units, mainly in the Downtown area (within half a mile of the train station). At a per-capita wastewater flow of 100 gpcpd, the increase in flow within Bridgeport was estimated at 2.7 mgd.

5.3.3 Inter-municipal Flows

Bridgeport currently receives inter-municipal flows from all neighboring towns. Additionally, to the north of Trumbull is the Town of Monroe which does not currently have any sanitary sewer but desires a future system. Monroe and the three neighboring towns to Bridgeport are discussed below.

5.3.3.1 Stratford

To the east is the Town of Stratford which recently underwent an upgrade to their WWTP; therefore, an inter-municipal agreement is not expected. However, there are approximately 100 direct bill customers in Stratford which discharge to the East Side service area. All direct bill customers are located in fully developed areas of the Town and any new development in Town would be conveyed to Stratford's WWTP. Therefore, no additional inter-municipal sanitary flows were projected.

5.3.3.2 Fairfield

To the West is the Town of Fairfield which has its own WWTP and is mostly developed along the border with Bridgeport; therefore, future inter-municipal flows are not expected. However, there are approximately 70 direct bill customers in Fairfield which discharge to the West Side service area. All direct bill customers are located in fully developed areas of the Town and any new development in the Town would be conveyed to Fairfield's WWTP. Therefore, no additional inter-municipal sanitary flows were projected.

5.3.3.3 Trumbull

To the North is the Town of Trumbull which currently has an inter-municipal agreement to discharge an average of 4.2 mgd. The population in the Town of Trumbull is approximately 36,000 and is expected to remain stagnant or decrease in the future according to the Town's Plan of Conservation and Development. After discussions with the Town of Trumbull, expansion of the collection system within the Town will continue, but some areas will remain without sewer.

The WPCA will plan for a future scenario in which the current daily average limit of 4.2 mgd is held. Any future growth or extension of the sanitary system which exceeds the allocation would need to be offset with corrective measures to reduce I/I. It was assumed that the future sanitary and I/I components of the 4.2 mgd allocation would be 3.2 mgd and 1.0 mgd respectively. This represents an increase in the sanitary component and therefore would contribute higher concentration of BOD_5, TSS, and nutrients. The sanitary flow component increase will be distributed linearly over the planning period. All flow from Trumbull is conveyed through Bridgeport's collection system to the West Side WWTP.

5.3.3.4 Monroe

The Town of Monroe is located to the north of Trumbull and currently does not have a sanitary sewer system. However, per the Town's Plan of Conservation and Development, there is a desire to construct sewer along Routes 25 and 111 to provide sewer service for existing and future commercial businesses. The most logical interconnection would be with the Town of Trumbull and therefore would contribute flow to the West Side WWTP.

While a connection has not yet been approved by Trumbull or Bridgeport, it was assumed that a connection would be made within the planning period. Metcalf and Eddy 5th Edition provides a planning level value of 800 to 1,500 gallons per acre per day of commercial land. The low end of this range was applied to Monroe given its low density of development and uncertainty of future development. From the Plan of Conservation and Development, it was assumed that 2.8 miles of sewer would be constructed to serve 300 acres of commercial land which would be equivalent to 320,000 gpd of sanitary flow. Additionally, 35,000 gpd of additional infiltration due to an expanded sewer system was estimated assuming 1,000 gpd/inch-diameter-mile. It was estimated that two miles of the piping would be 8-inch and the remainder would be 12-inch diameter. In total, an additional average of 360,000 gpd is estimated to contribute to the West Side WWTP from the Town of Monroe by 2050.

5.3.4 Industrial, Commercial and Large Residential Flows within Bridgeport

The City of Bridgeport's Department of Planning and Economic Development provided a list of potential future commercial, industrial, and large residential projects. The list was reviewed, and flow projections were developed for projects that would contribute additional flows to the WWTPs. The following assumptions were used to approximate future flows from these projects.

- Hotel – 56 gallons per guest per day (Metcalf & Eddy 5th Edition)

- School – 12 gallons per student per day (Metcalf & Eddy 5th Edition)
- Theater – 3 gallons per seat per day (Metcalf & Eddy 5th Edition)
- Shopping Center – 0.057 gallons per square foot per day (Connecticut DEEP's Subsurface Sewage Disposal Systems Design Manual, 2006)
- Health Center – 0.62 gallons per square foot per day (Maryland Department of the Environment's Design Guidelines for Wastewater Facilities, 2013)

The total additional flow from the reported potential projects was calculated at 140,000 gpd for all of Bridgeport. Of this, 90,000 gpd is attributable to residential projects and is already accounted for in the population growth previously presented. The difference in total flow and residential flow from these projects is 50,000 gpd which was assumed to cover all projects between 2020 and 2030. Assuming a similar level of development in the two subsequent decades, a total of 150,000 gpd can be attributed to commercial and industrial projects.

However, given the conservatism built into the residential growth projections, no additional flow was included for commercial and industrial development.

5.3.5 Community Growth Summary

The total additional average sanitary flow projected for the East Side WWTP is 680,000 gpd. The total additional average sanitary flow projected for the West Side WWTP is 3.8 mgd.

5.4 Future Predicted Flows and Loads

5.4.1 West Side WWTP

5.4.1.1 Future Flows

Future average and peak flows were determined in a similar manner to the methods presented for the existing flows, including using the same peaking factors. Flows were predicted for the years 2030, 2040, and 2050. The predicted additional average daily flow of 3.8 mgd between now and 2050 accounts for increases in flow due to population and non-residential growth. Since the service area only changes with the addition of a Monroe connection, there is only a slight increase in infiltration. **Table 5.4-1** presents a summary of the estimated average, maximum day, and peak hourly flows for the years 2030, 2040 and 2050.

Table 5.4-1 Existing, 2030, 2040 and 2050 Maximum Day and Peak Hourly Influent Flow Estimates

Parameter	Existing (2017-2019)	2030	2040	2050	"Increase Conveyance" (200 mgd)
Average Day					
Sanitary Wastewater Only	8.3	9.5	10.8	12.0	12.0
Infiltration (average)	11.9	12.0	12.0	12.0	12.0
Total Dry Weather Flow (no inflow)	20.2	21.4	22.7	24.0	24.0
Inflow	1.9	1.9	1.9	1.9	1.9
Average Daily Flow	22.1	23.3	24.6	25.8	25.8
Maximum Day					
Sanitary Wastewater Only	14.1	16.1	18.3	20.4	20.4
Infiltration (maximum)	17.0	17.1	17.1	17.1	17.1
Dry Weather Flow (no inflow)	31.1	33.2	35.4	37.5	37.5
Inflow	11.0	11.0	11.0	11.0	11.0
Maximum Day Flow	42.1	44.2	46.4	48.6	48.6
Peak Hour					
Sanitary Wastewater Only	21.5	30.4	34.4	38.5	38.5
Infiltration (maximum)	17.0	17.1	17.1	17.1	17.1
Dry Weather Flow (no inflow)	38.5	47.4	51.5	55.6	55.6
Inflow	42.7	42.7	42.7	42.7	144.4
Peak Hour Flow	81.2	90.1	94.2	98.3	200.0

The existing plant is rated for a wet weather flow of 90 mgd. CDM Smith also explored the benefits of conveying more flow to the plant with the goal of reducing combined sewer overflows (CSOs) in the service area. A SWMM model was used to model the volume of CSOs at treatment plant capacities of 80, 90, 140, 160, 180, and 200 mgd at the West Plant during a 1-year frequency storm. The resulting CSO volume at each plant capacity is shown in **Figure 5.4-1**. For comparison to the 2050 projected flows, a 200 mgd flow condition, called the "Increase Conveyance (200 mgd)" is shown in Table 5.4-1**.** Additional details on the model configuration and CSO benefits of increased capacity are discussed in Sections 3.6 and 7.1 respectively.

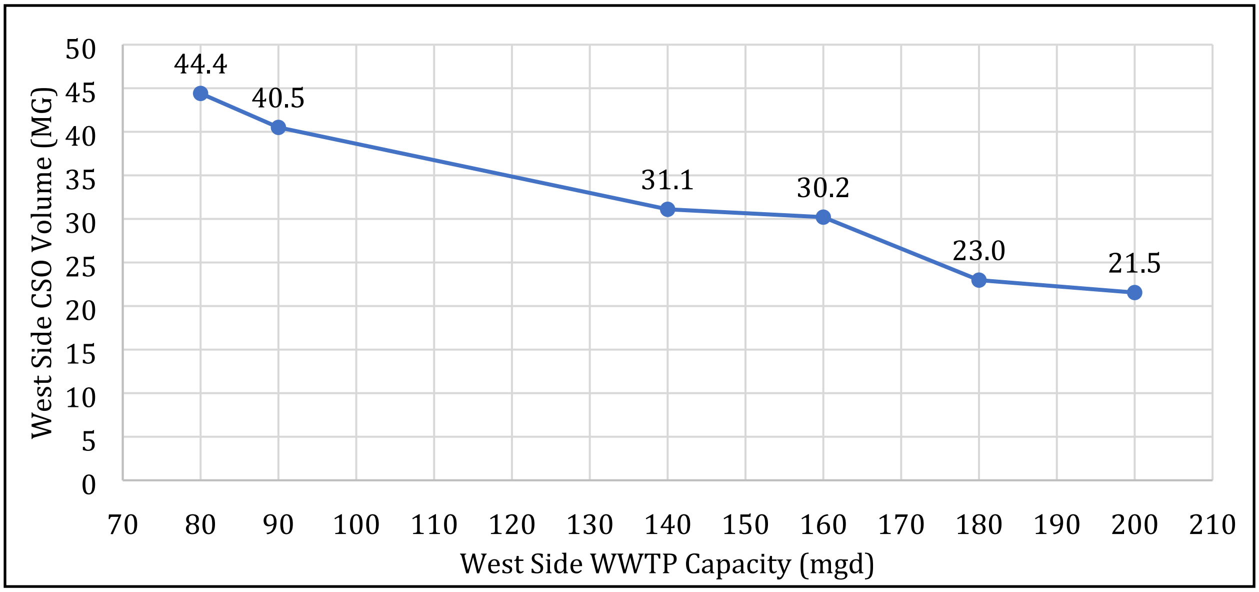

Figure 5.4-1
Simulated West Side WWTP Capacity vs CSO Volume during the 1-year, 24-hour Design Storm

5.4.1.2 Future Loads

Table 5.4-2 presents the estimated loads at two future scenarios compared to the existing loads. The first scenario is for the flows at the 2050 average daily flow of 25.8 mgd. The second scenario maintains the existing design average daily flow of 30 mgd.

Table 5.4-2 Summary of Existing and Future Raw Influent Loads

Parameter	Existing (2017-2019)	2050 (25.8 mgd)	"Existing-Rating" (30 mgd)
Average Day			
BOD_5	28,000	35,000	40,000
TSS	42,000	54,000	62,000
TKN	4,500	5,500	6,300
Phosphorus	780	1,000	1,100
Maximum Day			
BOD_5	55,000	68,000	79,000
TSS	106,000	136,000	156,000
TKN	7,800	9,500	11,000
Phosphorus	1,400	1,800	2,100
Maximum Month			
BOD_5	42,000	52,000	60,000
TSS	80,000	103,000	118,000
TKN	6,400	7,800	9,000
Phosphorus	1,200	1,500	1,700
Minimum Day			
BOD_5	11,000	13,000	15,000
TSS	9,800	13,000	14,000
TKN	2,200	2,700	3,200
Phosphorus	350	440	510

The future average day loads were estimated by multiplying the average concentrations for each parameter by the future average daily flow for both scenarios. The future average loads include septage loads as the average load concentrations were developed using influent sampling and septage impacts as presented for the existing loads. The future peak loads at maximum day and maximum month were estimated by multiplying the future average day loads by the peaking factors from Table 5.2-6.

5.4.1.3 Future Sidestream Loads

Given the uncertainties of the existing sidestream loads, future sidestream loads were estimated at ten percent of the influent loads. **Table 5.4-3** presents the projected sidestream loads.

Table 5.4-3 Projected Sidestream Loads

Parameter	Future Average Sidestream Loads	
	2050 (25.8 mgd)	Existing Rating (30 mgd)
BOD_5	3,500	4,000
TSS	5,400	6,200
TKN	550	630
TP	100	110

5.4.1.4 West Side Facility Influent Design Criteria

CDM Smith recommends that the hydraulic design flow for the facility upgrades be 30 mgd on an average daily flow basis, as this matches the existing design flow and will allow for significant residential and population growth. The secondary treatment hydraulic system capacity will be maintained at 58 mgd. Preliminary, primary, and disinfection treatment capacities will be equivalent to the peak hourly design flow.

Tables **5.4-4** and **5.4-5** present the recommended design criteria for flows and loads based on the analyses presented previously in this section.

Table 5.4-4 Design Flows

Design Influent (mgd)	Design Flows
Average Daily Flow	30.0
Maximum Monthly Flow	40.2
Maximum Day Flow[1]	58.0
Peak Hourly Flow	90 to 200
Minimum Day Flow[2]	14.6

[1] 58.0 mgd represents the secondary treatment hydraulic capacity
[2] 14.6 mgd represents the existing minimum flow at the plant.

Table 5.4-5 Design Loads at 30 mgd

Parameter	Design Influent Loads (lbs/day)
Average Day	
BOD_5	40,000
TSS	62,000
TKN	6,300
TP	1,100
Maximum Day	
BOD_5	79,000
TSS	156,000
TKN	11,000
TP	2,100
Maximum Month	
BOD_5	60,000
TSS	118,000
TKN	9,000
TP	1,700
Minimum Day	
BOD_5	15,000
TSS	14,000
TKN	3,200
TP	510

5.4.1.5 Primary Effluent Design Criteria

The design loads from the primary effluent were established by multiplying the sum of design influent and sidestream loads by traditional primary removal efficiencies presented below in **Table 5.4-6**. Primary removal efficiencies for BOD_5 and TSS were obtained from TR-16. The primary removal efficiency for TKN was taken as the average removal efficiency from the three-year data set for East Side WWTP. Projected primary effluent loads are presented in **Table 5.4-7**.

Table 5.4-6 Traditional Primary Removal Efficiencies

Parameter	Primary Removals	Source
BOD_5	30%	TR-16
TSS	60%	TR-16
TKN	14%	2017-2019 data set

In order to avoid over-sizing, the WWTP's secondary system, the activated sludge process for the upgraded facility will be sized according to treatment objectives of two different flow and loading conditions:

- Condition A: BNR: the biological treatment system will be designed to achieve effluent NPDES limits for BOD_5 and TSS in addition to the effluent TN load from the general permit at the WWTP's projected 2050 design year maximum month conditions.
- Condition B: Conventional Treatment: the biological treatment system will be designed to achieve effluent NPDES limits for BOD_5 and TSS under maximum month conditions at the

WWTP's permitted hydraulic capacity. However, nutrient levels may exceed the TN effluent load limit during cold weather months when flows and loadings exceed the projected 2050 design year maximum month conditions.

The design criteria for each of these conditions is presented in Table 5.4-7.

Table 5.4-7 Future Loads to Secondary System for Two Design Conditions

Parameter	Primary Effluent Loads	
	Condition A: BNR + Conventional Treatment	Condition B: Conventional Treatment (only)
Average Day Flow, mgd	**25.8 mgd**	**30.0 mgd**
BOD_5, lbs/day	27,000	31,000
TSS, lbs/day	24,000	27,000
TKN, lbs/day	5,200	6,000
Maximum Day, mgd	**58.0 mgd**[1]	**58.0 mgd**[1]
BOD_5, lbs/day	53,000	61,000
TSS, lbs/day	60,000	69,000
TKN, lbs/day	9,000	10,000
Maximum Month, mgd	**34.6 mgd**	**40.2 mgd**
BOD_5, lbs/day	40,000	46,000
TSS, lbs/day	45,000	52,000
TKN, lbs/day	7,400	8,600
Minimum Day, mgd	**17.1 mgd**	**19.9 mgd**
BOD_5, lbs/day	12,000	13,000
TSS, lbs/day	7,100	8,200
TKN, lbs/day	2,800	3,300

[1]The maximum day flow is equivalent to the hydraulic capacity of the secondary treatment system.

5.4.2 East Side WWTP

5.4.2.1 Future Flows

Future average and peak flows were determined in a similar manner to the methods presented for the existing flows, including using the same peaking factors. Flows were predicted for the years 2030, 2040, and 2050. The predicted additional average daily flow of 0.7 mgd between now and 2050 accounts for increases in flow due to population and non-residential growth. Since the service area does not change within the planning period and future projects will aim to reduce I/I, infiltration was conservatively assumed to remain the same. **Table 5.4-8** presents a summary of the estimated average, maximum day, and peak hourly flows for the years 2030, 2040 and 2050.

Table 5.4-8 Existing, 2030, 2040 and 2050 Maximum Day and Peak Hourly Influent Flow Estimates

Parameter	Existing (2017-2019)	2030	2040	2050	"Increase Conveyance" (80 mgd)
Average Day					
Sanitary Wastewater Only	2.3	2.6	2.8	3.0	3.0
Infiltration (average)	2.9	2.9	2.9	2.9	2.9
Total Dry Weather Flow (no inflow)	5.2	5.4	5.7	5.9	5.9
Inflow	0.5	0.5	0.5	0.5	0.5
Average Daily Flow	5.7	5.9	6.2	6.4	6.4
Maximum Day					
Sanitary Wastewater Only	4.7	5.1	5.6	6.1	6.1
Infiltration (maximum)	5.1	5.1	5.1	5.1	5.1
Dry Weather Flow (no inflow)	9.8	10.2	10.7	11.2	11.2
Inflow	3.0	3.0	3.0	3.0	3.0
Maximum Day Flow	12.8	13.2	13.7	14.2	14.2
Peak Hour					
Sanitary Wastewater Only	7.5	8.2	9.0	9.7	9.7
Infiltration (maximum)	5.1	5.1	5.1	5.1	5.1
Dry Weather Flow (no inflow)	12.6	13.3	14.1	14.8	14.8
Inflow	15.6	15.6	15.6	15.6	65.2
Peak Hour Flow	28.2	28.9	29.7	30.4	80.0

The existing plant is rated for a wet weather flow of 40 mgd which is greater than the 2050 projected peak hour flow. CDM Smith also explored the benefits of conveying more flow to the plant with the goal of reducing CSOs in the service area. A SWMM model was used to model the volume of CSOs at treatment plant capacities of 35, 40, 60, and 80 mgd at a 1-year frequency storm. The resulting CSO volume at each plant capacity is shown in **Figure 5.4-2**. For comparison to the 2050 projected flows, an 80 mgd flow condition, called the "Increase Conveyance (80 mgd)" is shown in Table 5.4-8**.** It was assumed that any additional flow beyond the 2050 projected flow would be inflow. Additional details on the model configuration and CSO benefits of increased capacity are discussed in Sections 3.6 and 7.1 respectively.

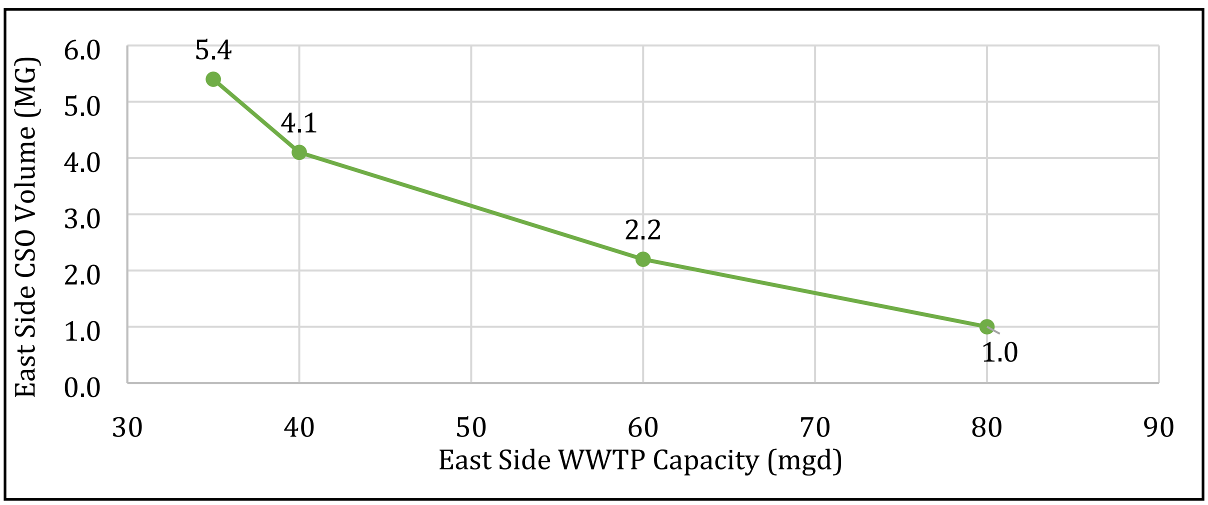

Figure 5.4-2
Simulated East Side WWTP Capacity vs CSO Volume during the 1-year, 24-hour Design Storm

5.4.2.2 Future Loads

Table 5.4-9 presents the estimated loads at two future scenarios compared to the existing loads. The first scenario is for the flows at the 2050 average daily flow of 6.4 mgd. The second scenario maintains the existing design average daily flow of 10 mgd.

Table 5.4-9 Summary of Existing and Future Raw Influent Loads

Parameter	Existing (2017-2019)	2050 (6.4 mgd)	"Existing-Rating" (10 mgd)
Average Day			
BOD_5	5,700	6,400	10,000
TSS	6,200	6,900	11,000
TKN	1,200	1,300	2,100
Phosphorus	160	180	280
Maximum Day			
BOD_5	11,000	12,000	19,000
TSS	13,000	14,000	22,000
TKN	2,200	2,500	3,900
Phosphorus	350	400	620
Maximum Month			
BOD_5	7,800	8,700	14,000
TSS	11,000	13,000	20,000
TKN	1,700	1,900	2,900
Phosphorus	270	300	470
Minimum Day			
BOD_5	2,700	3,000	4,800
TSS	2,100	2,300	3,600
TKN	700	800	1,300
Phosphorus	100	110	170

The future average day loads were estimated by multiplying the average concentrations for each parameter by the future average daily flow for both scenarios. The future peak loads at maximum day and maximum month were estimated by multiplying the future average day loads by the peaking factors from Table 5.2-20.

5.4.2.3 Future Sidestream Loads

Given the uncertainties of the existing sidestream loads, future sidestream loads were estimated at ten percent of the influent loads. **Table 5.4-10** presents the projected sidestream loads.

Table 5.4-10 Projected Sidestream Loads

Parameter	Future Average Sidestream Loads	
	2050 (6.4 mgd)	Existing Rating (10 mgd)
BOD_5	640	1,000
TSS	690	1,100
TKN	130	210
TP	20	28

5.4.2.4 East Side Facility Influent Design Criteria

CDM Smith recommends that the hydraulic design flow for the facility upgrades be 10 mgd on an average daily flow basis, as this matches the existing design flow and will allow for residential and population growth. The secondary treatment hydraulic system capacity will be maintained at 24 mgd. Preliminary, primary, and disinfection treatment capacities will be equivalent to the peak hourly design flow.

Tables **5.4-11** and **5.4-12** present the recommended design criteria for flows and loads based on the analyses presented previously in this section.

Table 5.4-11 Design Flows

Design Influent (mgd)	Design Flows
Average Daily Flow	10.0
Maximum Monthly Flow	14.6
Maximum Day Flow[1]	24.0
Peak Hourly Flow	40.0 to 80.0
Minimum Day Flow[2]	3.3

[1] 24.0 mgd represents the secondary treatment hydraulic capacity
[2] 3.3 mgd represents the existing minimum flow at the plant.

Table 5.4-12 Design Loads at 10 mgd

Parameter	Design Influent Loads (lbs/day)
Average Day	
BOD_5	10,000
TSS	11,000
TKN	2,100
TP	280
Maximum Day	
BOD_5	19,000
TSS	22,000
TKN	3,900
TP	620
Maximum Month	
BOD_5	14,000
TSS	20,000
TKN	2,900
TP	470
Minimum Day	
BOD_5	4,800
TSS	3,600
TKN	1,300
TP	170

5.4.2.5 Primary Effluent Design Criteria

The design loads from the primary effluent were established by multiplying the sum of design influent and sidestream loads by the traditional primary removal efficiencies presented below in **Table 5.4-13**. Primary removal efficiencies for BOD_5 and TSS were obtained from TR-16. The primary removal efficiency for TKN was taken as the average removal efficiency from the three-year data set.

Table 5.4-13 Traditional Primary Removal Efficiencies

Parameter	Primary Removals	Source
BOD_5	30%	TR-16
TSS	60%	TR-16
TKN	14%	2017-2019 data set

In order to avoid over-sizing, the WWTP's secondary system, the activated sludge process for the upgraded facility will be sized according to treatment objectives of two different flow and loading conditions:

- Condition A: BNR: the biological treatment system will be designed to achieve effluent NPDES limits for BOD_5 and TSS in addition to the effluent TN load from the general permit at the WWTP's projected 2050 design year maximum month conditions.
- Condition B: Conventional Treatment: the biological treatment system will be designed to achieve effluent NPDES limits for BOD_5 and TSS under maximum month conditions at the WWTP's permitted hydraulic capacity. However, nutrient levels may exceed the TN effluent load limit during cold weather months when flows and loadings exceed the projected 2050 design year maximum month conditions.

The design criteria for each of these conditions is presented in Table **5.4-14**.

Table 5.4-14 Future Loads to Secondary System for two Design Conditions

Parameter	Primary Effluent Loads	
	Condition A: BNR + Conventional Treatment	Condition B: Conventional Treatment
Average Day Flow, mgd	**6.4 mgd**	**10 mgd**
BOD_5, lbs/day	4,900	7,700
TSS, lbs/day	3,100	4,800
TKN, lbs/day	1,300	2,000
Maximum Day, mgd	**24.0 mgd**[1]	**24.0 mgd**[1]
BOD_5, lbs/day	9,300	14,500
TSS, lbs/day	6,300	9,800
TKN, lbs/day	2,400	3,700
Maximum Month, mgd	**9.3 mgd**	**15.6 mgd**
BOD_5, lbs/day	6,700	10,500
TSS, lbs/day	5,600	8,800
TKN, lbs/day	1,800	2,700
Minimum Day, mgd	**3.7 mgd**	**5.8 mgd**
BOD_5, lbs/day	2,600	4,000
TSS, lbs/day	1,200	1,900
TKN, lbs/day	800	1,300

[1]The maximum day flow is equivalent to the hydraulic capacity of the secondary treatment system.

5.4.3 Plant Consolidation

This Facility Plan includes an evaluation of consolidating a portion or all of WWTP facilities. To assist in evaluating consolidation alternatives, **Table 5.4-15** summarizes the design flows and loads at each WWTP and a fully consolidated WWTP. The flows and loads for alternatives can be determined by adding together the respective values. The consolidation alternatives are presented in Section 6.

Table 5.4-15 Design Flows and Loads for Plant Consolidation

Parameter	East Side WWTP	West Side WWTP	Consolidated WWTP
Average Day, mgd	**10.0 mgd**	**30.0 mgd**	**40.0 mgd**
BOD_5, lbs/day	10,000	40,000	50,000
TSS, lbs/day	11,000	62,000	73,000
TKN, lbs/day	2,100	6,300	7,400
TP, lbs/day	280	1,100	1,400
Maximum Day, mgd	**22.3 mgd**	**57.3 mgd**	**79.6 mgd**
BOD_5, lbs/day	19,000	79,000	98,000
TSS, lbs/day	22,000	156,000	178,000
TKN, lbs/day	3,900	11,000	14,900
TP, lbs/day	620	2,100	2,700
Maximum Month, mgd	**14.6 mgd**	**40.2 mgd**	**54.8 mgd**
BOD_5, lbs/day	14,000	60,000	74,000
TSS, lbs/day	20,000	118,000	138,000
TKN, lbs/day	2,900	9,000	11,900
TP, lbs/day	470	1,700	2,200
Minimum Day, mgd	**3.3 mgd**	**14.6 mgd**	**17.9 mgd**
BOD_5, lbs/day	4,800	15,000	19,800
TSS, lbs/day	3,600	14,000	17,600
TKN, lbs/day	1,300	3,200	4,500
TP, lbs/day	170	510	680
Peak Hour, mgd	**40 - 80 mgd**	**90 - 200 mgd**	**130 - 280 mgd[1]**

[1] The consolidated peak hour flow may not be equivalent to the peaks at the two WWTPs as the peaks may not occur simultaneously. If a consolidated option is selected, the peak hour rating should be further refined.